The viability of using the stormwater ponds on the Diep River in the Constantia Valley for stormwater harvesting

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Declaration

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Abstract

Harvesting stormwater to supplement water demands has attracted a growing interest in South Africa as concerns over the security of the country's water supply increase. Whilst stormwater harvesting has been shown to offer a viable alternative water resource, there are often concerns about its storage requirements due to space constraints in urban areas. Stormwater ponds offer a potential solution to these concerns. Since stormwater ponds are typically designed for the sole responsibility of attenuating the periodic peak stormwater flows that are associated with large storm events, they often remain underutilised. By introducing Real Time Control (RTC) systems to operate stormwater pond outlets, ponds could potentially be used to store stormwater. This could increase the benefits that stormwater ponds provide as well as offer a viable alternative water resource.

To investigate the economic viability of harvesting stormwater from existing stormwater ponds, a case study was performed on a representative urban catchment – the Diep River subcatchment, located in Cape Town, South Africa. The catchment contains seven stormwater ponds, which could be retrofitted for harvesting purposes. Sixteen different stormwater harvesting scenarios were developed that modelled various non-potable demands in the vicinity as well as different storage and harvesting arrangements, created using RTC strategies, of the catchment's existing ponds. These scenarios were modelled using an assortment of modelling tools which include: a catchment stormwater model; water distribution network models; and a Life Cycle Cost Analysis (LCCA).

The economic viability of harvesting stormwater from the Diep River subcatchment's stormwater ponds was most susceptible to the cost of the system's water distribution infrastructure. Consequently, stormwater harvesting was most economically viable if used to supply toilet, clothes washing and irrigation demands to residential properties situated in close vicinity to the system's harvesting pond as this minimised the extent of the water distribution network. The results also revealed that distributing storage amongst ponds situated throughout the catchment is an effective method of increasing the volume of stormwater a stormwater harvesting system could yield without reducing its economic viability. However, this is on the condition that the system only extracts stormwater from the most downstream pond in the catchment. Importantly, the study also revealed that the attenuation of peak flows of large storm events (up to 1-in-20 year return period), achieved when harvesting stormwater from the existing stormwater ponds would be comparable to what the ponds currently provide.

The study concluded that harvesting stormwater from existing stormwater ponds is potentially viable. It also demonstrated an effective method to maximise a catchment's storage capacity using distributed storage. For stormwater harvesting to be viable however, stormwater should be used to supplement a large percentage of non-potable end-uses and requires significant uptake amongst catchment residents.

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Glossary

This glossary contains terms relating to stormwater management, engineering and geographical information systems that are used throughout this dissertation. The sources that were utilised for the compilation of this glossary were Woods-Ballard et al. (2007), Vice, (2011), Armitage et al. (2013), SANRAL, (2013) and Fisher-Jeffes (2015).

Anaerobic is a state in which there is an absence of oxygen in any form

Attenuation is defined as the reduction of the peak stormwater flow

Catchment is an enclosed area in which any rainfall will drain to a common point or area through surface flow

Channel is a natural or artificial watercourse through which water flows perpetually or periodically and forms connecting links between waterbodies

Conduit refers to infrastructure that is used to convey stormwater

Detention pond is usually a depression that is used to store stormwater runoff temporarily to attenuate peak floods

Disaggregation is a process to divide or separate an object or information into smaller information units

Filtration refers to the filtering of stormwater runoff pollutants that are conveyed with sediment by trapping these constituents in vegetation species or soil matrices

Geographical Information System (GIS) is a system designed to capture, store, manipulate, analyse, manage and present geographical information

Impervious surface refers to a surface that retards the entry of water into upper soils layers. Typical examples include: roads, parking areas, sidewalks, etc.

Infiltration is a process in which runoff permeates through the upper soil layers and into the earth's surface

Pathogens are bacteria, viruses or microorganisms that can cause disease

Peak flow is the maximum flow rate of water passing a given point during or immediately after a storm event

Perennial stream is a watercourse that has continuous flow in its stream bed throughout the year in years of normal rainfall

Potable describes water that is safe to drink

Recurrence interval or return period is the average time-interval in between hydrological events of a particular magnitude

Retention pond is an artificial lake or pond-like structure that retains a permanent body of water. It is used to attenuate storm events whilst retaining water for a sufficient time to allow

enhancement of the water quality through sedimentation and/or provide an amenity or opportunity to encourage some biodiversity

Runoff coefficient is an integrated value that represents the most significant factors influencing the rainfall-runoff relationship. It represents the part of the storm rainfall that contributes to the peak flood runoff at the outlet of the catchment

Sedimentation is the deposition of soil particles that were suspended within the water. This process occurs when water velocity decreases to below the minimum transportation velocity

Shapefile is a popular geospatial vector data format for geographic information system software

Stormwater is the water that accumulates from natural precipitation, which includes rainwater, groundwater and spring water

Stormwater conveyance network is the infrastructure system used to convey collected stormwater from a drainage area to a specified disposal point. It includes pipes, channels and other flow control facilities

Stormwater harvesting is the collection and storage of stormwater that is then treated and distributed to users who can use the stormwater for an appropriate purpose. The harvested stormwater is typically used to substitute non-potable demands

Stormwater network refers to the system used to manage, convey, treat and dispose of stormwater. It can consist of constructed or natural facilities

Stormwater ponds are basins that are constructed to attenuate peak storm discharges. Alternative approaches to stormwater management (e.g. SuDS) have expanded the objectives of stormwater ponds to include: water treatment, provision of recreational and aesthetic amenity and the preservation of biodiversity

Sustainable Drainage Systems (SuDS) incorporate stormwater management practices that are intended to handle and drain surface runoff in a manner that is more sustainable than conventional stormwater management practices

Surface runoff refers to the accumulation of stormwater that runs over the ground surface

Time of concentration is defined as the minimum time for runoff from the entire catchment to contribute to the flow at the point of interest

Watershed refers to the boundary that separates a catchment area from adjacent catchments

Wetland is a land area that is saturated with shallow water either permanently or seasonally and supports the growth of bottom-rooted aquatic vegetation

Acronyms and abbreviations

AADD	Average Annual Daily Demand
ARC	Agriculture Research Council
BLRPM	Bartlett-Lewis Rectangular Pulses Rainfall Model
CHI	Computational Hydraulic Institute
CoCT	City of Cape Town
CPAP	Contract Price Adjustments Provisions
CSIR	Council for Scientific and Industrial Research
DAF	Dissolved Air Flotation
DoCOGTA	Department of Cooperative Governance and Traditional Affairs
DTM	Digital Terrain Model
DWA	Department of Water Affairs
DWAF	Department of Water Affairs and Forestry
EAC	Equivalent Annualised Cost
EMC	Event Mean Concentration
GIS	Geographical Information System
GSM	Global System for Mobile Communication
IDF	Intensity-Duration-Frequency
IPF	Instantaneous Peak Factor
LAC	Local Automatic Control
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LiDAR	Light Detection and Ranging
LPV	Little Princess Vlei
MAP	Mean Annual Precipitation
MAR	Managed Aquifer Recharge
MDB	Multi-Purpose Detention Basins
MOUSE	Model for Urban Sewers
MPCA	Minnesota Pollution Control Agency
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
MWD	Monthly Water Demand

NOAA	National Oceanic and Atmosphere Administration
NPV	Net Present Value
OIT	US Department of Energy's Office of Industrial Technologies
PATC	Professional Accountants and Tax Consultants
PLC	Programmable Logic Controllers
PVF	Princess Vlei Forum
RBC	Rule-Based Control
RC	Regional Control
REUM	Residential End-Use Model
RTC	Real Time Control
SANRAL	South African National Roads Agency
SAWS	South Africa Weather Services
SCADA	Supervisory Control and Data Acquisition
SCS	Soil Conservation Service
SLAMM	Source Loading And Management Model
SRP	Single Residential Property
SRTC	Sensitivity-based Radio Tuning Calibration
StatsSA	Statistics South Africa
SuDS	Sustainable Drainage Systems
SWMM	Storm Water Management Model
TP	Total Phosphorus
TSS	Total Suspended Solids
UDB	Uni-Use Detention Basins
uPVC	Unplasticized Polyvinyl Chloride
US EPA	United States Environmental Protection Agency
UV	Ultraviolet
UVQ	Urban Volume and Quality
UVT	Ultraviolet Transmittance
VAT	Value Added Tax
WERF	Water Environment and Reuse Foundation
WWO	World Weather Online
ZAR	South Africa Rands

Symbols

C	Expenditure (ZAR)
Cap	Capital Cost
DF	Discount factor
D_t	Water demand during time t
$DWD_{Pond's}$ suburb	Daily water demand of a SRP in a harvesting pond's respective suburb
E	CoCT electricity tariff
e	End-use
EAC	Equivalent Annual Cost
f_d	The total duration in which the target water demand is not fully met
$f_{m,\ garden\ irrigation}$	Garden irrigation factor per month
$f_{m,\ pool\ evaporation}$	Pool evaporation factor per month
f_s	The number of continuous periods in which the target water demand is not fully met
g	Gravitational constant
h	Pumping head
i	Real discount rate
I_t	Volume of water entering storage unit
k	Crop factor or pool factor
kℓ	Kilolitres
M	Month
m^2	Square metres
m^3	Cubic metres
m^3/s	Cubic metres per second
Mℓ	Megalitres
N	Number of time-steps in which the target water demand was fully met
n	Number of years from present year
$No.\ years$	Total number of years in life cycle analysis
$No.\ of\ SRPs$	Number of SRPs a harvesting pond can supply during a 'dry' period
NPV	Net Present Value
o	Outdoor

$O\&M$	Operation and Maintenance costs
O_T	Overflow ratio
$p_{A\text{-}pan}$	Pan evaporation
Q	Water demand of system per time-step
r	Effective Rainfall
R	Monthly Rainfall
R^2	Coefficient of determination
Res	Residual cost
R_T	Time-based reliability
R_v	Volumetric Reliability
$s_{m,\ garden\ irrigation}$	Irrigation surface area per month
$s_{m,\ swimming\ pool}$	Swimming pool surface area per month
S_t	Volume of water spilled by storage unit
t	Time (hours, minutes, seconds)
T	Total number of time-steps in simulation period
$Total\ no.\ of\ SRPs$	Total number of SRPs considered in scenario
$TWD_{'dry'\ period}$	Total water demand of a SRP during a ‘dry’ period
$V_{harvesting\ pond}$	Harvestable volume
Y_t	The yield during time t
η	Pump efficiency factor
ρ	Density of water
ϕ	Resilience

1. Introduction

1.1 Background to study

Throughout much of the world, urbanisation, concomitant with rapid population growth and heightened standards of living, along with ineffective management practices of freshwater resources, has shifted the urban water cycle to an unsustainable position characterised by water scarcity (Gleick, 1998; Fletcher *et al.*, 2007; Carden *et al.*, 2016). At the same time, the expansion of urban areas has led to the replacement of natural landscapes by impermeable surfaces. This has caused an increase in both the volume and peak flow rate of stormwater runoff which is furthermore highly polluted. Conventional drainage systems, which are designed to rapidly convey runoff to the nearest receiving water typically using impervious conduits, have exacerbated the problem by shortening the time of concentration thus further raising the flood peaks. The consequence of all of this is considerable damage to downstream aquatic systems (Fletcher *et al.*, 2007; Burns *et al.*, 2012; Armitage *et al.*, 2013). The traditional opinion of stormwater management stakeholders that stormwater is a 'nuisance' has led to the many shortcomings of conventional drainage systems; however, stormwater's latent value is becoming increasingly recognised with the burgeoning threat to water security (Fletcher *et al.*, 2007; McArdle *et al.*, 2011; Burns *et al.*, 2012).

In recent years, an alternative approach to stormwater management has been developed in order to mitigate the negative impacts that conventional drainage systems and the expansion of urban land take can have on stormwater runoff (Mitchell *et al.*, 2007). This alternative approach to stormwater management, referred to as Sustainable Drainage Systems (SuDS) in South Africa, aims to manage stormwater in a holistic and sustainable manner by incorporating stormwater practices that aim to replicate the natural hydrological cycle (Woods-Ballard *et al.*, 2007; Gaborit *et al.*, 2012; Armitage *et al.*, 2013). Furthermore, SuDS have expanded stormwater management objectives beyond merely flood protection to include other priorities such as: pollution control, preservation of biodiversity and the creation of aesthetic and recreational amenity (Nascimento *et al.*, 1999; Mitchell *et al.*, 2007; Rezende *et al.*, 2011; Armitage *et al.*, 2013; Shamsudin *et al.*, 2014). Nevertheless, despite the development of a more sustainable and holistic approach to managing stormwater, stormwater management is still not fully integrated into the entire urban water sector. Instead, it is still commonly managed in isolation from other components of the urban water cycle (Mitchell *et al.*, 2007; Bahri, 2012).

Concerns over future water security has led to the need to augment traditional water supplies with alternative water sources (Coombes & Barry, 2007; Fletcher *et al.*, 2007; Mitchell *et al.*, 2007). Since stormwater is a relatively abundant local source of water, interest is growing in making use of this alternative water resource for potable water substitution – ultimately enhancing stormwater management's integration amongst other sectors of urban water management (Mitchell *et al.*, 2006). Furthermore, certain SuDS practices, stormwater ponds in particular, are well suited to incorporate the harvesting of stormwater (Mitchell *et al.*, 2007). Stormwater ponds address one of the common concerns about stormwater harvesting systems –

storage requirements. Stormwater ponds are designed to attenuate peak stormwater flows through the provision of temporary storage (Nascimento *et al.*, 1999; Shamsudin *et al.*, 2014). Consequently, stormwater ponds may have the potential to store stormwater for harvesting purposes. At the same time, harvesting stormwater from ponds would extend their functionality beyond just flood control, which is a key objective of SuDS practices.

1.2 Purpose of the research

Currently, the City of Cape Town (CoCT) manages some 800 stormwater ponds of which many were designed only to provide flood protection, effectively disregarding the other SuDS objectives (Haskins, 2012; Rohrer, 2014). This leaves the CoCT with a sizeable capital infrastructure investment that could be significantly improved by increasing its functionality. Furthermore, like many South African cities, the CoCT's water security is in precarious position; conservative predictions estimate Cape Town's water demand will outstrip its bulk water supply by 2025 (Western Cape Government, 2005; De Sousa-Alves *et al.*, 2015). Stormwater harvesting presents the CoCT an opportunity to improve their water security whilst also getting added benefits from their stormwater ponds.

This dissertation investigates whether retrofitting stormwater ponds within an urban catchment for stormwater harvesting purposes is a viable method of creating an alternative water resource. The catchment chosen for investigation was the Diep River subcatchment – part of the greater Sand River catchment – located in the Southern Suburbs of Cape Town. The choice of catchment was predominantly based on: the number of stormwater ponds it contained – which included seven ponds of various types and sizes; convenience of the catchment's location; its extent – it covers an area of 20 ha. – and the variability of affluence amongst people residing in the catchment – level of wealth will have an effect on how water is used on a property. The assessment was made using a selection of modelling tools which included: a long-term continuous *PCSWMM 6.2* hydrological model of the Diep River subcatchment, *EPANET 2.0* water distribution network models; and a Life Cycle Cost Analysis (LCCA). The hypothesis is as follows:

A viable alternative water resource can be created for the Diep River subcatchment by retrofitting its stormwater ponds – which are situated along the Diep River and its tributaries in the Constantia Valley – for stormwater harvesting purposes.

In an effort to prove this hypothesis, the research investigated in what manner could stormwater harvesting be incorporated into the stormwater ponds along the Diep River to provide a viable alternative water resource for either residential or commercial usage in the surrounding area. The study had the following scope and limitations:

i) This study only considered increasing the functionality of the catchment's stormwater ponds through stormwater harvesting.

ii) The study focussed solely on the quantifiable results of stormwater harvesting, which were: the economic viability, average annual harvesting yield, volumetric reliability, overflow ratio, the resilience of the system and stormwater management implications.

iii) The availability of data proved to be a significant limitation throughout the entirety of this study. For several aspects of this research, it was not possible to acquire data at the level of detail that was required, hence limiting the accuracy of the research. The specific areas in which data was lacking included: long-term (greater than ten years) sub-hourly rainfall data, long-term sub-hourly flow data, evaporation data, water end-use demand data, and life cycle cost analysis data.

The study showed that harvesting stormwater from catchment's stormwater ponds is economically viable, but would require significant uptake amongst catchment residents.

1.3 Chapter outline

This dissertation consists of seven chapters, a list of references and nine appendices. The chapter outline is detailed below:

Chapter 1 introduced the purpose of the dissertation; it provided a contextual background, the research objectives, and the limitations of the study.

Chapter 2 provides a literature review that encapsulates the following: South Africa's current water concerns, stormwater harvesting systems, stormwater ponds; and methods to model, analyse and improve a stormwater harvesting system.

Chapter 3 aims to acquaint the reader with the Diep River subcatchment – the chosen catchment for this study. The chapter describes the catchment in regards to: its location, demographic, climate and rainfall characteristics, and the history of its stormwater ponds.

Chapter 4 is a discussion of the method of research employed for this study. It describes the data acquisition process, the construction of the catchment stormwater model, the development of the modelled stormwater harvesting scenarios and an explanation of the economic analysis the study incorporated.

Chapter 5 presents and discusses the results of this research. By analysing the performance measures of the stormwater harvesting scenarios, the chapter discusses the viability of harvesting stormwater from the Diep River subcatchment stormwater ponds.

Chapter 6 concludes the dissertation, providing a concise summary of its the findings.

Chapter 7 offer a list of recommendations for future research**.**

The **Appendices** provide further documentation to support the findings present in the body of the dissertation.

2. Literature review

Retrofitting stormwater ponds for harvesting purposes could offer an opportunity to get added benefits from stormwater ponds whilst at same time alleviate growing water concerns. In order to evaluate the viability of this opportunity, it is necessary to understand the various factors that would be involved. This chapter serves to provide a brief overview of the water crisis South Africa is facing, the components of a stormwater harvesting system, the three basic types of stormwater ponds and methods of modelling, analysing and improving the performance of a stormwater harvesting system.

2.1 The threat of water scarcity

Water scarcity is recognised as potentially the most significant constraint to future human-development prospects (Appelgren & Klohn, 1999; Noemdoe *et al.*, 2006; Veldkamp *et al.*, 2015). Water is essential to maintaining life on Earth; not only is it vital to human health, it is indispensable to the success of all ecological functions, economies, and communities (Gleick, 1998; Rockström *et al.*, 2014). Whilst it may appear that freshwater is in abundance, its availability is limited. Of all the water present on the Earth's surface, less than 1% of the total volume is readily available as freshwater resources (Oki & Kanae, 2006). Escalating water demands caused by population growth and higher standards of living are decreasing the security of the world's limited water supply (Gleick, 1998; Walsh *et al.*, 2012). This security is put in further doubt due to the uncertainty of future climatic conditions as well as the deterioration of the quality of existing freshwater supplies that has resulted from conventional urban stormwater management practices (Armitage *et al.*, 2014).

Whilst water scarcity is considered a global concern, the issue is particularly prevalent in South Africa (DWAF, 2004; Noemdoe *et al.*, 2006). Situated primarily in a semi-arid climate, South Africa is ranked as the 30th driest country in the world with an average annual rainfall of approximately 450 mm that is well below the world average of 860 mm per year (Hedden & Cilliers, 2014). This rainfall is not evenly distributed across the country, which leaves numerous areas with limited water (DWAF, 2004). Furthermore, in South Africa's pursuit to support a dynamic and growing economy, the country has already allocated 98% of its available water resources (Muller *et al.*, 2009; Hedden & Cilliers, 2014).

Current water demand forecasts predict that South Africa's water demand will surpass available supplies as early as 2025 (Muller *et al.*, 2009). Hence, it is clear that the country's water supply is in a precarious position. This point was highlighted in 2015 after a severe drought left South Africa in the midst of a water crisis. The drought caused five of the seven South African provinces to be declared 'drought disaster areas' whilst numerous areas around the country had to implement severe water restrictions (Hawker, 2015; Ngoepe, 2015; Singh, 2015). Thus, if South Africa is to ensure water security and to achieve social and economic prosperity, it will require more effective management of its existing water systems than at present (Muller *et al.*, 2009; Armitage *et al.*, 2014).

2.2 Using harvested stormwater to augment water supplies

As many countries around the world, including South Africa, are beginning to reach the limits of their sustainable water supply, governments should be considering incorporating alternative water resources to augment existing water supplies (Fletcher *et al.*, 2007). Currently, water demand management is the most common strategy that is used to reduce water use and enhance water supplies. Water demand management involves practices such as educational awareness campaigns, stepped water tariffs and the imposition of restrictions to curb water demand (Coombes & Barry, 2007; Bahri, 2012; Armitage *et al.*, 2014). Whilst water demand management has been recognised as an important tool in the effort to ensure the longevity of our traditional water supplies, ultimately, this approach cannot sustain the growing water demand (Coombes & Barry, 2007). Inevitably, alternative water sources need to be incorporated into the urban water cycle to provide more resilient water supplies. Since, desalination and groundwater harvesting both have significant potential negative issues (e.g. high cost and the risk of aquifer contamination, respectively), the viability of stormwater harvesting is attracting a growing interest throughout the world (Fletcher *et al.*, 2007; Hagare *et al.*, 2013).

Stormwater harvesting is a practice that involves the collection, storage, treatment and distribution of rainfall-runoff with the intention of using the harvested water as a non-traditional water resource (Mitchell *et al.*, 2008; Philp *et al.*, 2008; Sydney Water, 2013; Fisher-Jeffes, 2015). It is considered a developing field that is capable of achieving a range of water conservation, water quality and environmental flow objectives (Brodie, 2008; Fisher-Jeffes, 2015). Stormwater can technically be used supplement any end-use demand; however, stormwater runoff from urban areas is often highly polluted and thus requires some form treatment to ensure it is of an acceptable standard for its intended end-use (Mitchell *et al.*, 2006; Akram *et al.*, 2014). For this reason, harvested stormwater is typically only used for non-potable end-uses such as garden irrigation, toilet flushing, clothes washing etc. as they require water of a lower quality and subsequently lower treatment costs. While both Hatt *et al.* (2006) and Mitchell *et al.* (2007) affirm that much progress has been made in stormwater harvesting, the field is lacking extensive research that identifies under which conditions stormwater harvesting is viable.

2.3 Elements of a stormwater harvesting system

Stormwater harvesting systems comprise of four basic elements: collection, treatment, storage and distribution. This section will describe each of these elements.

2.3.1 Collection

Stormwater infrastructure conveys runoff using either conventional drainage infrastructure (e.g. subterranean pipes or channel systems), alternative stormwater practices (e.g. swales, buffers or

biofiltration systems) or a combination of both. Stormwater harvesting systems use stormwater conveyance infrastructure to collect and convey runoff to a storage facility. There are benefits and limitations associated to collecting runoff from either conventional drainage infrastructure or alternative stormwater practices since both types of infrastructure have different effects on the manner in which stormwater is conveyed.

Conventional drainage systems are designed to convey stormwater to the nearest receiving water as rapidly and efficiently as possible. This is typically performed using concrete pipes and channels that are only susceptible to minor water losses that typically occur through cracks in the walls of drainage infrastructure. Furthermore, conventional drainage systems are designed to handle storm events with up to a two-year rainfall return period. According to Mitchell *et al.* (2007) and Wong *et al.* (2000) rainfall events with less than a one-year return period produce the highest proportion of the total volume of runoff. For these reasons, conventional drainage systems are able to convey a large percentage of total runoff to a stormwater harvesting system's storage facility (Hatt *et al.*, 2006; Philp *et al.*, 2008; Fisher-Jeffes, 2015). However, conventional drainage systems provide negligible treatment to stormwater runoff and are thus likely to carry highly polluted stormwater into the storage facility. This is undesirable as the collected stormwater will then require considerable treatment before distribution (Hatt *et al.*, 2006; Mitchell *et al.*, 2007; Philp *et al.*, 2008).

Collecting stormwater using alternative stormwater practices (e.g. swales, buffers or biofiltration systems) offer various benefits and limitations compared with conventional infrastructure. Since alternative stormwater practices make use of vegetated systems, they experience substantial water losses through exfiltration and evapotranspiration, which reduces the volume of harvestable runoff. Mitchell *et al.* (2007) states the severity of these losses varies depending on factors such as local climate, soils type and the surface area of the alternative stormwater practice. However, exfiltration losses can be prevented by lining the systems. Unlike conventional drainage infrastructure however, alternative stormwater practices are capable of removing many stormwater contaminants. Thus, these systems improve the quality of water entering the storage facility potentially reducing further treatment requirements (*ibid.)*.

The manner in which stormwater is collected should consider, *inter alia*, the land-use from where the stormwater it conveys originates. Pollution loading is highly dependent of the catchment's land-use characteristics; a fact illustrated by the land-use restrictions imposed in Singapore. In Singapore, stormwater is harvested to supply potable uses. In order to improve the quality of harvested stormwater, the Singapore government prevents certain land uses that are linked to poor quality (e.g. industrial areas), from being developed in stormwater harvesting catchments. An effective stormwater harvesting system should have a collection system that balances the benefits of conventional drainage systems and alternative drainage devices in order to yield runoff of an optimal quantity and quality (Philp *et al.*, 2008; Fisher-Jeffes, 2015).

2.3.2 Treatment

Harvested stormwater is used on a 'fit-for-purpose' basis. In regards to water management, the term 'fit-for purpose' refers to varying the quality of water used to match the minimum quality requirements of the water's end-use (Fisher-Jeffes, 2015). Thus, the treatment that harvested stormwater requires is dependent on its intended end-use (Philp *et al.*, 2008; Akram *et al.*, 2014; Armitage *et al.*, 2014). There are two main methods of treating collected stormwater, which are broadly defined by the following:

- **Sustainable Drainage Systems (SuDS) treatment** – is an alternative drainage strategy that incorporates technologies that focus on minimising the negative effects of stormwater on receiving waters (Mitchell *et al.*, 2006; Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). Whilst this alternative drainage strategy is referred to as SuDS in South Africa and the United Kingdom, it is known as Low Impact Development (LID) in the United States of America, Water Sensitive Urban Design (WSUD) in Australia and Low Impact Urban Design and Development (LIUDD) in New Zealand. A number of SuDS practices (e.g. sand filters, swales and buffers, infiltrations systems, etc.) are typically implemented in a sequential arrangement known as a 'treatment train' for pollution control (Hatt *et al.*, 2006; Armitage *et al.*, 2013). Whilst the composition of SuDS practices within a 'treatment train' is case specific, its overall aim is to improve the quality of runoff to a suitable standard before it enters the stormwater harvesting system's storage facility. Additionally, the storage facility itself can form part of the 'treatment train' if a SuDS device such as a retention pond or wetland is used (described in Section 2.4.2 and 2.4.3, respectively). The treatment that SuDS devices provide to stormwater occurs through various different physical, chemical, and/or biological treatment processes (Fisher-Jeffes, 2015). Unfortunately, their pollutant reduction capability is variable. This can limit stormwater treated in this fashion to non-potable uses (Hatt *et al.*, 2006; Philp *et al.*, 2008).
- **Advanced treatment and disinfection** – advanced treatment processes are typically used when the desired end-use for the harvested stormwater requires a uniform and reliable quality (e.g. potable and certain non-potable uses) (Hatt *et al.*, 2006; Philp *et al.*, 2008). Typical advanced treatment processes include: coarse and fine screening, dissolved air flotation (DAF), microfiltration, reverse osmosis, aeration, biological treatment and electrolytic flocculation (Mitchell *et al.*, 2006; Philp *et al.*, 2008). Mitchell *et al.* (2006) noted that these treatments are characteristic of those employed in potable water and wastewater treatment plants. Advanced treatment processes are not commonly incorporated within a stormwater-harvesting system as they are typically too expensive and complex to operate at a small scale (Hatt *et al.*, 2006; Mitchell *et al.*, 2006). Both Philp *et al.* (2008) and Mitchell *et al.* (2007) however, state disinfection is required if the end-use of the water involves deliberate or potentially accidental human contact. Common disinfection techniques include chlorination, ultraviolet (UV) radiation, oxidation, and membrane filtration. The appropriate advanced treatment process and/or disinfection method is case specific and should be chosen based on the site characteristics and the

system's requirements (Philp *et al.*, 2008). Unfortunately, at present, the affordability of reliable treatment techniques has hindered the widespread implementation of the stormwater harvesting (Philp *et al.*, 2008; Fisher-Jeffes, 2015).

Stormwater harvesting is currently being practiced, although it is not effectively regulated (Hatt *et al.*, 2006). This leaves stormwater harvesting in a vulnerable position as the public could begin to doubt the adoption of stormwater re-use if a case of public health or environmental failure gained notoriety (Fisher-Jeffes, 2015). Community acceptance is critical to the success of a stormwater harvesting system; however, community acceptance varies depending on the intended end-use of harvested stormwater (Mitchell *et al.*, 2006; Wu *et al.*, 2012). Mitchell *et al.* (2006) state that the public is generally supportive of stormwater re-use for non-potable purposes, but they are apprehensive of human contact with re-used stormwater due to perceived health risks. Community acceptance of stormwater harvesting may be improved by informing the public in connection with the system about the quality of the water it is re-using (Fletcher *et al.*, 2007; Wu *et al.*, 2012). Treating harvested stormwater is necessary for public health and safety considerations, but by publicising the standard to which harvested stormwater was treated can also offer public assurance.

2.3.3 Storage

The primary purpose of a stormwater harvesting system's storage facility is to deliver a reliable water supply that can provide for intermittent end-use demands (Mitchell *et al.*, 2006). In theory, a stormwater harvesting system has the potential to supply a considerable proportion of an urban area's water demand if unlimited space for storage was available. However, in reality, there is limited land available in urban areas that could be used to develop storage facilities; thus, this limits the total water demand a stormwater harvesting system is capable of supplying. Additionally, the magnitude and temporal patterns of both the water demands and the catchment runoff influence the storage size requirements. The design of a storage facility is ultimately a compromise between maximising volumetric reliability and minimising the storage capacity and its associated costs (Mitchell *et al.*, 2007; Fisher-Jeffes, 2015).

A storage facility can be placed in two general positions in to respect to the watercourse from which it harvests. Storage facilities can either be positioned directly in-line of a watercourse's path – on-line storage – or aside from the watercourse's path – off-line storage. Off-line storage facilities capture runoff by way of pumping or diverting stormwater runoff from an existing watercourse (Mitchell *et al.*, 2006; Philp *et al.*, 2008). On-line storage facilities provide better flood and pollution control as well as yield a higher runoff volume than off-line storage facilities; whereas off-line storage units require less maintenance and have lower associated costs than on-line storage facilities (Philp *et al.*, 2008). At the same time, a storage facility can be designed to offer multi-functional benefits such as visual and recreational amenity, stormwater pollution control, flood mitigation, environmental flow protection, and habitat

provision. This opportunistic approach is often incorporated in golf courses and parklands (Mitchell *et al.*, 2006; Fletcher *et al.*, 2007; Brodie, 2008; Philp *et al.*, 2008). The general position and design of a storage facility varies depending on its intended function (Philp *et al.*, 2008); however, all storage systems can be broadly categorised as one of the following:

- **Open storage** – open storage facilities can generally be defined as a waterbody that is exposed to surface evaporation and incident precipitation. Hatt *et al.* (2006) considers open storage units to includes retention ponds, dams, reservoirs, and wetlands. However, Armitage *et al*. (2013) and Woods-Ballard *et al*. (2007) note that stormwater harvesting systems should avoid using natural waterbodies for storage to prevent irreparably damaging the waterbody through contamination (Fisher-Jeffes, 2015). As shown in Figure 2-1, several storage components (dead, active and flood mitigation) exist within the capacity of an open storage unit (Mitchell *et al.*, 2006). The dead storage component represents the permanent volume of water (occurring below a minimum water depth) within the storage facility that is maintained in order to ensure a minimum water quality, allow for sedimentation, preserve aquatic habitat and maintain visual amenity. The active storage component (found above the minimum water depth), is the volume of water used for harvesting under normal conditions. The flood mitigation storage component accounts for the volume of storage used for flood attenuation (Mitchell *et al.*, 2006; Brodie, 2008; Philp *et al.*, 2008). Unfortunately, open storage facilities are susceptible to water losses such as infiltration losses, which can be significant if the storage facility is not situated in soils of low hydraulic conductivity or lined with an impermeable layer (Heal, 2000). Furthermore, open storage facility lose water due to evaporation; however, due to incident rainfall, the net evaporation losses can be relatively small in areas with non-seasonal rainfall (Mitchell *et al.*, 2006). In theory, evaporation losses can be further reduced by increasing the volume to surface area ratio, but this is often difficult to achieve in practice (Philp *et al.*, 2008).

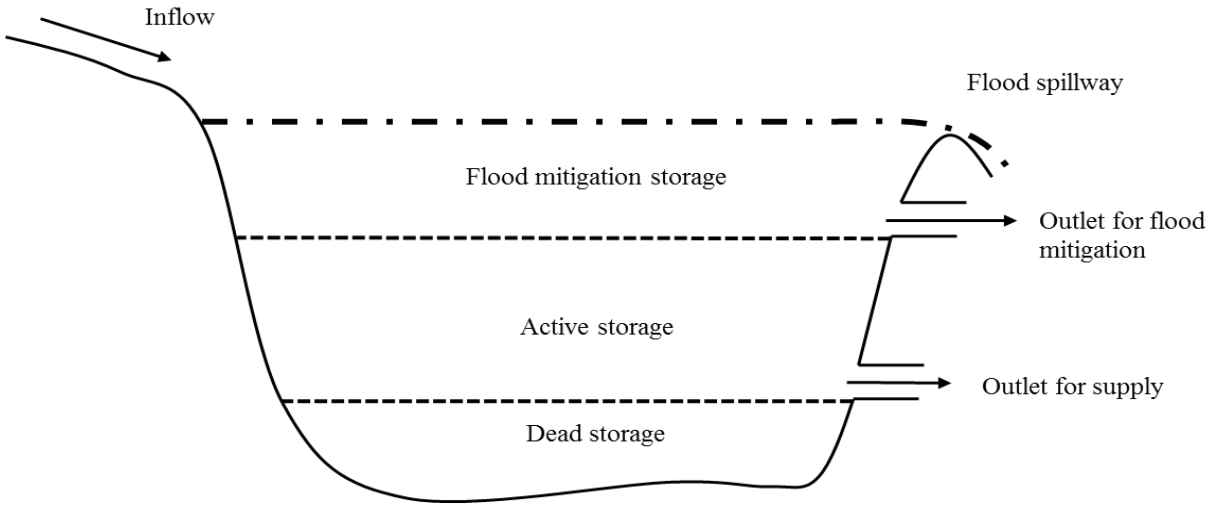

Figure 2-1: The storage components of an open storage unit (Mitchell *et al.*, 2006)

- **Closed storage** – closed storage units are any form of storage in which the water that is stored does not experience any water gains or losses (e.g. water gained from incident precipitation or lost from evaporation/infiltration) (Fisher-Jeffes, 2015). Tanks, as well as underground vaults, are examples of closed storage devices. Hatt *et al.* (2006) noted that in Australia, closed storage units are the most commonly used form of storage in stormwater harvesting systems. This type of storage minimises public safety concerns as well as protecting the stored volume from external contamination (Philp *et al.*, 2008). However, closed storage units are mostly suited to smaller catchments. Hatt *et al.* (2006) identified that closed storage devices are used less frequently as the catchment size increases as their cost becomes prohibitive when storing larger volumes. Additionally, the size alone of the closed storage devices can create issues as underground storage requires excavations deep enough to bury the storage facility whilst above ground storage can create visual amenity issues (Philp *et al.*, 2008).
- **Managed Aquifer Recharge** – Managed Aquifer Recharge (MAR) provides a possible storage option for areas that experience high evaporation rates and have little available space (Philp *et al.*, 2008). It involves intentionally recharging aquifers for future use or for environmental benefits (Dillon *et al.*, 2009; Wu *et al.*, 2012; Fisher-Jeffes, 2015). The treatment provided by MAR (e.g. filtration, extended retention times, etc.) can remove pathogens. Dillon *et al.* (2009) states there are numerous techniques used for recharging aquifers such as, *inter alia*, infiltration ponds, percolations tanks, or recharge weirs. Whilst MAR can be used to store significant volumes of stormwater, it cannot be implemented everywhere since it requires a suitable aquifer (Fisher-Jeffes, 2015).

2.3.4 Distribution

Water from stormwater harvesting systems can be distributed using either open space irrigation systems or non-potable distribution systems (Mitchell *et al.*, 2006, 2007; Philp *et al.*, 2008; Fisher-Jeffes, 2015). Non-potable water distribution systems, otherwise known as a dual reticulation system, introduces a 'third pipe' to the household that supplies non-potable water (Armitage *et al.*, 2014). The most suitable method of distribution for a stormwater harvesting system is dependent on numerous factors, including, *inter alia*, the spatial scale of the distribution area and the intended end-uses (Mitchell *et al.*, 2006, 2007; Philp *et al.*, 2008; Fisher-Jeffes, 2015).

Mitchell *et al.* (2006 and 2007) and Fisher-Jeffes (2015) state that the knowledge used in designing, operating and maintaining potable or recycled wastewater distribution systems is also applicable to stormwater harvesting distribution systems. However, whilst the design integrity of non-potable distribution systems should mimic potable distribution systems, their reliability need not be as stringent unless the harvested water is used for fire-fighting (US EPA, 2004). Fisher-Jeffes (2015) further states that whilst more experience is needed in South Africa in regards to implementing and managing stormwater harvesting distribution systems, there are already

established guidelines used for designing potable water reticulation systems – 'Guidelines for Human Settlement and Planning' (CSIR, 2005) – in South Africa that could appropriately be adapted to designing distribution systems for stormwater harvesting systems.

2.4 A description of stormwater ponds

Urban expansion has led to the replacement of natural landscapes by impermeable surfaces such as roads or paved areas. This has amplified the quantity and peak flow rate of stormwater. Conventional drainage infrastructure has exacerbated the problem by shortening the time of concentration thus further raising flood peaks (Armitage *et al.*, 2013). In order to mitigate these negative impacts without restraining the rate of urban expansion, an alternative approach to urban drainage was developed that manages stormwater in a more holistic manner (Nascimento *et al.*, 1999). This approach is referred to as Sustainable Drainage Systems (SuDS) in South Africa, but is also known as Low Impact Development (LID), Water Sensitive Urban Design (WSUD), or Low Impact Urban Design and Development (LIUDD) in other countries throughout the world. The aim of SuDS is to restore the natural hydrological cycle by introducing local and regional controls that treat runoff at the source minimising the impact on the stormwater network (Woods-Ballard *et al.*, 2007). Furthermore, SuDS aim to ensure that the promotion of ecological preservation, sustainability and the enhancement of quality of life within the urban drainage system remains a priority (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013; Shamsudin *et al.*, 2014).

Stormwater ponds form a key component of SuDS. When stormwater ponds were first introduced, their primary objective was to attenuate peak flows and high volumes of surface runoff generated from urban areas by providing temporary storage (Nascimento *et al.*, 1999; Shamsudin *et al.*, 2014). As a result, they are considered to be the 'last line of defence' in a stormwater management system – they are typically used to manage the stormwater runoff from various developments (Vice, 2011; Armitage *et al.*, 2013). Soon after their inception, it became apparent that stormwater ponds also improved water quality. Furthermore, it was realised that stormwater ponds could be better integrated into urban environments through adapting them to include cultural and recreational features (Nascimento *et al.*, 1999). The introduction of stormwater ponds as multifunctional devices therefore, could lead to a more holistic and sustainable management of stormwater within an urban context ensuring they fit seamlessly into the SuDS ideology (Rezende *et al.*, 2011). It is this multifunctional ability of stormwater ponds that allow them to be readily adapted for stormwater harvesting purposes.

There are three distinct types of stormwater ponds: detention ponds, retention ponds and constructed wetlands; which will be discussed in subsequent sections. All three types of ponds have a variety of different characteristics that enable them to manage runoff in different environments. While the primary objective of a stormwater pond remains the storage of stormwater runoff to reduce flood peaks, certain types of ponds are more suited to catering to the other objectives and are thus implemented where appropriate.

2.4.1 Detention ponds

Detention ponds are dry basins that temporarily store stormwater. They attenuate peak stormwater flows by capturing and detaining runoff for a period of up to 72 hours (varies based on the design), whilst simultaneously releasing the captured runoff at a reduced and controlled flow rate. Detention ponds typically perform this role for storm events of a recurrence interval of ten-years or greater (Clar *et al.*, 2004). Consequently, detention ponds are usually empty except for during and immediately after a storm event. Unfortunately, this limits their ability to improve water quality.

Figure 2-2: An example of a detention pond (Google Earth, 2013)

Park *et al.* (2014) states that detention ponds are the most commonly used SuDS device owing to their ease of application. This, coupled with their periodic flood attenuation responsibilities, allow detention ponds to serve multiple purposes. Lee & Li (2009) classified detention ponds as either Uni-Use Detention Basins (UDB) or Multi-Purpose Detention Basins (MDB). As these names suggest, a UDB has a singular purpose which is to attenuate flood peaks whilst a MDB integrates its flood attenuation ability with other functions. Additional functions that a MDB may incorporate include *inter alia*: sports facilities, parking areas and public parks (Nascimento *et al.*,

1999; Armitage *et al.*, 2013). In addition to providing desirable public facilities, if MDBs are properly maintained, they have proven to be more cost-effective than UDBs whilst also enhancing the value and public perception of the area in which they are situated (Lee & Li, 2009; Park *et al.*, 2014).

Figure 2-3: A multi-purpose detention pond that also serves a sports field (Ubriaco, 2015)

The following aspects are cited as advantages of detention ponds (Armitage *et al.*, 2013):

- They are able to handle a wide range of storm events providing temporary storage and attenuating downstream flow rates.
- They are simple and relatively cheap to design and construct.
- They have the potential for multiple uses.
- They are easy to maintain.

The following aspects are cited as disadvantages of detention ponds (Woods-Ballard *et al.*, 2007):

- They offer minimal water treatment benefits.
- They offer minimal reduction of flood volumes.

- Settled sediment may become re-suspended during storm events.
- Their requirement for large areas compromises their suitability to high-density urban areas (Armitage *et al.*, 2013).

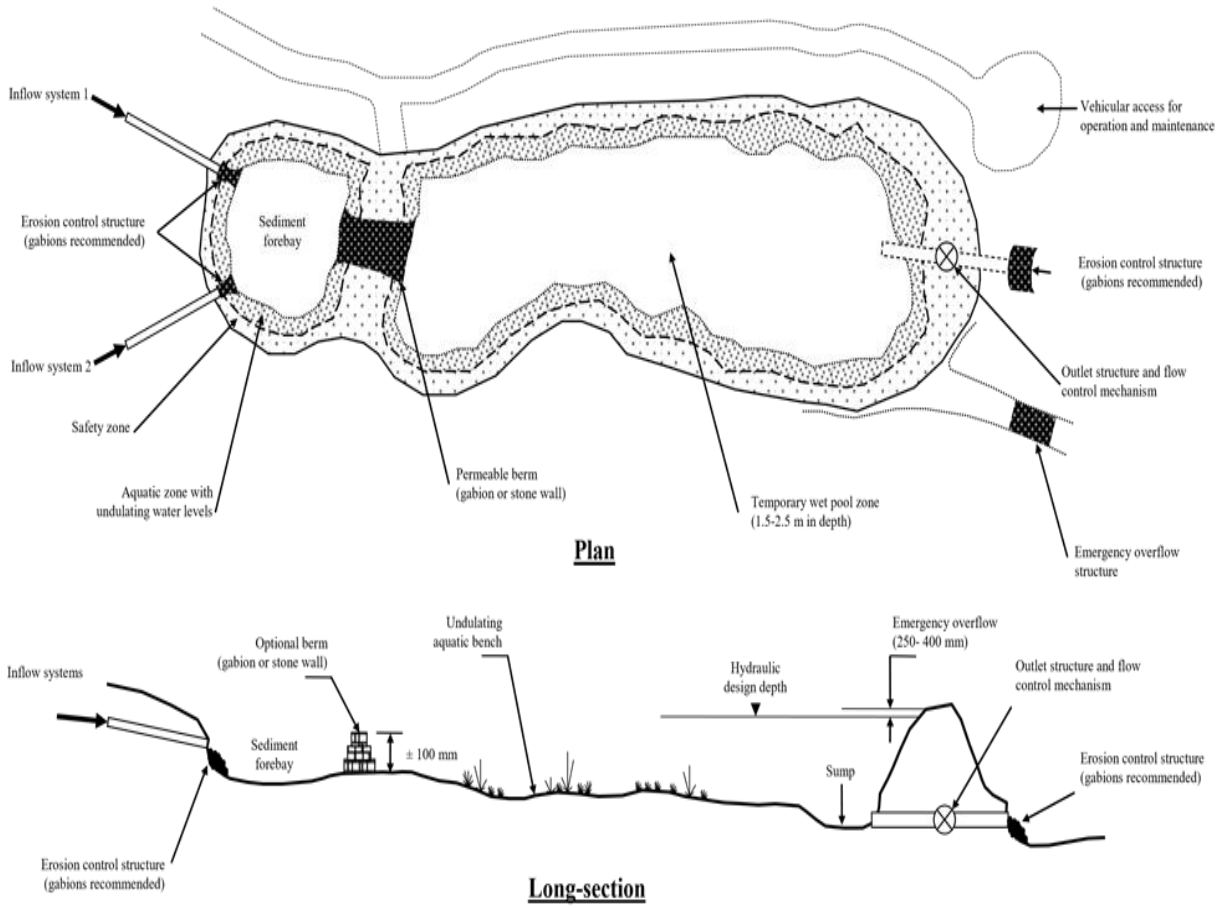

Figure 2-4: General design of a detention pond (Armitage *et al.*, 2013)

2.4.2 Retention ponds

A retention pond is a stormwater attenuation device that includes a permanent volume of water within its total storage capacity. The permanent water volume is intended to provide water treatment benefits whilst the remaining storage capacity is used to manage stormwater flows from a variety of storm events (Woods-Ballard *et al.*, 2007; Hancock *et al.*, 2010). By providing the capacity for the long-term storage of stormwater, retention ponds can remove pollutants (not including pathogens) through a combination of sedimentation which occurs in the permanent pool as well as filtration, infiltration and biological uptake by vegetation along the ponds embankment (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). Additionally, the stormwater

retained by retention ponds can be harvested (US EPA, 2004; Mitchell *et al.*, 2006; Armitage *et al.*, 2013).

Retention ponds can be constructed from natural depressions, newly excavated depressions or constructed embankments. Whilst they are typically implemented to attenuate and treat stormwater, retention ponds can enhance an area's aesthetic and ecological value on the condition that they receive proper maintenance. Due to the multitude of benefits that they can provide, retention ponds have become popular amongst new developments (Heal, 2000). However, if a retention pond does not receive proper maintenance, the issues that can occur include, *inter alia*: mosquito breeding, algal blooms, water stagnation and a reduction in capacity due to sedimentation (Åstebøl *et al.*, 2004; Clar *et al.*, 2004; Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013).

Figure 2-5: An example of a retention pond (Alliance, 2010)

The following aspects are cited as advantages of retention ponds (Woods-Ballard *et al.*, 2007):

- They can cater for a variety of storm events and attenuate peak flows.
- They are a cost effective SuDS strategy (Armitage *et al.*, 2013).
- They provide water treatment benefits and can remove a variety of pollutants.

- They are well accepted within the community when managed effectively as they provide aesthetic and ecological value to the local area.

The following aspects are cited as disadvantages of retention ponds (Woods-Ballard *et al.*, 2007):

- They cannot remove fine particles or pathogens from the stormwater runoff.
- If the permanent water volume is not aerated, anaerobic conditions may occur which can cause water stagnation and mosquito breeding.
- They can pose health and safety risks such as drowning.
- Retaining water for a long period can increase its temperature which can disturb downstream natural habitats when the water is released (Lieb & Carline, 2000; Missaghi, 2010; Hamer *et al.*, 2012).

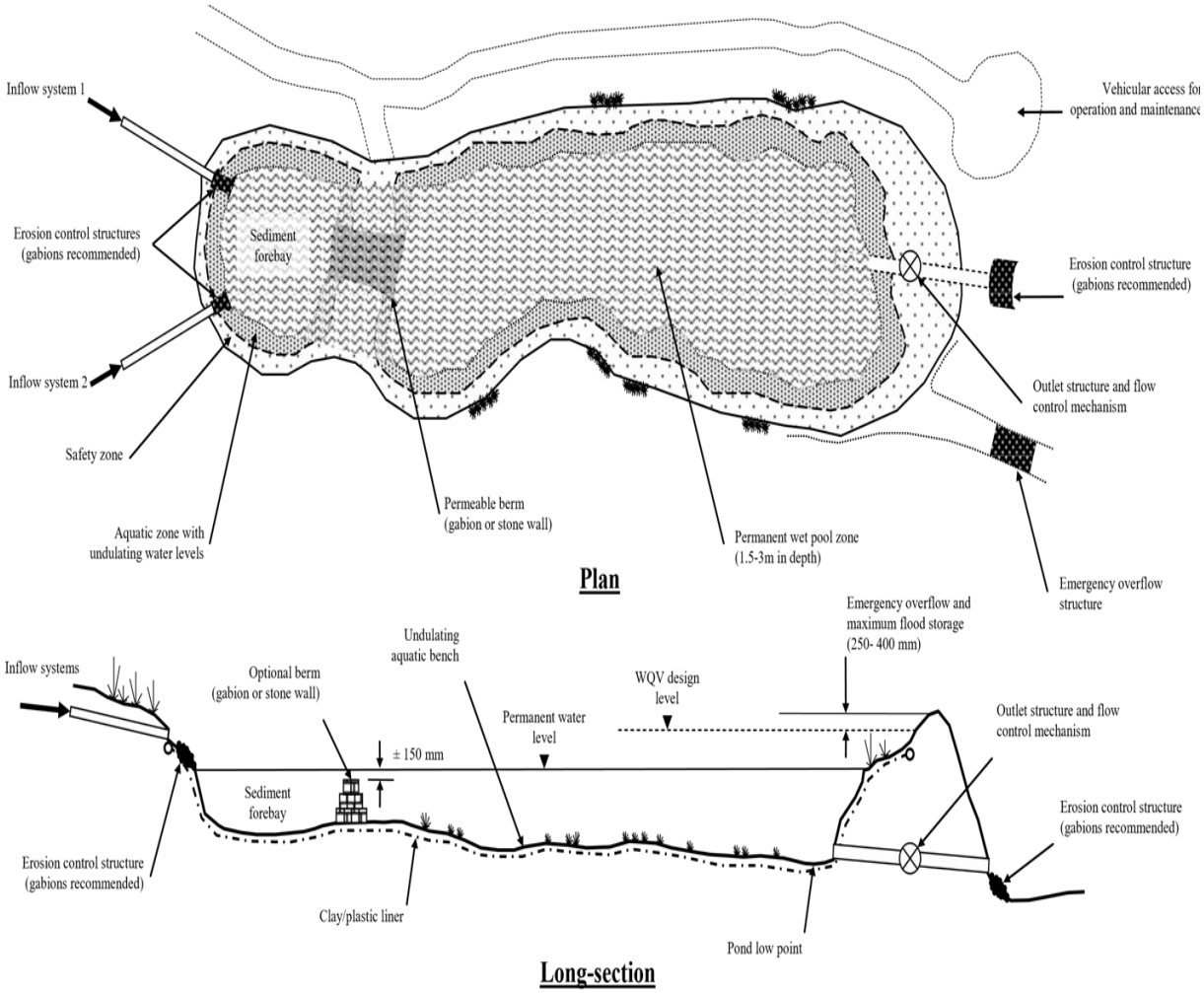

Figure 2-6: General design of a retention pond (Armitage *et al.*, 2013)

2.4.3 Constructed wetlands

Wetlands can be described as marsh systems that contain shallow water and are covered by aquatic vegetation (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013). Whilst wetlands do provide some flood attenuation benefits, they are most suited to providing water treatment and ecological services (Wong *et al.*, 1999; Deletic *et al.*, 2014). They are capable of removing particulate material, dissolved nutrients, heavy metals and a percentage of total pathogens through various processes which include: sedimentation, fine particle filtration, and biological nutrient and pathogen removal (Armitage *et al.*, 2013). They are also capable of supporting a diverse variety of habitats. Due to these desirable qualities, constructed wetlands have been developed to provide these services in an urban environment. Unfortunately, wetlands require a substantial land take to be effective, which reduces their suitability for high-density development areas. Furthermore, they also require a perennial inflow to sustain their diverse plant-life (Woods-Ballard *et al.*, 2007; Armitage *et al.*, 2013).

Figure 2-7: A constructed wetland, Century City (Johannes, 2011)

The following aspects are cited as advantages of constructed wetlands (Woods-Ballard *et al.*, 2007):

- They can improve the quality of stormwater through the removal of contaminants such as particulate material, dissolved nutrients, heavy metals and pathogens.

- They preserve biodiversity.
- They can provide recreational and educational services.

The following aspects are cited as disadvantages of constructed wetlands (Armitage *et al.*, 2013):

- The flood attenuation ability of constructed wetlands is limited.
- The inflow rate of a wetland must be controlled to prevent the destruction of the wetland's vegetation.
- They are not suited to all areas as they require a large amount of land and can only be built in flat areas.
- During dry seasons, wetlands may require additional water.

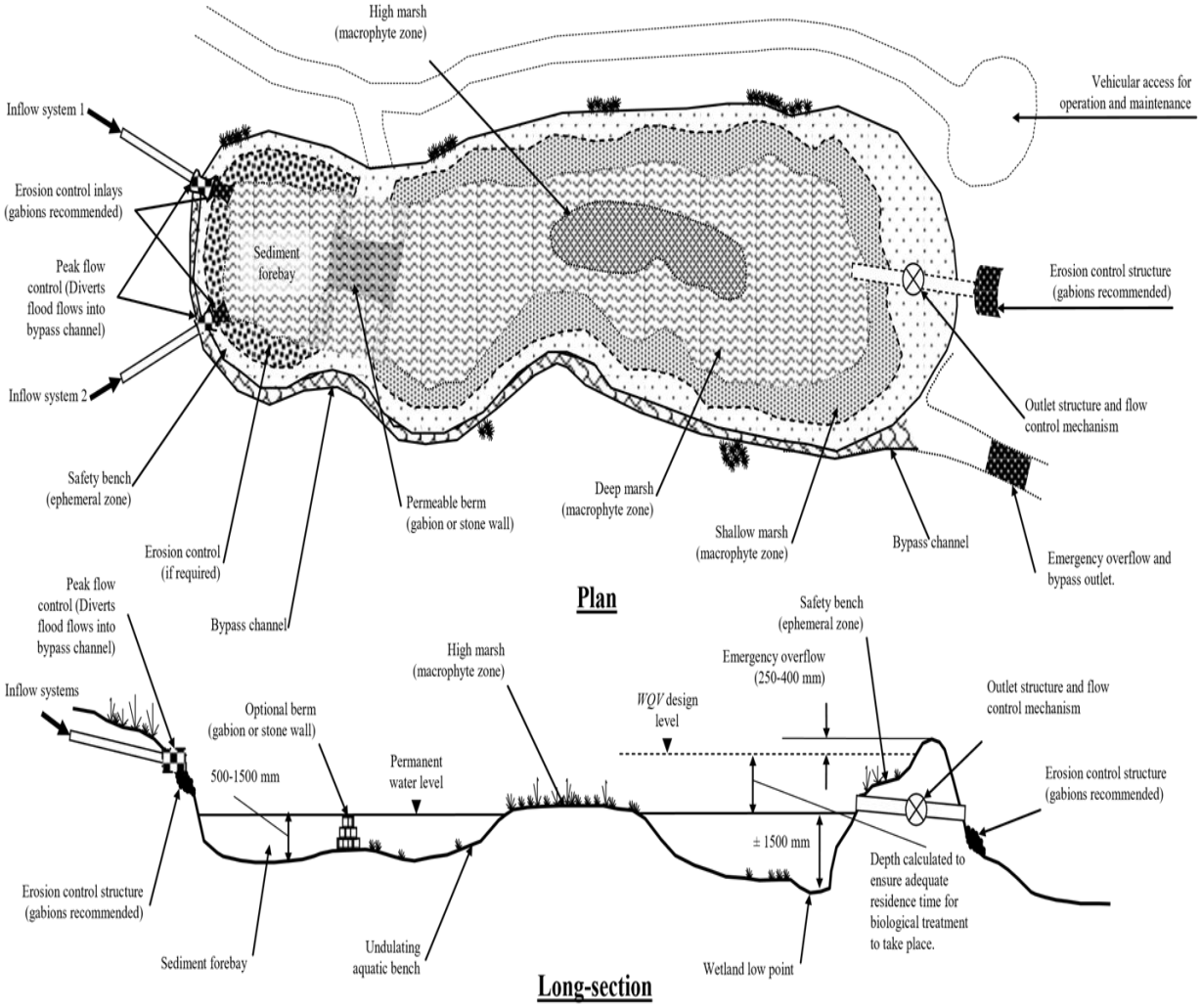

Figure 2-8: General design of a constructed wetland (Armitage *et al.*, 2013)

2.4.4 Summary of the abilities of stormwater ponds

Table 2-1 summarises the attributes of the three different types of stormwater ponds.

Table 2-1: Stormwater pond attributes (Woods-Ballard *et al*., 2007; Armitage *et al*., 2013)

		Detention Ponds	Retention Ponds	Constructed Wetlands
Water Quantity	Water storage			
	Attenuation of peak flow			
	Reduction in flood volume			
Water Quality	Removal of coarse sediment			
	Removal of fine sediment			
	Removal of pathogens			
Provision of Amenity	Recreational facilities			
	Educational facilities			
	Aesthetical appeal			
	Ability to be multi-functional			
	Applicability to high density urban areas			
Preservation of Biodiversity	Creation of habitat			
	Promotion of indigenous vegetation			

Rating	**Key**
Good	
Adequate	
Poor	

2.5 Modelling a stormwater harvesting system

As interest in stormwater harvesting has grown, a number of computational based studies have occurred to investigate if harvested stormwater would create a viable water resource. Investigations completed for this purpose typically emphasise the importance of realistically approximating the volume of harvestable stormwater and storage performance (Mitchell *et al.*, 2008). The volume of harvestable stormwater and the performance of storage facilities is typically approximated using a catchment stormwater model; this process will be discussed in the following section (Section 2.5.1). Also to be discussed in the following section are: the indicators used to analyse the performance of a stormwater harvesting system and Real Time Control (RTC) – a method that can be used to increase the storage efficiency of a harvesting system.

2.5.1 Stormwater modelling

Computer aided software that simplifies the creation of computer based stormwater models was first developed in the 1970s (Zoppou, 2001). Since then, numerically modelling stormwater behaviour in urban drainage systems has become widespread – they are frequently used to analyse, manage, plan and design stormwater management strategies (Zoppou, 2001; Siriwardene & Perera, 2006; James *et al.*, 2010; Armitage *et al.*, 2014; Song *et al.*, 2015). Elliot & Trowsdale (2007) state that this is because computer aided software reduces the complex and highly variable natural processes of large scale systems that computer based stormwater models simulate into a manageable and understandable form.

All computer based stormwater models can generally be separated into two basic components: a hydrological component and a hydraulic component. The hydrological component deals with surface runoff generation, infiltration, evaporation, and pollutant build-up and washoff, whilst the hydraulic component deals with the conveyance of flows and pollutants (Zoppou, 2001; Siriwardene & Perera, 2006). However, the degree of complexity to which a stormwater model may simulate these processes can vary. For example, a complex stormwater model might continuously simulate a catchment's overall water balance for a multiple-season duration whilst a simpler model may only predict the runoff generated from a single storm event (Zoppou, 2001; MPCA, 2015). Naturally, the complexity of the model has implications on the computational resources its requires, its limitations and the reliability of the results it produces. Zoppou (2001) states that: "*Results from simpler models can also be extracted from the results of more sophisticated models, however the converse is not generally true".* Importantly though, James (2005) emphasises that the complexity of a model can only increase its reliability up to a point, and that ultimately, the reliability of a stormwater model is dependent on the data that is available.

The reliability of a stormwater model can be regarded as a question of the level of uncertainty in the model. Uncertainty is intrinsic with urban drainage models and may originate from sources such as model parameters, input data, calibration data or the model structure

(Deletic *et al.*, 2012). Whilst uncertainty is unavoidable in stormwater models, it is possible to reduce it. Efforts to reduce uncertainty largely focus on the calibration methods of model parameters (*ibid.*). Song *et al.* (2015) explains that stormwater models often include a high degree of uncertainty as it is difficult to obtain accurate values for numerous parameters. Fortunately, parameters that are difficult to obtain can be estimated through calibrating the stormwater model against reliable calibration data (Siriwardene & Perera, 2006).

The range of application of computer based stormwater models has increased substantially since their inception (Song *et al.*, 2015). In addition to simulating the water quantity and quality processes of a catchment, some stormwater modelling software offer tools which can model stormwater management techniques such as SuDS or Real Time Control (RTC) systems (described in Section 2.5.3). Thus, modellers should carefully consider which modelling software is most suited to delivering the intended objectives of their research. Fortunately, Zoppou (2001), Elliot & Trowsdale (2007), Armitage *et al.* (2014) and MPCA (2015) have performed comprehensive reviews of the most widely used and recognised stormwater modelling software to aid this decision (Table 2-2).

Table 2-2: Potential software options for this study
(Elliott & Trowsdale, 2007; Armitage *et al.*, 2013)

	Public education	**Research**	**Developing sizing rules for devices**	**Planning of land use in catchment/cities**	**Preliminary design of regional controls**	**Preliminary design of a subdivision or site**	**Detailed design of regional drainage system**	**Detailed design of subdivision or site**	**Site layout and materials selections**
MOUSE		■	■	■	■	■	▒	▒	▒
MUSIC		■	■	■	■	■			
P8			■	■	■	▒			
SLAMM			■	■	■	▒			
StormTac				■	■				
SWMM		■	■	■	■	■	▒	▒	▒
PCSWMM		■	■	■	■	■	▒	▒	▒
UVQ		■	▒	▒	▒	■			
WinDes (Quantity Only)			▒		■	■	■	■	▒
Key	■	Software is suitable for use		▒	Software is marginally suited for use			Software is not suitable for use	

It should be noted that the stormwater modelling software listed in Table 2-2 predominantly focus on the quantitative components of the stormwater system and fail to address wider aspects – particularly social and institutional obstacles (Armitage *et al.*, 2014). This is widely acknowledged as a current limitation of computer modelling software. Social aspects are often overlooked in modelling tools due to the difficulty in quantifying their effect (Hatt *et al.*, 2006; Bastien *et al.*, 2012). Nonetheless, certain techniques such as hedonistic pricing, scorecard assessments or public engagement have been developed to measure the effect of social aspects (Bastien *et al.*, 2012; Moore & Hunt, 2012; Rooney *et al.*, 2015). Unfortunately, these techniques are most suited to post-development scenarios which limit their effectiveness for many modelling situations.

2.5.2 Harvesting performance indicators

Performance indicators are a useful tool that enables the evaluation of a system under a range of different conditions. Over the past two decades, a number of different performance indicators have been developed to assess the performance of water storage infrastructure. McMahon *et al.* (2006) provided a comparative analysis of these performance indicators and detailed the appropriateness of their application. Mitchell *et al.* (2008) note that the performance indicator should be selected based on the aim of the study and the aspect of the system that requires analysis. Based on McMahon *et al.* (2006) findings and the study performed by Mitchell *et al.* (2008), the following performance indicators are considered to be the most useful for evaluating the effectiveness of a stormwater harvesting system:

- **Volumetric reliability (R_V)** is defined as the volume of water that is supplied divided by the total water demand during the entire simulation period (Mitchell *et al.*, 2008). Also known as water savings efficiency, this performance indicator measures the water savings provided by the system (Fewkes & Butler, 2000; Palla *et al.*, 2011).

$$R_V = \frac{\sum_{t=1}^{T} Y_t}{\sum_{t=1}^{T} D_t} \qquad 2\text{-}1$$

  Where: R_V = volumetric reliability; Y = yield (m^3); D = water demand (m^3); T = total number of time-steps in the simulation period; t = time-step.

- **Time-based reliability (R_T)** reflects the proportion of simulation time-step intervals during the entire simulation period in which the target water demand is fully met (McMahon *et al.*, 2006).

$$R_T = \frac{N}{T} \quad 2\text{-}2$$

Where: R_T = time-based reliability; N = number of time-steps in which the target water demand was fully met; T = total number of time-steps in the simulation period.

- **Resilience (ϕ)** is a measurement of how quickly a storage unit will recover or 'bounce-back' from a period in which it was in failure (Hashimoto *et al.*, 1982; McMahon *et al.*, 2006). According to Hashimoto *et al.* (1982), resilience is equivalent to the average probability that a storage unit will recover from failure in its next simulation time-step. McMahon *et al.* (2006) also notes that resilience is equal to the inverse of the average failure duration. Since it is a temporal measurement, the result is sensitive to the chosen simulation time-step and care should be taken over its interpretation (McMahon *et al.*, 2006; Mitchell *et al.*, 2008)

$$\phi = \frac{f_s}{f_d} \quad f_d \neq 0 \quad 2\text{-}3$$

Where: ϕ = resilience; f_s = the number of continuous periods in which the target water demand is not fully met; f_d = the total duration in which the target water demand is not fully met.

A hypothetical example of the performance of a storage unit over a twelve-day period using a daily time-step was developed to demonstrate how resilience is calculated. The system is in failure for time-steps in which the supplied demand does not equal the target demand.

Table 2-3: Yield of hypothetical storage unit

Time-step (day)	1	2	3	4	5	6	7	8	9	10	11	12
Target demand (Mℓ)	5	3	3	3	4	6	11	15	12	11	10	9
Supplied demand (Mℓ)	5	3	0	0	0	6	0	15	12	0	0	9

Table 2-3 shows that the hypothetical system entered into a period of continuous failure (f_s) on three separate occasions between time-intervals 3-5; 7; and 10-12. The hypothetical system failed to meet the target demand for a total duration (f_d) of six days. Therefore,

$$\phi = \frac{3}{6} = 0.5 \quad 2\text{-}4$$

As McMahon *et al.* (2006) stated, resilience is also equal to the inverse of the average failure duration. This is demonstrated in Equation 2-6. The hypothetical system failed on three separate occasions for three, one and two days, respectively.

$$\text{Average failure duration} = \frac{3 + 1 + 2}{3} = 2 \qquad 2\text{-}5$$

$$\phi = \frac{1}{\text{Average failure duration}} = \frac{1}{2} = 0.5 \qquad 2\text{-}6$$

- **Overflow ratio (O_T)** represents the ratio of the volume of water spilled by the storage unit to the volume of water entering the storage unit during the simulation period (Palla *et al.*, 2011).

$$O_T = \frac{\sum_{t=1}^{T} S_t}{\sum_{t=1}^{T} I_t} \qquad 2\text{-}7$$

Where: O_T = overflow ratio; S_t = volume of water spilled by storage unit (m^3); I_t = volume of water entering the storage unit (m^3); T = total number of time-steps in the simulation period; t = time-step.

Volumetric reliability is regarded as having the most practical applications of the performance indicators listed above and is the performance indicator that is most commonly applied in industry when analysing stormwater-harvesting systems (Mitchell *et al.*, 2008; Palla *et al.*, 2011). As time-based reliability and resilience are both temporal performance indicators, their results can vary significantly based on the computational time-step that was used. Additionally, the accuracy of time-based reliability and resilience estimates are dependent on the length of data record (McMahon *et al.*, 2006). Mitchell *et al.* (2008) consider a ten-year climate time series to be the minimum time series length that should be used when analysing performance indicators. As the length of the time series increases, temporal performance indicators will begin to converge towards steady state (McMahon *et al.*, 2006).

2.5.3 Real Time Control

As stated in Section 2.4, the expansion of urban areas has generally led to increased stormwater volumes and peak flow rates because natural surfaces have been replaced by impervious surfaces. As a result, existing stormwater conveyance networks are frequently incapable of handling the increased loading during storm events. However, enlarging the capacity of these networks to

handle increased loading is considered to be an inefficient and unfeasible approach, as already, during inter-events large portions of these networks remain underutilised (Vallabhaneni & Speer, 2011). Whilst SuDS are proven to limit the negative impacts of urban expansion on stormwater systems by managing stormwater in a 'natural' manner, another method to manage increased stormwater volumes that is gaining interest is using Real Time Control (RTC) to optimise the storage available in the existing stormwater conveyance network (Gaborit *et al.*, 2012; Beeneken *et al.*, 2013; García *et al.*, 2015).

US EPA (2006) defines RTC as: *'a system that dynamically adjusts the operation of facilities in response to online measurements in the field to maintain and meet the operational objectives'*. Vallabhaneni & Speer (2011) further state, RTC incorporated into the stormwater conveyance network creates dynamic control allowing for the manipulation of automatic flow regulators to optimise the hydraulic capacity and increase the retention time of the network. In contrast, conventional and alternative stormwater management approaches are limited to static control of the stormwater conveyance network due to their use of pre-designed fixed outlets (Gaborit *et al.*, 2012; Beeneken *et al.*, 2013; Muschalla *et al.*, 2014; García *et al.*, 2015). Since these outlets are designed based on peak flows, this introduces large redundancy into the network. RTC offers the opportunity to reduce these redundancies by optimizing existing systems, and so, delaying the need to build new enlarged systems (Colas *et al.*, 2004). In addition to improving the safety, reliability and flexibility of stormwater conveyance networks (Colas *et al.*, 2004; Beeneken *et al.*, 2013), US EPA (2006) state the following operational advantages of RTC systems:

- Can capture pollutants associated with 'first-flush-effect'.
- Capable of managing the flow rate and path during anticipated (e.g. major construction) and unanticipated (e.g. network failures) events.
- Capable of managing the sewer discharge flow rate.

The type and scope of RTC systems will vary depending on the situational context. However, Colas *et al* (2004), US EPA (2006), Vallabhaneni & Speer (2011), Beeneken *et al.* (2013) and García *et al.* (2015) all concur that any RTC system can be placed into one of three general classifications depending on its complexity. These classifications, described in order of ascending complexity, are as follows:

- **Local control –** Local control is the simplest method of RTC. A RTC system is defined to be under local control if adjustments made to the system are dependent solely on measurements taken at the same location at which the adjustments are made. These adjustments can be made through either manual adjustments or using automatic actuators (Colas *et al.*, 2004; US EPA, 2006). Further, the decision support system behind these adjustments in locally controlled RTC systems is typically made using conventional Rule-Based Controls (RBC). RBC strategies are generally straightforward and incorporate 'if-

then' rules (i.e. if this happens, then do this). The control rules that they incorporate are developed before the implementation of the RTC system and are typically a function of measurement (García *et al.*, 2015). Local control RTC systems are often favoured as they are simple to operate and understand, yet their adjustments are limited to on-site conditions, essentially disregarding the conditions throughout the entire system. This limitation can result in conservative RBCs (US EPA, 2004; García *et al.*, 2015).

- **Regional control –** Regional control is similar to local control except that adjustments to the system are made based on measurements taken from a location – or several locations – that are remote to the location at which the adjustments are made (Colas *et al.*, 2004; US EPA, 2004). Thus, regional control RTC systems require communication components and a Supervisory Control and Data Acquisition (SCADA) central server system (Colas *et al.*, 2004; US EPA, 2006; García *et al.*, 2015). Further, unlike local control, regional control RTC systems can operate under either pre-defined RBCs or under optimisation-based algorithms. Optimisation-based algorithms seek to manipulate the system in real-time so that it reaches a desired state (García *et al.*, 2015). Either of these methods can be implemented automatically or under the supervision of a system operator. Like local control, regional control RTC systems' optimisation is limited as it does not consider the conditions throughout the entire system, but instead only considers the conditions in the system where control logic is based (US EPA, 2006).

- **Global control –** Being the most complex method of RTC, global control RTC systems are incorporated to provide optimal operational efficiency (Colas *et al.*, 2004). For global control RTC systems, data is centralised, so adjustments to the network can be made at any point at which there is an actuator based on the data provided from any measurement device that is part of the system. These adjustments are made using decision support systems that either incorporate RBC, optimisation-based algorithms, predicative forecasting or a combination of the aforementioned decision support systems (Colas *et al.*, 2004; US EPA, 2006). To clarify, predicative forecasting is a method that estimates future flows in the network using forecasted rainfall and then adjusts the system to react accordingly (*ibid.)*. Although the most complex, global control RTC system offers the greatest functionality, but it also demands the most understanding. It requires rigorous network analysis and planning before it is implemented – as well as supervisory control by an operator who has a good understanding of system dynamics (US EPA, 2004).

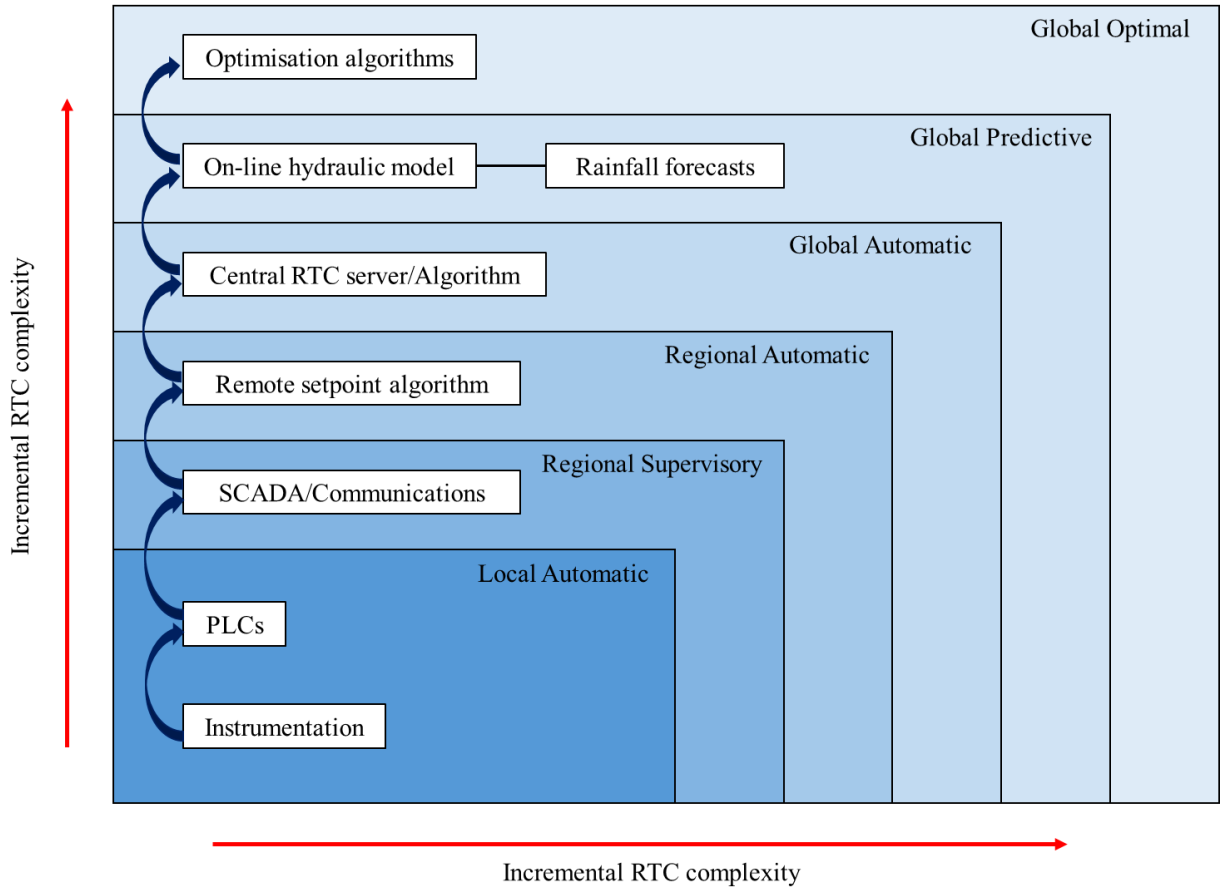

Figure 2-9: The various levels of complexity of Real Time Control systems
(Vallabhaneni & Speer, 2011)

When developing a RTC system, utmost consideration must be given to what system complexity is appropriate. US EPA (2006) stress that developers should only implement a RTC system with a complexity level that they are capable of understanding and operating. RTC systems are not constrained by current technology and the success of RTC systems is more often hindered due to operational or organisational procedures rather than technological issues. Furthermore, RTC is only beneficial in stormwater networks in which there is unused storage capacity during inter-event periods and so it may not always be cost-effective for all situations (Colas *et al.*, 2004; US EPA, 2006).

RTC systems require a number of components in order to function. The number of components required increases with the complexity of the system. The typical components of a RTC system include: measurement instrumentation, actuators, control devices (typically Programmable Logic Controllers (PLC)), data transfer structures, and control systems (SCADA) (Beeneken *et al.*, 2013).

In recent years, incorporating RTC into stormwater conveyance networks has garnered much interest, although the technology it applies is not new. García (2015) claims that the first

RTC prototype was introduced into an urban drainage system in the late 1960s. Since then, an increasing number of RTC systems have been implemented into stormwater conveyance networks, particularly in Europe and North America (García *et al.*, 2015). Whilst research on RTC systems initially focussed on increasing retention time within the stormwater conveyance network to reduce sewer overflow, more recent research has begun to focus on determining the improvement to water quality that RTC systems provide through increasing retention time in stormwater ponds (Fuchs *et al.*, 2004; Muschalla *et al.*, 2014; Vezzaro *et al.*, 2014).

2.6 Summary of literature review

Like many countries throughout the world, South Africa's water security is in a precarious position due to escalating water demands as well as the deterioration of existing freshwater supplies. Strategies such as water demand management and alternative drainage solutions (e.g. SuDS) have been developed to alleviate water concerns. Whilst these strategies are essential to increasing the resilience of water supplies, Coombes & Barry (2007) state that they only defer the inevitable need to augment conventional water supplies with alternative water resources.

The need for alternative water resources has led to a growing interest into the viability of stormwater harvesting. Stormwater is a relatively abundant water resource in urban areas and can be used to supplement water demands. However, as highlighted in this literature review, due to the limited availability of land to develop in urban areas, storage requirements, *inter alia,* are a common concern for stormwater harvesting system developers. Fortunately, stormwater ponds provide a potential solution to this issue. Stormwater ponds have frequently been implemented in urban areas to, *inter alia,* provide temporary storage to attenuate runoff generated from large storm events. This is a periodic responsibility however, which often leaves stormwater ponds underutilised. For this reason, the storage available in stormwater ponds could potentially be used for harvesting purposes to provide a solution to the storage concerns facing stormwater harvesting system developers as well as to get added benefits from stormwater ponds.

Simulating runoff generated within a catchment using a stormwater model is an effective method to understand the complex and highly variable behaviour of runoff. Furthermore, stormwater models present the opportunity to simulate the effect of various catchment conditions, stormwater network alterations or management strategies such as Real Time Control (RTC) – a method used to optimise the performance of a system by dynamically controlling it according to online field measurements. The development of stormwater models has been simplified by computer aided-software, which consequently has become widespread. For this reason, investigations into the viability of stormwater harvesting frequently make use of stormwater models that were developed using computer aided software.

3. The Diep River subcatchment

The choice of catchment was pertinent to this research. In order to ensure that the chosen catchment was appropriate for this research, it was necessary that the catchment could offer the following:

- The catchment had several stormwater ponds which range in size and location.
- There was data available to construct a catchment stormwater model.
- The properties enclosed by the catchment's watershed had a diversity of land uses as well as residents of various levels of affluence as these factors will have an effect on how water is used on a property.

The catchment that was selected for this research is drained by the Diep River. There are two rivers situated in the City of Cape Town (CoCT) named the 'Diep River'. The one considered in this study is located in the Southern Suburbs of Cape Town. The other 'Diep River' is located in another of the CoCT's major catchments named the Diep River catchment, which is located towards the north of the CoCT metropolitan boundary. In order to avoid confusion, the catchment considered in this study will be referred to as the 'Diep River subcatchment'. Starting at the southern end of Table Mountain, immediately south of Wynberg Hill, the Diep River subcatchment stretches from the affluent suburbs of the Constantia Valley to the much less affluent Retreat, eventually discharging into Little Princess Vlei (Brown *et al.*, 2009). The Diep River subcatchment forms the upper regions of the greater Sand River catchment, otherwise known as the Zandvlei catchment. Figure 3-1 illustrates the catchment watershed; the blue line represents the course of the Diep River and its tributaries through the catchment.

The Diep River subcatchment is approximately 2000 ha. in area. In total, it contains or crosses the borders of 21 suburbs, which have a total combined population of approximately 41 000 people (StatsSA, 2011). This population is unevenly distributed throughout the catchment. The suburbs that are located closest to the foot of Table Mountain (mostly the upper reaches of the catchment) typically have a population density of less than 16 people per hectare whilst suburbs located further from the mountain, mostly towards the eastern boundary of the catchment, have population densities that ranged between 30 to 80 people per hectare (*ibid.*). In some of these areas, where there are residential blocks of flats, the population density reaches a maximum of 180 people per hectare. Levels of wealth are distributed in a similar fashion to population density with the most affluent areas located towards the upper reaches of the catchment and the less affluent areas situated towards the lower reaches of the catchment. Finally, whilst the catchment has a range of different land uses (Figure 3-2), it predominantly contains single residential properties (SRPs).

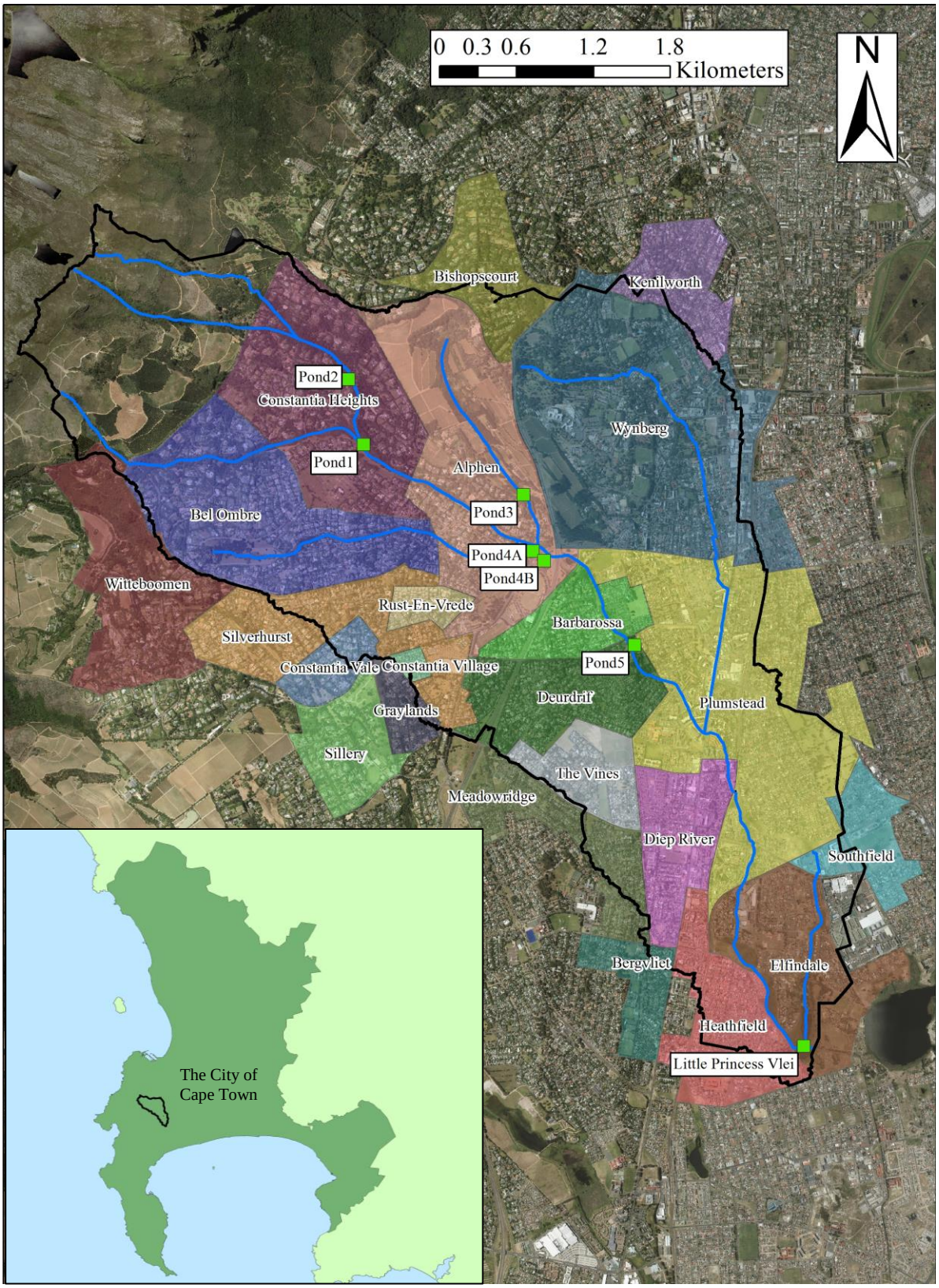

Figure 3-1: The Diep River subcatchment

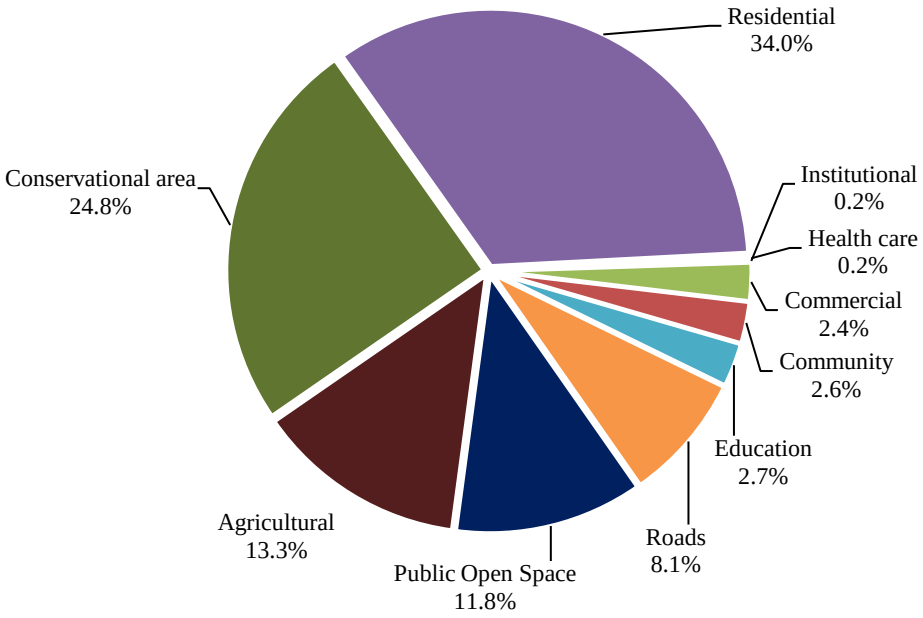

Figure 3-2: The division of land use allocation for the Diep River subcatchment (COCT, 2009a)

Cape Town has a Mediterranean climate; it experiences warm, dry summers and mild, wet winters. However, owing to the City's surrounding mountains, there are a number of microclimates causing some areas of the City to experience much more rainfall than others (WWO, 2012). Being located at the foot of the Table Mountain range, the Diep River subcatchment is one such area, experiencing an average annual rainfall range of between 800 and 1400 mm/yr. Despite this variation, the entire catchment still experiences significantly higher rainfall than the average annual rainfall of the CoCT (530 mm/yr.). The variation in mean annual precipitation (MAP) of the entire metropolitan district of the CoCT is depicted in Figure 3-3.

Between 1970 to 1980, the Diep River subcatchment experienced extensive urban development. Consequently, the pre-existing stormwater network did not have the capacity to cope with the increased volumes of runoff that resulted from the increased impervious area. This led to several serious flooding events. A particularly devastating event occurred in 1977 that incurred significant damages and financial implications (CoCT, 1986). To mitigate these more frequent flooding events, the CoCT responded by implementing the 'Diep River Sand River Flood Control Scheme' in 1986 that introduced six detention ponds along the Diep River and its upstream tributaries (*ibid.)*. These ponds were designed to handle up to 1 in 20-year flood events. Further, CoCT (1986) stated that the ponds had multipurpose capabilities; yet, other than being located in an aesthetical and recreationally appealing area – the Constantia greenbelt – it is not clear what other benefits these ponds provide in addition to flood protection. The discharge point of the Diep River, Little Princess Vlei, whilst being a *vlei* (a shallow, minor lake), is a retention

pond. Along with its neighbour, Princess Vlei, Little Princess Vlei is steeped in cultural and ecological significance (Kotze, 2011; Ernstson & Sörlin, 2013; PVF, 2015). This has led to the formation of civic-led groups that oppose the proposals for development that would lead to the destruction of these *vleis* and instead promote the restoration of the cultural and recreational facilities that the *vleis* offer (*ibid.).* Little Princess Vlei and its upstream detention ponds therefore provide an opportunity to investigate how the functionality of stormwater ponds could be enhanced. The stormwater ponds that were modelled for this research can be seen in Figure 3-4.

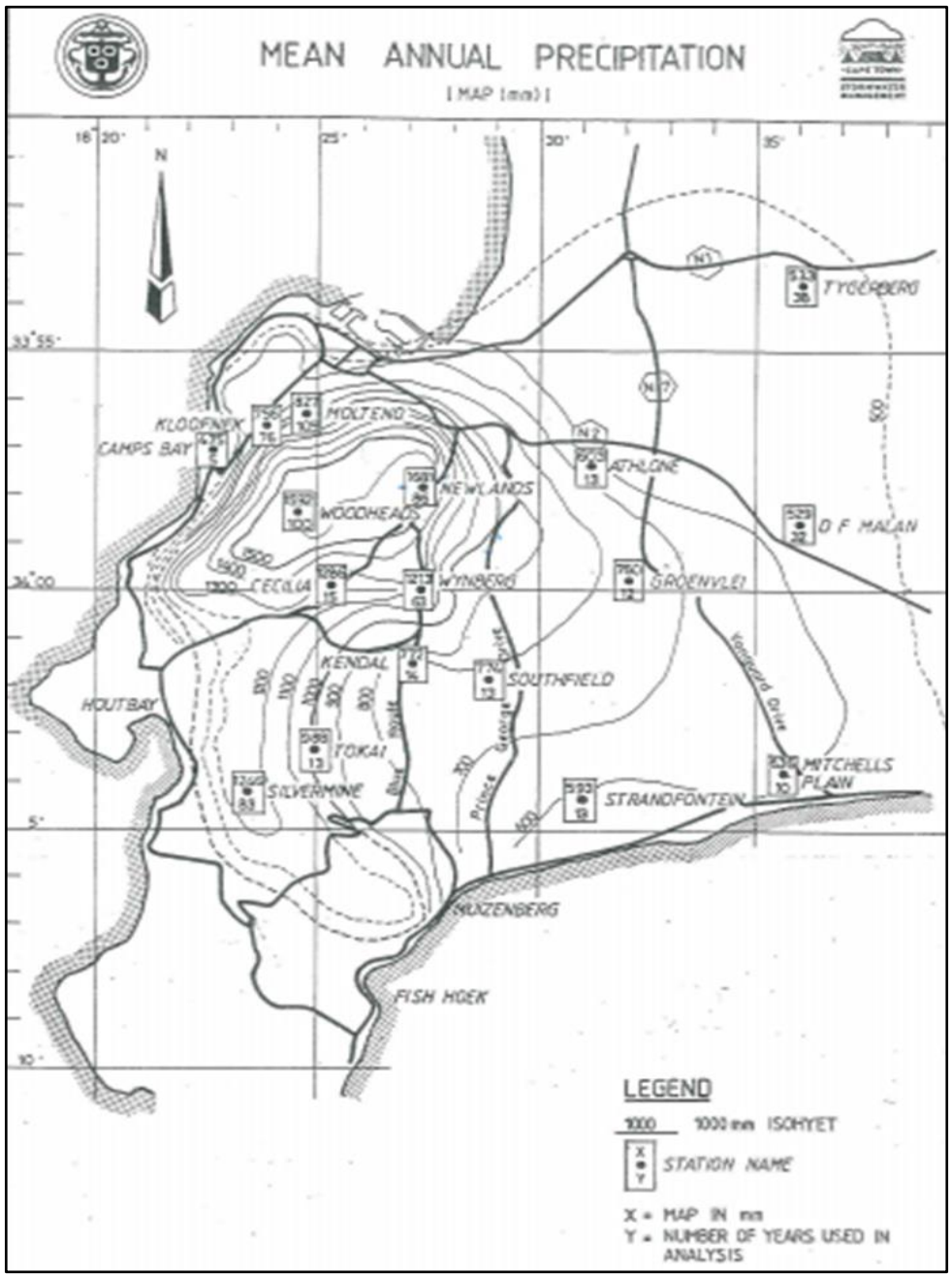

Figure 3-3: Mean annual precipitation map of Cape Town Metropolitan district

Figure 3-4: a) Pond 1; b) Pond 2; c) Pond 3; d) Pond 4B; e) Pond 5; f) Little Princess Vlei

The ponds shown in Figure 3-4 (refer to Figure 3-1 for their locations) are all detention ponds except for Little Princess Vlei which is a retention pond. A watercourse flows directly into Pond 1, 2 and 3 as they are all on-line ponds. All three ponds release water through outlets in their pond walls which are at the same level as their pond floors. Pond 4B and Pond 5 are off-line ponds that both receive stormwater through a side weir. Water is released from these two ponds through multiple non-return outlets which are at the same level as their pond floors.

4. Method

This chapter describes the method that was followed during this research. The chapter focuses on the following: the acquisition of data, the construction of the stormwater model, the development of the stormwater harvesting scenarios and the Life Cycle Cost Analysis (LCCA) that was performed.

4.1 Data acquisition

This research required a substantial amount of data in order to model the various aspects of a stormwater harvesting system. The following data was collected for the modelling process:

- Catchment topography
- Land use and soil infiltration classifications
- Rainfall, evaporation, flow and water quality data records
- Water usage records
- Economic data

4.1.1 Catchment topography

For this study, it was necessary to obtain an accurate representation of the catchment topography; therefore, Light Detection and Ranging (LiDAR) data was obtained from the CoCT. '*LiDAR is an optical remote-sensing technique that uses laser light to densely sample a surface producing highly accurate x, y, z measurements'* (ESRI, 2016b). By using airborne LiDAR data, data that is generated from an airborne platform, it is possible to produce a high resolution digital terrain model (DTM) (LiDAR UK, 2016). The LiDAR data that was acquired from the CoCT was based on the MSL datum and recorded vertical measurements accurate to 0.5 m; its points were horizontally spaced 0.5 m apart.

As LiDAR data captures the details of all surfaces upon which the emitted laser is returned (e.g. ground surfaces, buildings, vegetation, etc.), the data requires post-processing to assign a classification to each LiDAR point depending on the object that reflected the laser pulse. LiDAR points are differentiated from one another based on the time difference between multiple return signals (ESRI, 2016b; LiDAR UK, 2016). Fortunately, the LiDAR data obtained from the CoCT was already post processed. Therefore, only the ground surface points were extracted from the LiDAR data, which were then used to produce a high resolution 0.5 m DTM.

Two types of LiDAR exist: bathymetric and topographic. The difference between the two types is that bathymetric uses a green light that can penetrate water whilst topographic uses a near-infrared laser that rebounds off of open water (NOAA, 2016). The CoCT's LiDAR data is topographical LiDAR. Therefore, for the areas that held open water, such as Little Princess Vlei

or in the Diep River, the DTM represented the height above mean sea level of the water's surface rather than the ground surface. This meant that there were limitations in calculating channel sections or volumes from areas in the DTM that contained water.

4.1.2 Land use categorisation

To accurately model the runoff conditions in the Diep River subcatchment, it was necessary to approximate certain catchment parameters such as catchment imperviousness, depression storage and Manning coefficients for overland flow. Typical values for these parameters that are associated with various land uses are listed in literature. For this reason, it was necessary to obtain data on the land use classifications of the Diep River subcatchment. This study consulted James *et al.* (2010) and a report produced by SRK Consulting (2012) to approximate these parameters.

Land use data was obtained from the CoCT in the form of a GIS shapefile. The data provided the land use classification for each property in the CoCT metropolitan district. It also provided each property's demarcated boundary and registered property number. Unfortunately, the GIS shapefile required 'cleaning' as the data set contained duplicate entries for several individual properties whilst it also misclassified the land uses for a small number of properties. In order to rectify these errors, individual properties' land uses were confirmed by manually inspecting each property using an orthorectified 8 cm high resolution image whilst duplicate properties were deleted from the data set. A property's land use was only reclassified if it was undoubtedly incorrect.

4.1.3 Soil infiltration data

When stormwater runoff flows over pervious surfaces it experiences losses due to infiltration; this reduces the volume of runoff (Mitchell *et al.*, 2007). To model infiltration losses, it is necessary to know the soil properties of pervious areas in the catchment. Of the multiple infiltration models that *SWMM 5* provides (Horton infiltration, Green-Ampt infiltration and SCS Curve Number), it was decided to model infiltration using the Green-Ampt infiltration model as the soil parameters that the model requires are widely available in literature – for this study James *et al.* (2010) was consulted to estimate these the soil parameters.

The Diep River subcatchment's soil conditions were identified from a GIS shapefile that delineated the founding conditions (upper soil type zones) present in the catchment. This GIS shapefile was obtained from the CoCT. This data set was particularly useful for estimating infiltration parameters as the Green-Ampt model determines infiltration that occurs in the upper soil zone. The details of multiple boreholes that were drilled in the Diep River subcatchment's stormwater ponds were also acquired from the CoCT. The borehole details were used to identify the soil parameters within the stormwater ponds as well as to confirm the soil types surrounding the ponds that were described by the GIS shapefile.

4.1.4 Historic rainfall records

Mitchell *et al.* (2008) recommended that a rainfall time series of ten years should be the minimum length of rainfall used for analysing a continuous simulation of a stormwater harvesting system. This recommendation is based on the need to *'capture the influence of both intra- and inter-annual climate variability'.* Mitchell *et al.* (2008) further state that using longer rainfall records will produce more accurate estimations of performance indicators, particularly volumetric reliability (described in Section 2.5.2), whilst shorter records (e.g. one or two years' length time series) could lead to significant estimation inaccuracies. The chosen time-step interval is also an important consideration when analysing a stormwater harvesting system. Coombes & Barry (2007) found that simulations that used daily-time-steps significantly under-estimated stormwater yields compared to when the same simulations used a six-minute time-step. For continuous simulation models, five-minute time-interval data is preferred, as it accounts for the response time of the smallest subcatchments; however, acceptable estimations can still be produced using fifteen-minute or hourly time-interval data (James, 2009).

4.1.4.1 Collection and analysis of rainfall records

As described in Chapter 3, the rainfall that occurs in the Diep River subcatchment is highly variable. To represent this variance, it was necessary to obtain a number of rainfall records spatially distributed throughout the catchment that met the requirements as specified by Mitchell *et al.* (2008) and Coombes & Barry (2007). Obtaining rainfall records that matched the described level of detail proved to be a challenging exercise. This is largely attributed to the fact that the accessible gauges that recorded rainfall to the required level of detail were producing records that were undoubtedly incorrect or were not located within the catchment. After an extensive period of searching various sources, twenty separate rainfall records, of various length and time-interval, were collected. The sources of these records ranged from the CoCT, South African Weather Services (SAWS), Agriculture Research Council (ARC), Department of Water Affairs of South Africa (DWA) and private citizens. Details on the time-interval, length and reliability of these records can be seen in Appendix A whilst their location can be seen in Figure 4-1. Unfortunately, none of the rainfall records that were collected entirely matched the level of detail recommended by Mitchell *et al.* (2008) and Coombes & Barry (2007).

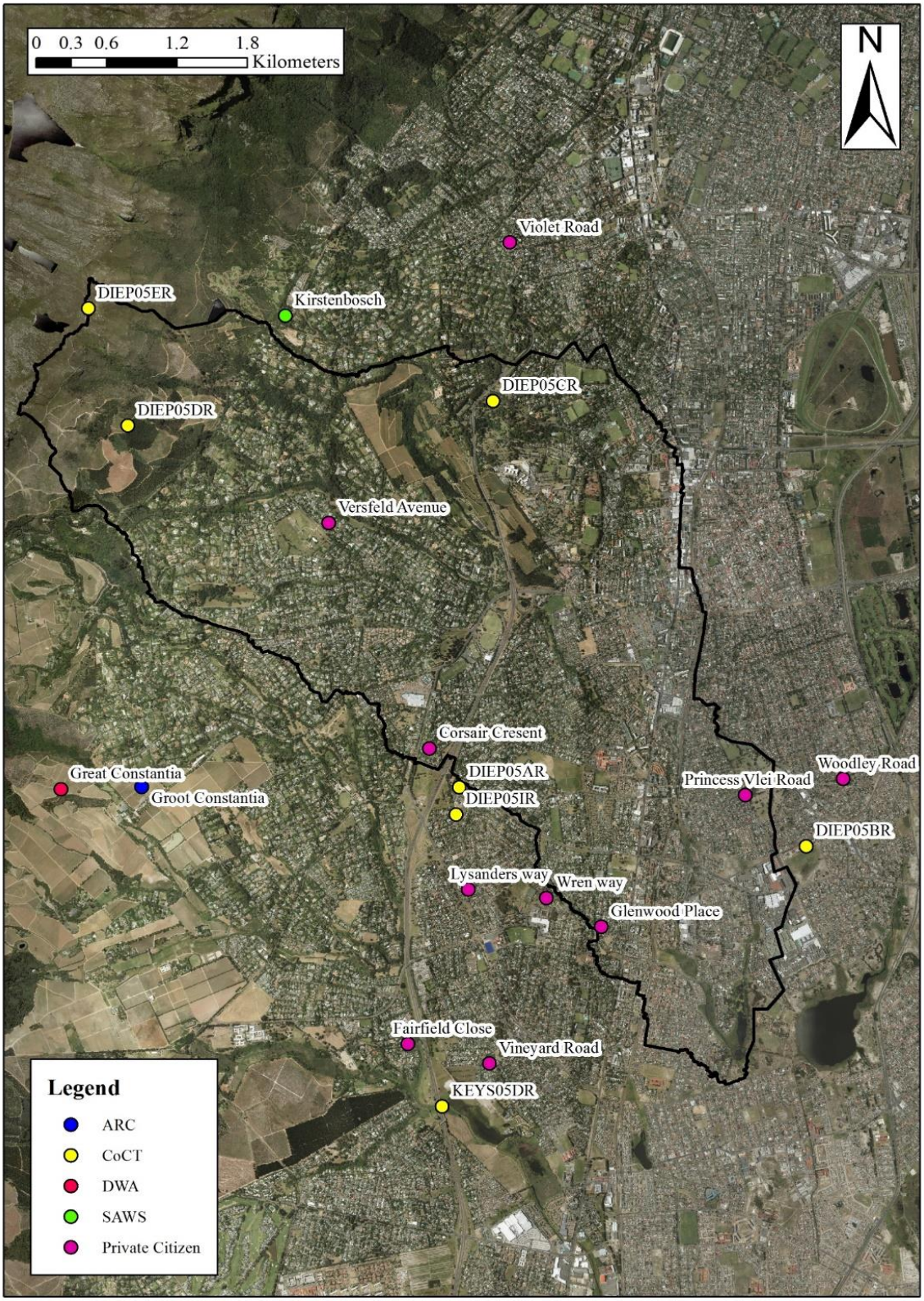

Figure 4-1: Location of collected rainfall records

Sub-hourly rainfall data (five-minute time-interval) is available for Cape Town. SAWS currently has several rain gauges positioned throughout the Cape Town metropolitan district that record rainfall to this level of detail. Whilst none of their gauges were located within the catchment, one of their gauges – Kirstenbosch – was located approximately 200 m outside the border of the upper reaches of the catchment in similar conditions to the upper region of the catchment. Similarly, like SAWS, the CoCT own a number of rainfall gauges that record rainfall at five-minute intervals. Seven of these gauges were located within or on the border of the catchment. After assessing the validity of their data however, only four of these records (DIEP05AR, DIEP05BR, DIEP05CR, DIEP05ER) were deemed to be reliable (i.e. comparable rainfall to neighbouring gauges and absence of erroneous data entries). Furthermore, due to the location of the catchment's flow gauge, described in Section 4.1.6, only DIEP05AR and DIEP05ER could be used for calibration purposes. Unfortunately, the length of these rainfall records proved to be a further limitation. Kirstenbosch had only six years' worth of sub-hourly data, whilst the CoCT only had two years of reliable sub-hourly data. Hence, simulations for this study could not rely on recorded sub-hourly data from rainfall gauges located within the catchment.

It was thus necessary to obtain additional records to get a better reflection of the Diep River subcatchment's rainfall, due to the limitation of the acquired sub-hourly data. In particular, longer rainfall records were needed to represent the catchment's rainfall history. It was therefore decided to request rainfall data from private citizens who had been keeping personal records. This was done by placing an advertisement in the local newspaper – the *Constantiaberg Bulletin* – as well contacting community groups such as the *Zandvlei Trust.* In total, data was obtained from nine separate individuals as well as from the ARC. The records ranged in length (1 year to 37 years) and time-interval (daily and monthly). The validity of each record was assessed based on: the record's correlation with other nearby rainfall records, the amount/ length of data gaps and consistency of record keeping. Additionally, when a private citizen's data was collected in person (i.e. not through email), the position of their rain gauge was inspected to ensure the measurements were not affected by rainfall interception. From the assessment, three rainfall records (Corsair Crescent, Versveld Avenue and Vineyard Road) were deemed acceptable to use as they provided a time series of sufficient length and quality. Corsair Crescent and Versveld Avenue however, required data patching to adjust readings where the record taker had indicated the reading represented more than one day or to fill gaps when the record taker had gone on a holiday. Data gaps were filled by linearly scaling the rainfall total for the absent period according to the rainfall pattern of the closest (reliable) rainfall gauge. Vineyard Road did not require data patching as the record taker has taken stringent measures to ensure readings were recorded every day. Overall, these records proved invaluable in depicting the rainfall characteristics of the catchment. Also, by comparing the MAP of a number of private records to the nearby CoCT gauges, it was possible to determine when the CoCT gauges were providing erroneous data.

By the end of the collection and analysis period, there were a number of rainfall records distributed throughout catchment that the study could use to describe the rainfall characteristics of the Diep River subcatchment. In total, three ten-year daily rainfall records from the records obtained from Corsair Crescent, Versveld Avenue and Vineyard Road were used for model

simulations whilst two sub-hourly records (DIEP05AR and DIEP05ER) were used for calibration purposes.

4.1.4.2 Disaggregation of daily rainfall

As mentioned in Section 4.1.4, Coombes & Barry (2007), Mitchell *et al.* (2008) and James (2009) reported that a continuous stormwater harvesting simulation requires sub-hourly rainfall data of at least ten-years in length. Unfortunately, whilst the three rainfall records that were to be used for model simulations (Corsair Crescent, Versveld Avenue and Vineyard Road) were of an adequate length, they had a daily time-step. Therefore, it was necessary to disaggregate these rainfall records in order to generate sub-hourly data. The rainfall disaggregation process is outlined in Figure 4-2. Whilst the shortest of these three rainfall records was about twelve years (Corsair Crescent), the hydrological period of analysis was limited to ten years (January 2005 – December 2014). This was because of the computational limitations of *PCSWMM 6.2*. Due to insufficient storage memory, *PCSWMM 6.2* cannot a simulate a time series with a fifteen-minute time-step that is longer than ten-years.

Hourly rainfall data was disaggregated from daily rainfall data using software called *Hyetos* (Koutsoyiannis & Onof, 2001). *Hyetos* is a software package for temporal stochastic disaggregation of rainfall based on the Bartlett-Lewis Rectangular Pulses Rainfall Model (BLRPM) that disaggregate daily rainfall to an hourly time-step. The model demands six input parameters (presented in Appendix C), which were derived using the descriptive statistics: mean, variance, lag-1 autocorrelation and proportion of dry periods for rainfall of one-hour, six-hour, twelve-hour and 24-hour time-intervals. These statistics were derived from the Kirstenbosch rainfall data, as it was the most reliable and longest sub-hourly time series that was collected. The three rainfall records (Corsair Crescent, Versveld Avenue and Vineyard Road) of adequate simulation length were then temporally disaggregated to hourly rainfall data. The disaggregated records were then refined to sub-hourly data (fifteen-minute time-interval) using *PCSWMM 6.2*. *PCSWMM 6.2* offers a disaggregation tool that also incorporates the BLRPM, but only reduces hourly data to fifteen-minute data. Again, hourly data was reduced based on the Kirstenbosch rainfall data statistics. Overall, the disaggregation process provided three statistically probable ten-year sub-hourly rainfall records. Whilst disaggregation does not produce the actual rainfall that occurred, it produces sub-hourly rainfall that statistically represents what rainfall could have occurred. Further, it preserves the total daily rainfall and its temporal nature.

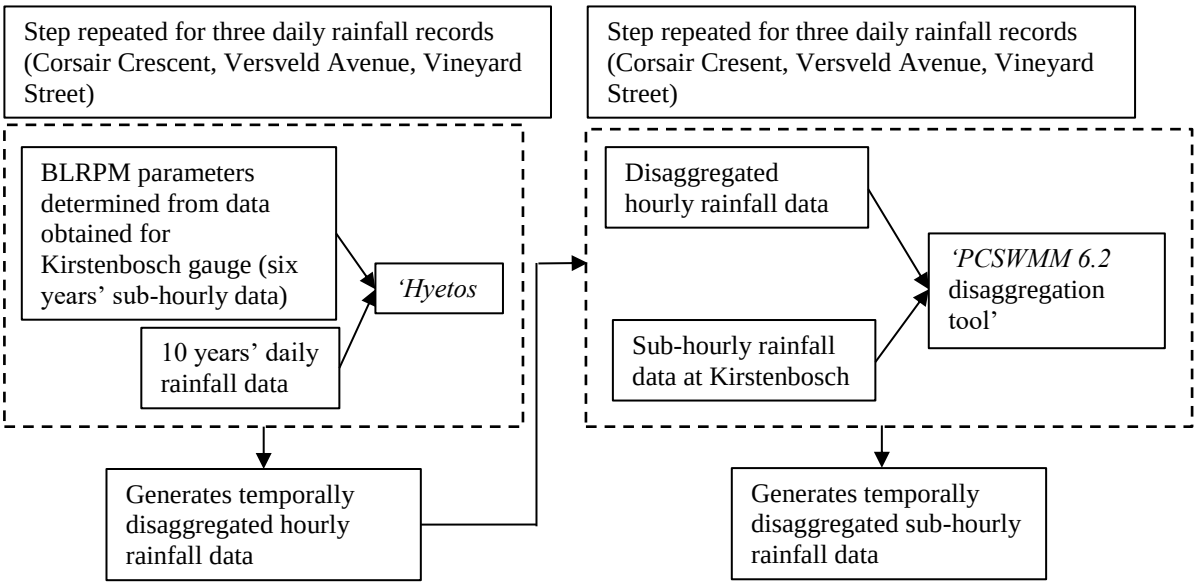

Figure 4-2: Schematic of rainfall disaggregation process

4.1.5 Evaporation data

It is important to model evaporation as, like infiltration, evaporation will incur water losses from the system. Unfortunately, for this study, it was not possible to obtain evaporation data as there was no operating evaporation gauging station positioned within the catchment area. The DWA previously recorded daily evaporation rates within the catchment at their Constantia station; however, this evaporation gauge has been out of operation since 1990, long before the period of analysis. As a result, daily evaporation data had to be computed using *PCSWMM 6.2,* which derives evaporation totals using the Hargreaves' method. The Hargreaves method is an empirical equation that uses temperature data to calculate evaporation totals. This method has been shown to provide satisfactory results (Xu & Singh, 2001; Allen *et al.*, 2006). The historic temperature data, *viz.* the daily maximum and minimum, that the Hargreaves' method requires was obtained from the ARC's Constantia station (Figure 4-1). As there was no observed evaporation data for the period of analysis, the computed evaporation data could not be calibrated. However, the computed data was deemed valid as it was comparable to the historic monthly evaporation totals of the Constantia station before it became defunct.

4.1.6 Flow data

For this study, continuous flow data was required to calibrate the catchment stormwater model. Currently, the CoCT monitors the flow rates of a number of their major rivers and channels at flow monitoring stations positioned throughout the city. Three of these stations, Wynb05bS, Diep05cS and Lpvl05aS, are located in the upper, middle and lower regions of the catchment, respectively (Figure 4-3). Unfortunately, sub-hourly continuous flow data was only available

subsequent to 2012. Further, due to the lack of maintenance, some of the stations were clearly giving false readings; Wynb05bS and Lpvl05aS, were two such stations – all data entries had the same value. After scrutinising Diep05cS's readings, the gauge was judged to be reliable as readings fluctuated in accordance to rainfall events. This meant the catchment's only working gauge was positioned in the middle of the catchment. This limited the accuracy of the calibration as it meant that a significant portion of the contributing subcatchments were ungauged.

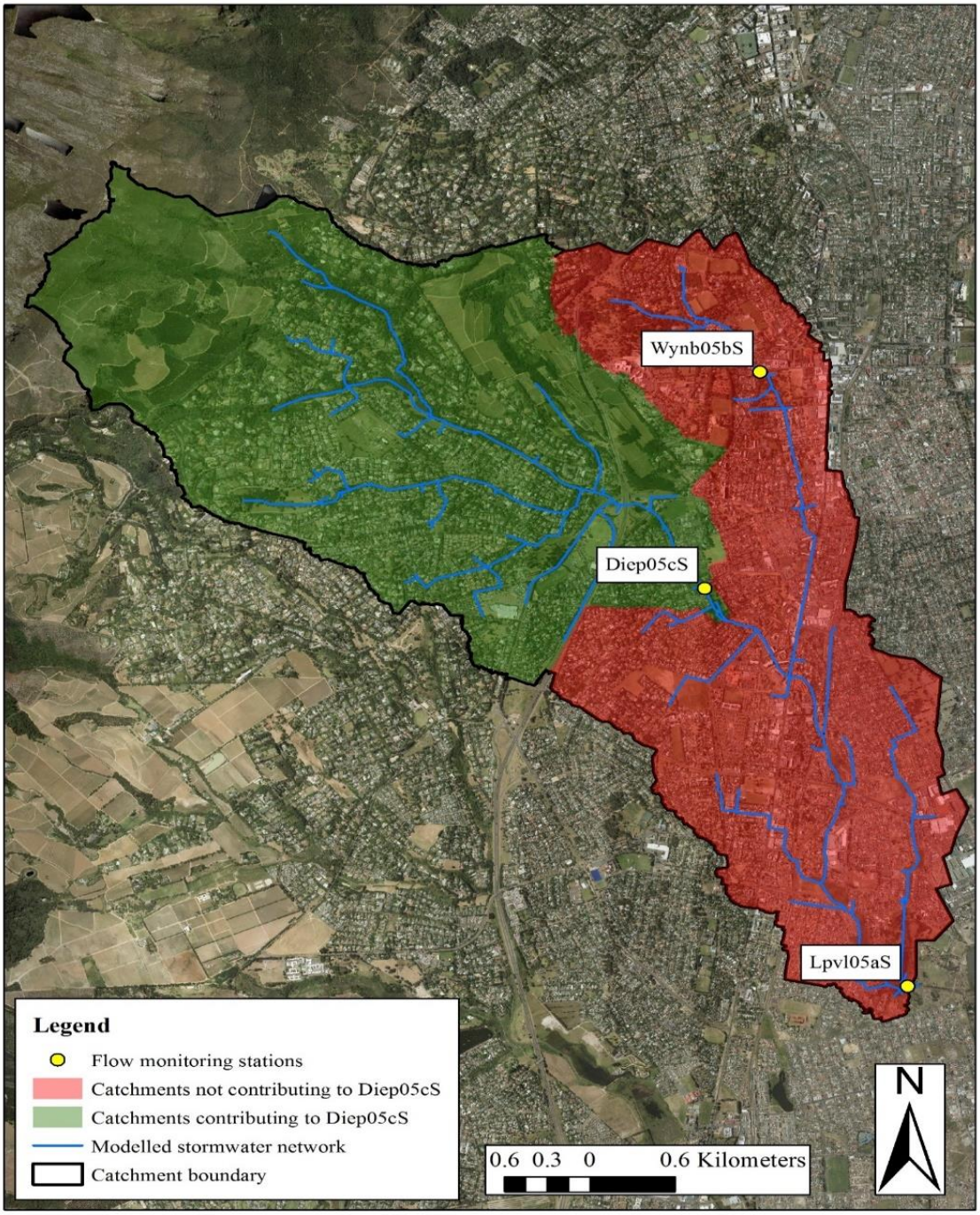

Figure 4-3: Position of flow monitoring stations

Since the CoCT's flow monitoring stations record the water depth in their respective channels, it was necessary to obtain each monitoring station's rating curve to convert the water depths (m) into flow rates (m^3/s). Whilst the rating curve for both Wynb05bS and Lpvl05aS were obtained from CoCT reports, their erroneous data meant that the converted flow rates were unusable. For Diep05cS, the rating curve had to be derived from a calibration table that was obtained from the CoCT. The calibration table related water depth to flow rate through a Parshall flume located at the monitoring station. The calibration table listed water depths from 0 - 2 m at intervals of 0.05 m. The rating curve that was derived from the calibration table can be seen in Figure 4-4.

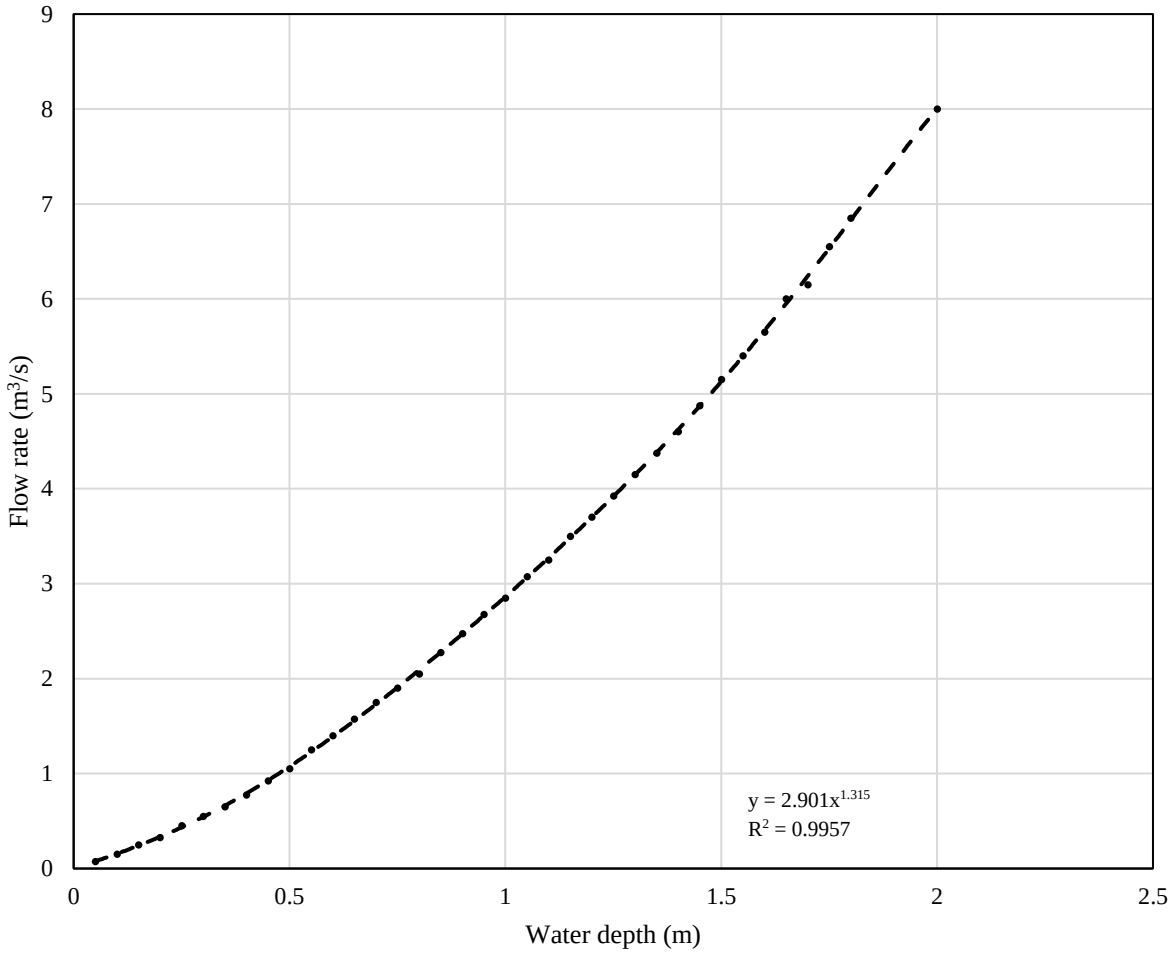

Figure 4-4: Generated rating curve of Diep05cS monitoring station's Parshall Flume

Whilst the data for Diep05cS was deemed reliable, it was discovered that during 'dry' periods there were readings that had zero flow. At first, it was assumed that this was caused as a result of errors in the flow readings as it was understood that the Diep River was a perennial stream (Brown & Magoba, 2009). A field inspection of the flow gauge, during a 'dry' period, established that the flow gauge was in working order; however, owing to minor river bed erosion at the location of the gauge, it was observed that very low flows did not pass through the flow gauge.

For example, at the time of inspection, the water level was ± 0.01 m in the channel centre at the gauge position, yet, it fell just below the gauge inlet. Based on this observation, it was assumed that the gauge did not record river flows with a water depth less than 0.02 m. Hence, whilst it did appear that the Diep River has perennial flow, it was not possible to model periods in which 'dry' weather flow occurred. This had ramifications on the calibration accuracy – described in Section 4.2.5.

4.1.7 Water quality

The CoCT has monitored the environmental quality of its rivers' water for several decades. Throughout the city, there are sampling points from which the CoCT takes monthly grab samples. Nine of these sampling points are found along the Diep River with some records dating back to 1989. Not all sampling points are still used however, only three of the locations provided results up until the end of 2015 (end of period of analysis). From these monthly grab samples, the CoCT tests the following parameters: Temperature, Dissolved Oxygen, Salinity, Suspended Solids, pH, Conductivity, Ammonia, Nitrites and Nitrates, Orthophosphates, Total Phosphorus (TP) and *E.coli*. Unfortunately, due to the low resolution of the water quality data (monthly time-step), it was not possible to create a calibrated water quality model. To model intricate water quality variations over time reliably, data of a much finer time-interval is required for calibration. Data of this detail was not available for this study. It was not possible to establish reasonable Event Mean Concentration (EMC) parameters for wet weather event estimates using a monthly time-step. The acquired data did have some use however, as it was necessary to know current quality of water in the catchment to determine the level of treatment harvested stormwater required (Section 4.3.4).

4.1.8 Water demand disaggregation

In a stormwater harvesting system, the water demand dictates when and what volume of water is extracted from the system's storage facility. Consequently, it is important that the water demand data used within the model realistically represents the spatial and temporal nature of the water demand (Fisher-Jeffes, 2015). For this reason, water demands for various residential and commercial end-uses were estimated. Jacobs & Haarhoff (2004) defines an end-use as the smallest identifiable use of water on a stand and can be categorised as either an indoor end-use or outdoor end-use. Monthly water demand data for 2013 was retrieved from the CoCT; however, as it only indicated total monthly water usage use per property, it was necessary to disaggregate this data into its end-uses and to an hourly time-step. The process used to disaggregate the available water demand data was adapted from the method used by Fisher-Jeffes (2015).

4.1.8.1 Average single residential household size

The water demand disaggregation process required the average household size per suburb. As StatsSA (2011) does not provide South Africa's 2011 census data at household level, the average household size had to be estimated from small area level data. To ensure anonymity, small area level data provides population information at a cluster level; thus, households are grouped into a small area that include a minimum of 500 people (Grobbelaar, 2005). As this data provides a count of household sizes (the number of people), it was possible to determine the median household size per suburb (which incorporates a number of 'small areas'). The median household size was determined for the 21 different suburbs that were located either within the Diep River subcatchment or on the periphery of the catchment. This process used information from 98 different small areas. From these calculations, it was established that the median household size across the catchment was almost uniform. All but two suburbs, Plumstead and Diep River, had a median household size of three people. It was decided to use median rather than mean so that the representative household size was not distorted by the presence of outliers. The median household sizes per suburb can be seen below in Table 4-1.

Table 4-1: Household size per suburb

Suburb	Household size (Capita per household)	Suburb	Household size (Capita per household)
Alphen	3	Heathfield	3
Barbarossa	3	Kenilworth	3
Bel Ombre	3	Meadowridge	3
Bergvliet	3	Plumstead	2
Bishopscourt	3	Sillery	3
Constantia Heights	3	Silverhurst	3
Constantia Vale	3	Southfield	3
Deurdrif	3	The Vines (Constantia)	3
Diep River	2	Witteboomen	3
Elfindale	3	Wynberg	3
Graylands	3		

4.1.8.2 Single residential indoor and outdoor water demand calculation

This section only describes how the water usage data for single residential properties (SRPs) was processed. Unfortunately, the water demand data acquired from the CoCT had limitations, most notably in regards to residential properties, and required 'cleaning' to discard erroneous data. These errors were typically caused from missing data entries or large monthly demand

fluctuations. In order to 'clean' the available data set, water usage records that met the following criterion were deemed valid:

- The water usage for every month was greater than zero. Leaks are common within South African households so it was considered unlikely that for any month, a property would have zero water use (Couvelis, 2012). For this reason, it was assumed that months with zero water usage were a result of a water meter error rather than a period of absence of the homeowner.
- The average monthly demand was between 6 kℓ to 250 kℓ per month. South Africa's Free Basic Water policy states every household in South Africa is entitled to 6 kℓ of water per month free of charge (CSIR, 2005). It was assumed that properties who use less than this would not make use of stormwater harvesting to augment their water supply. Water usage that was greater than 250 kℓ per month was deemed to be unrealistic as 250 kℓ per month is more than double the maximum monthly water demand for residential usage recommended by CSIR (2005). For this reason, properties whose water usage exceed 250 kℓ per month were ignored.
- It was assumed that summer demand should exceed the winter water demands. The Diep River subcatchment is located in an area that experiences seasonal winter rainfall. Therefore, it was assumed irrigation is generally not needed during winter. On the other hand, the area's warm, dry summers suggests that most properties would likely have a substantial irrigation demand during summer months. This assumption would also eliminate abnormalities that would have been caused by irregular behaviour that cannot be readily reconciled with end-use modelling (Fisher-Jeffes, 2015).
- The winter water demands should be reasonably constant in order to disregard irregular water usage (e.g. pipe bursts). Therefore, the minimum monthly winter demand should not be less than 40% of the maximum monthly water demand (Fisher-Jeffes, 2015).

With the application of the criteria listed above, half of the records were discarded as their water usage records were deemed invalid – largely due to missing data.

As the CoCT only provides one water meter to each individual property, indoor and outdoor water usage is recorded together. However, Cape Town experiences winter rainfall, so it was assumed that outdoor water usage (e.g. garden irrigation and swimming pool filling) generally only occurs during summer (Figure 4-5). This seasonal variation in water usage meant that indoor water usage could be extracted from the total monthly water usage based on the minimum-month method (Dziegielewski *et al.*, 1992; Palenchar *et al.*, 2009). The minimum-month method relies on the fact that indoor water usage has limited variability throughout the year as it excludes seasonal end-uses such as garden irrigation or swimming pool refilling (evaporation dependent activities) (Mayer *et al.*, 1999). It assumes that during winter, household water usage reflects solely indoor water use, as garden irrigation is not needed. However, this method is only applicable to areas that have a mild climate and irrigation is not practiced all year

round (Dziegielewski *et al.*, 1992; Mayer *et al.*, 2009; Palenchar *et al.*, 2009). Mayer *et al.* (1999), Palenchar *et al.* (2009), Coulson (2013) and Fisher-Jeffes (2015) all used this method to estimate indoor water usage.

The minimum-month method was used to estimate indoor water demands from the data that was deemed valid after 'cleaning'. However, like Coulson (2013) and Fisher-Jeffes (2015), monthly indoor water demand was represented by an SRP's average winter monthly water usage (May to August) rather than its absolute minimum monthly water usage (Figure 4-5). After categorising each property's indoor water demand according to their respective suburb, the median indoor water demand was determined for each suburb. It was decided that leaks should not represent an indoor end-use. Thus, estimated leakages were subtracted from the median indoor water demand using the findings of Couvelis (2012), who determined the average leakage rate for properties in various suburbs throughout Cape Town.

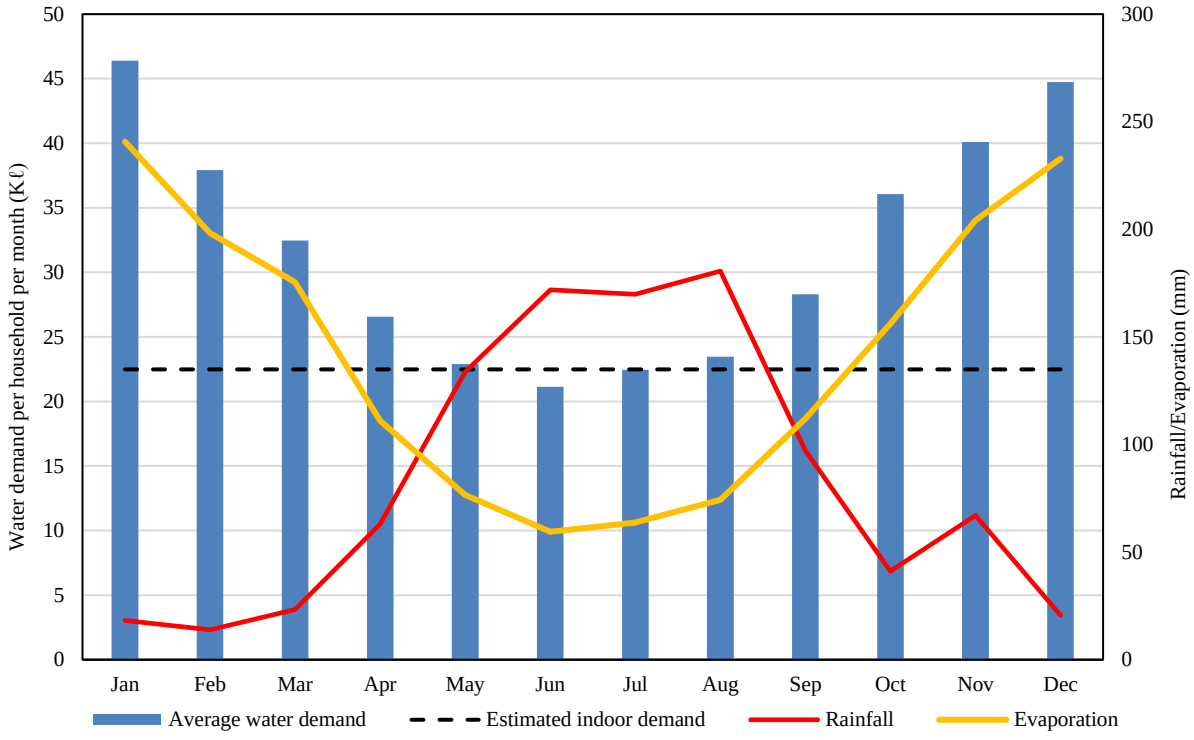

Figure 4-5: Seasonal fluctuation patterns of monthly water usage, rainfall, and evaporation for the Diep River subcatchment

The median indoor water demand was then used to determine a representative indoor water demand per capita. It was decided to use median rather than mean so that the representative indoor water demand was not distorted by the presence of outliers. Unfortunately, as the monthly water usage records obtained from the CoCT were for a period that occurred two years after the latest South African census took place, it was uncertain if the median household sizes had

changed. However, it was assumed that the median household size remained unchanged from 2011 to 2013. By dividing the median indoor water demand for each suburb by its respective median household size, it was possible to determine a representative indoor water demand per capita – Equation 4- 1.

$$Indoor\ water\ demand\ per\ capita = \frac{Median\ household\ indoor\ water\ demand}{Median\ household\ size} \quad 4\text{-}1$$

Indoor water demand per capita for each suburb was divided by 30.4 (average number of days in a month) in order to represent a daily indoor water demand per capita – i.e. the estimated indoor annual average daily demand (AADD). The estimated indoor AADD value for each suburb is presented in Table 4-2.

Table 4-2: Estimate indoor AADD per suburb

Suburb	Estimated indoor AADD (ℓ/capita.day)	Suburb	Estimated indoor AADD (ℓ/capita.day)
Alphen	280	Heathfield	170
Barbarossa	210	Kenilworth	260
Bel Ombre	270	Meadowridge	180
Bergvliet	180	Plumstead	240
Bishopscourt	360	Sillery	270
Constantia Heights	280	Southfield	150
Constantia Vale	300	Silverhurst	250
Deurdrif	220	The Vines (Constantia)	210
Diep River	250	Witteboomen	290
Elfindale	170	Wynberg	150
Graylands	270		

Probable indoor water demands were assigned to all the properties whose data was regarded as invalid during the data 'cleaning' process. The probable indoor water demand equalled the product of the indoor AADD estimate and household size for the property's respective suburb. This value was then multiplied by 30.4 so that it would represent the monthly water demand (MWD). This process is represented by the following equation:

$$(MWD\ per\ Household)_{suburb} = (indoor\ AADD\ estimates_{suburb} \cdot capita\ per\ household_{suburb}) \cdot 30.4 \quad 4\text{-}2$$

Where: suburb = the household's respective suburb

For the properties that had reliable data, their outdoor water use for summer months was determined by the subtracting the properties average indoor water usage from each summer month (September to April). These outdoor water use values were then used later in Section 4.1.8.5 for calibration purposes.

4.1.8.3 Residential flat blocks indoor water demand calculation

Like water demands for SRPs, the water usage data for residential flat blocks – properties that contain multiple self-contained housing units (flats) – required 'cleaning'. Fortunately, this process was far less intricate in comparison to SRP's data cleaning. Water usage records that were deemed valid met the following criterion:

- Residential flat blocks that had zero usage for a single month were deemed invalid. It was assumed that in a flat block building with multiple households, there would always be at least one household using water during the month. Equally, as leaks are common in South African properties, the leakage volume would create monthly usage data.
- Like SRPs, properties whose monthly water usage was less than 6 kℓ were not considered for the study.
- In order to discard irregular water usage (e.g. pipe bursts), residential flat blocks whose minimum monthly water usage was less than 40% of the maximum monthly water usage were regarded as invalid (Fisher-Jeffes, 2015).

As the CoCT only issues a single water meter to each residential flat block, the water usage data of each property represents the water usage of multiple households as well as any water used for communal areas on the property. It was assumed that the households contained within a residential flat block did not have their own private gardens and so the water used by these households only reflects indoor uses. Some residential flat blocks however, have a community garden or swimming pool on the property that creates an outdoor water demand. Therefore, indoor water demands were estimated for each residential flat block, based on the following assumptions:

- The indoor water demand of a residential flat block was represented by its average monthly water demand if it was determined that the property did not have a significant outdoor water demand. Properties were considered not to have an outdoor demand under two conditions:

$$80\% \text{ of average winter demand} \leq \text{average summer demand} \leq \text{average winter demand}$$

or

$$80\% \text{ of average summer demand} \leq \text{average winter demand} \leq \text{average summer demand}$$

These conditions identified properties that had a relatively constant monthly water usage, which was assumed to reflect only indoor use.

- The indoor water demand of a residential flat block was represented by its average winter demand if a property's average winter demand was less than 80% of the average summer demand. This assumption aimed to identify seasonal water usage.

The indoor water demand for residential flat blocks whose water usage data did not meet the above criteria (i.e. twelve properties did not) were estimated according to the following process:

i) Each flat block was individually inspected to determine how many flats the property contained. The inspection was performed using *Google Street View*. The number of flats in each residential flat block could typically be determined by counting the number of letter boxes belonging to the property or counting the number of buttons on the property's intercom.

ii) A median household size was assigned to each residential flat block based on the suburb in which it was located using median household sizes per suburb established in Section 4.1.8.1.

iii) An estimate indoor AADD was assigned to each flat block based on the suburb in which it was located using the estimated indoor AADDs determined in Section 4.1.8.2.

iv) Each flat block's indoor water demand was estimated by multiplying the residential flat block's number of flats by the assigned median household size and its assigned indoor AADD estimate.

4.1.8.4 Commercial indoor water demand calculation

Out of the entire water demand disaggregation process, commercial water usage data required the least rigorous processing. Only seventy commercial properties, situated in the general retail, office and manufacturing sectors, were selected for this study as they were in close vicinity to two of the stormwater ponds. After an inspection of these properties through *Google Street View*, it was decided that water usage for these properties was purely for indoor processes. Therefore, indoor water demand did not have to be extracted from the total water usage. Consequently, the commercial water usage data was subjected to the following criteria to assess its validity:

- Commercial properties that had zero water usage for any month were deemed invalid. Leaks are common in South African properties thus it deemed to be unlikely that a property would ever have zero water usage.
- To exclude highly irregular water use, properties whose minimum monthly water use was less than 30% of its maximum monthly use were discarded. This criterion was lower in comparison to flats blocks or SRPs as commercial indoor water use can experience

seasonal fluctuations; for example, during the Christmas period, retail stores will experience increased business whilst manufacturing business may shut down production.

With the application of the criteria listed above, only eleven of the seventy records were discarded as their water usage records were deemed invalid.

The indoor water demand of the properties whose data was discarded was estimated based on the relationship between property area to average monthly water demand. This relationship was developed using the property area and average monthly water demands of the properties whose water data was deemed valid. The relationship can be seen in Figure 4-6. This was adapted from the method used by CSIR (2005) who recommends that the AADD for shops and offices is equal to 400 ℓ/day per 100 m^2 of gross floor area.

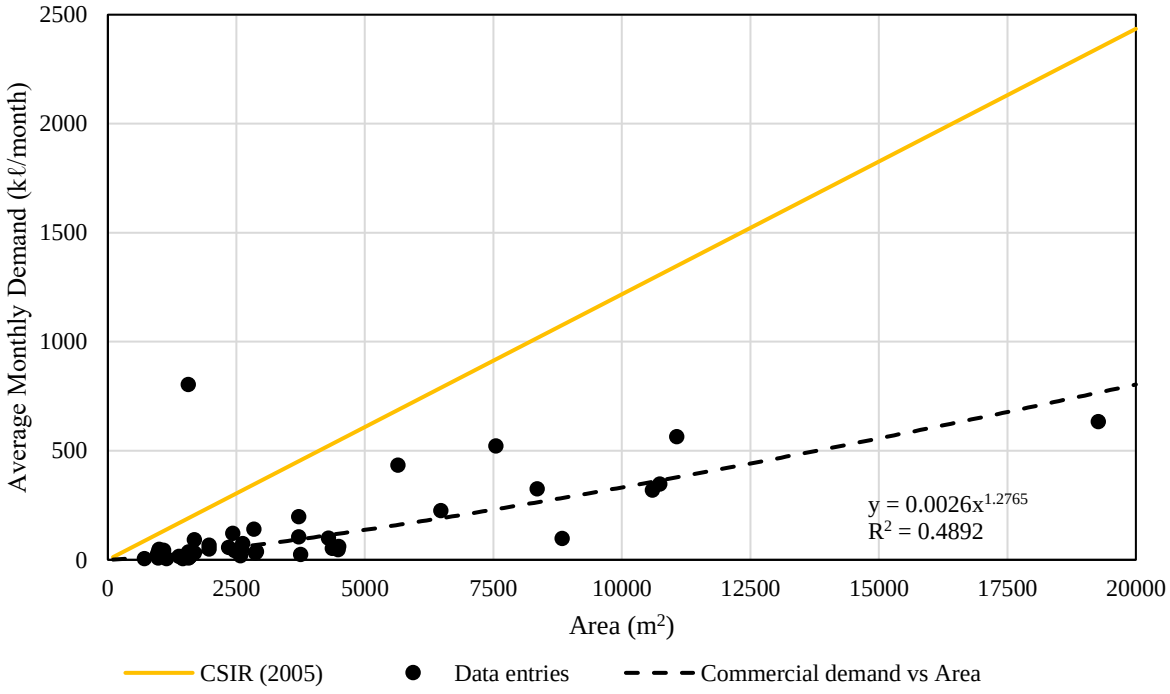

Figure 4-6: Average monthly demand vs. area for commercial properties

4.1.8.5 End-use determination

Water end-uses can be estimated using end-use models such as Jacobs & Haarhoff's (2004) Residential End-Use Model (REUM). These models often require numerous parameters for each end-use that also require calibration. For example, Jacobs & Haarhoff (2004) state that 48 parameters are required to estimate end-uses for twelve different residential indoor end-uses. Since it was not possible to obtain data at the level detail required to reliably estimate these parameters, it was decided not to use an end-use model to estimate indoor end-use demands.

Instead, indoor end-use demands, for both residential and commercial property, were derived from the total indoor water demand using indicative figures obtained from literature. The percentage allocation of total residential indoor water demand (both residential flats blocks and SRPs) to various indoor end-uses is shown in Figure 4-7. In regards to commercial properties, toilet demand was the only indoor end-use that was considered. Since the commercial properties considered in this study consisted of general retail stores, offices and manufacturing businesses, the toilet demands were derived from the total indoor demand using the percentage allocation shown in Table 4-3.

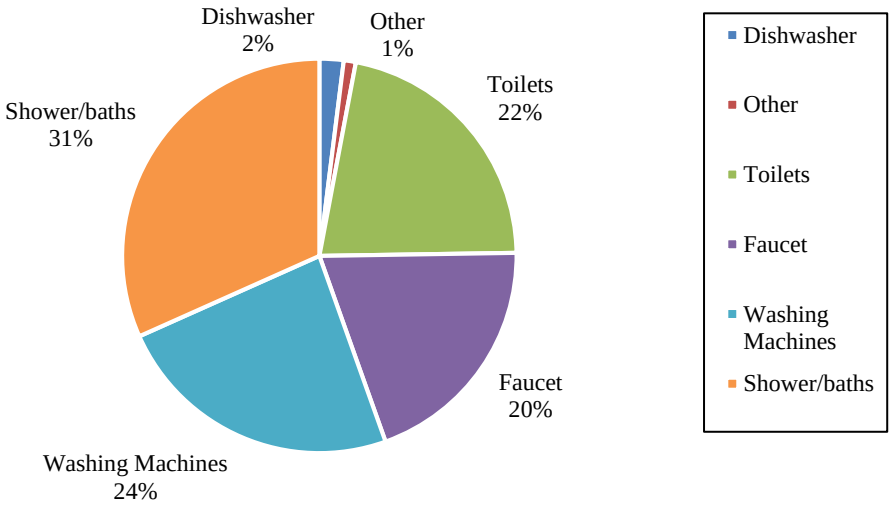

Figure 4-7: Typical breakdown of end-use for residential indoor water demand derived from international studies (Mayer *et al.*, 1999; Roberts, 2005; Willis *et al.*, 2010; Beal & Stewart, 2011; Fisher-Jeffes, 2015)

Table 4-3: The percentage allocation of total indoor water demand to toilets for commercial properties (Dziegielewski *et al.*, 2000; Gleick *et al.*, 2003)

Type of commercial Property	Percentage allocation of total indoor water demand to toilets
General retail	41%
Offices	41%
Manufacturing	5%

Since outdoor demand could be separated into two end-uses (i.e. garden irrigation and swimming pool filling) which required substantially less parameters than indoor end-uses, it was decided that Jacobs & Haarhoff's (2004) REUM could be used to provide reliable estimates of outdoor end-use demands. Other outdoor end-uses such as car washing were disregarded in this study as

they were considered to have a negligible effect on total outdoor water demand (Fisher-Jeffes, 2015). The equation used in Jacobs & Haarhoff's (2004) REUM model calculates the annual monthly daily demand (AMDD) for outdoor water demand. For calibration purposes, these equations were modified in order to calculate MWD. As a result, the following equation was used:

$$MWD_{m,e} = \left(f_{m,e} \cdot s_e\right) \cdot \left(\left(k_{m,e} \cdot p_f\right) - r_m\right) \quad 4\text{-}3$$

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

$$MWD_o = \sum_{e}^{2} MWD_{m,e} \quad 4\text{-}5$$

Where: MWD = monthly water demand (kℓ/month); m = month; e = end-use; f = garden irrigation factor/factor for pool cover use; s = surface area of irrigated area/surface area of pool water (m^2); k = crop factor or pool factor; $p_{A\text{-}pan}$ = pan evaporation (mm); r = effective rainfall; R = monthly rainfall (mm/month); o = outdoor.

The outdoor end-use water demands were calculated using REUM for each property neighbouring or contained within the Diep River subcatchment. REUM requires a number of input parameters that are difficult to obtain (i.e. k_m; $s_{garden\ irrigation}$; s_{pool}; $f_{garden\ irrigation}$; f_{pool}); fortunately, these parameters can be estimated from a summary table of typical parameter estimates in Jacobs & Haarhoff (2004). However, this meant that it was then necessary to calibrate these demands against known outdoor water demands in order to refine the estimated input parameters. As outdoor water demand is dependent on evaporation, each summer month had to be individually calibrated to determine the month's parameters. Since approximately half of the properties did not have reliable data, outdoor demands were calibrated using suburbs' median outdoor demand. The outdoor demands of properties with reliable data was determined by subtracting winter indoor demand from the total demand. As there were fifteen different suburbs used within this study, this would entail calibrating for five different parameters (k_m; $s_{garden\ irrigation}$; s_{pool}; $f_{garden\ irrigation}$; f_{pool}) for eight different months (September to April) for each of the fifteen suburbs. In order to simplify the calibration process, suburbs that neighboured one another, had similar indoor water demands and property sizes were combined to form larger regions. Due to the large degree of uncertainty over the value of each parameter that needed to be calibrated, it was judged that combining suburbs would not worsen the accuracy of the outdoor water demand estimation. The suburbs that were combined to form larger regions can be seen in Table 4-4.

Table 4-4: Combined suburbs for outdoor water demand calibration

Region	Suburbs contained in region
1	Alphen, Bishopscourt, Constantia Heights
2	Diep River, Plumstead, Wynberg
3	Barbarossa, Deurdrif, The Vines (Constantia)
4	Bel Ombre, Graylands, Sillery, Silverhurst, Witteboomen
5	Elfindale, Heathfield, Southfield

The known parameters that did not require calibration were monthly rainfall (R) and pan evaporation (p); monthly rainfall values that were obtained from the rainfall data described in Section 4.1.4 whilst monthly pan evaporation values were obtained from historic evaporation data from the DWA's Constantia gauge. For the parameters that did require calibration, the following assumptions were made to simplify the calibration process:

- A single crop factor (varied per month) that accounted for all vegetation types was used to model garden irrigation. While in reality, properties would likely have various types of vegetation in their gardens, the crop factor could be adjusted through calibration as an effective composition for this simplification.
- Based on a visual inspection using the orthorectified 8 cm high resolution image, it was apparent no property in Region 5 had a swimming pool. However, it appeared the majority of SRPs in the other regions had a swimming pool that did not have a cover. Thus, it was assumed that all properties, excluding those situated Region 5, had a swimming pool that did not have a swimming pool cover.
- It was assumed that the percentage of each property's stand area that is irrigated was constant for properties that were situated in the same region. Similarly, it was assumed that properties in the same region all had a swimming pool of a uniform surface area.

Jacobs & Haarhoff (2004) list typical parameters for residential properties that are required to calculate the various outdoor end-uses. These parameters were used to guide parameter estimations used in this study. Jacobs & Haarhoff (2004) state that the irrigated surface, which remains constant throughout the year, is typically 15 – 35% of the total property area for residential properties. To determine if these boundaries were appropriate, a number of properties, selected randomly, situated in each different region were inspected to assess if the size of their irrigated surface area ($s_{m,e}$) fell within these boundaries. As a result of the inspection, it was decided to extend the upper parameter boundary of irrigated surface area ($s_{m,e}$) to 55% as the irrigated surface area ($s_{m,e}$) of certain properties situation in Region 1 was found to be more than the recommended 35% of the stand area. The parameters recommended for swimming pool surface area ($s_{m,e}$) were checked in a similar manner. From a visual inspection of the orthorectified 8 cm high resolution image, it appeared that coastal tropical forest vegetation and

Kikuyu grass were the dominant vegetation types. Thus, initial crop factor estimates were also estimated from parameters listed in Jacobs & Haarof (2004).

Initially, only the irrigation factor ($f_{m,e}$) was calibrated after initial estimates were made. Calibration was performed through an iterative process (using *Microsoft Excel 2016's Solver* function) until the median modelled outdoor demand approximately equalled the actual median outdoor demand. The remaining parameters were then adjusted to 'fine-tune' the calibration. However, it was stipulated that the parameter had to remain with the boundaries stated in Table 4-5.

Table 4-5: Parameter boundaries used within calibration (after Jacobs & Haarhoff, 2004)

End-use parameter	Units	Parameter boundary		Note
		High	Low	
Garden Irrigation factor ($f_{m,\ garden\ irrigation}$)	dimensionless	2	0	A
Irrigation surface area ($s_{m,\ garden\ irrigation}$)	Percentage of stand area	55%	15%	B
Swimming pool surface area ($s_{m,\ swimming\ pool}$)	m^2	60	12	B
Pool evaporation factor ($f_{m,\ pool\ evaporation}$)	dimensionless	0	1	A
Monthly crop factor (K_m)	dimensionless	0.5	0.75	C

Notes:

A. Value can vary per month

B. Value must be consistent for each month throughout the year

C. For months January to April, the value should decrease each subsequent month from a maximum value in January with the minimum value occurring in April. For months September to December, the value should increase each subsequent month from a minimum value in September to a maximum value in December. The values for December and January should be equal. Months May to August were not considered as it was assumed that there was no outdoor demand during these months

4.1.8.6 Diurnal water use patterns

The volume available in a storage facility to attenuate a flooding event is reduced when using the storage facility to store stormwater. However, the act of harvesting stormwater from a storage facility increases the volume available in the storage facility for flood attenuation. Thus, to simulate the effect of stormwater harvesting, it is important that both the inflow and outflow (water demands) of the stormwater harvesting storage facility is modelled with at least an hourly time-step (Fisher-Jeffes, 2015). As with the derivation of indoor end-uses, the water demand thus was disaggregated based on recommendations from literature as high-resolution end-use data was not available. Diurnal water demands were superimposed on water demands to disaggregate them to an hourly time-step. Naturally, as residential and commercial properties use water in different manners, they had different diurnal water demand patterns. Further, two different diurnal water demand patterns were used for the two main types of commercial properties: a

'general retail' diurnal pattern and a 'offices and manufacturing' diurnal pattern. It was decided that offices and manufacturing properties would follow the same diurnal pattern as these properties were likely to follow conventional working hours. The various diurnal water demand patterns that were used in this study were adapted from the research presented by Funk & DeOreo (2011) and are shown in Figure 4-8. These diurnal patterns were established for properties in California, United States of America (USA) of similar socio-economic and climatic conditions to the properties considered in this study. Like Cape Town, California has a Mediterranean climate. Furthermore, California's most populated areas experience a similar Mean Annual Precipitation (MAP) as Cape Town.

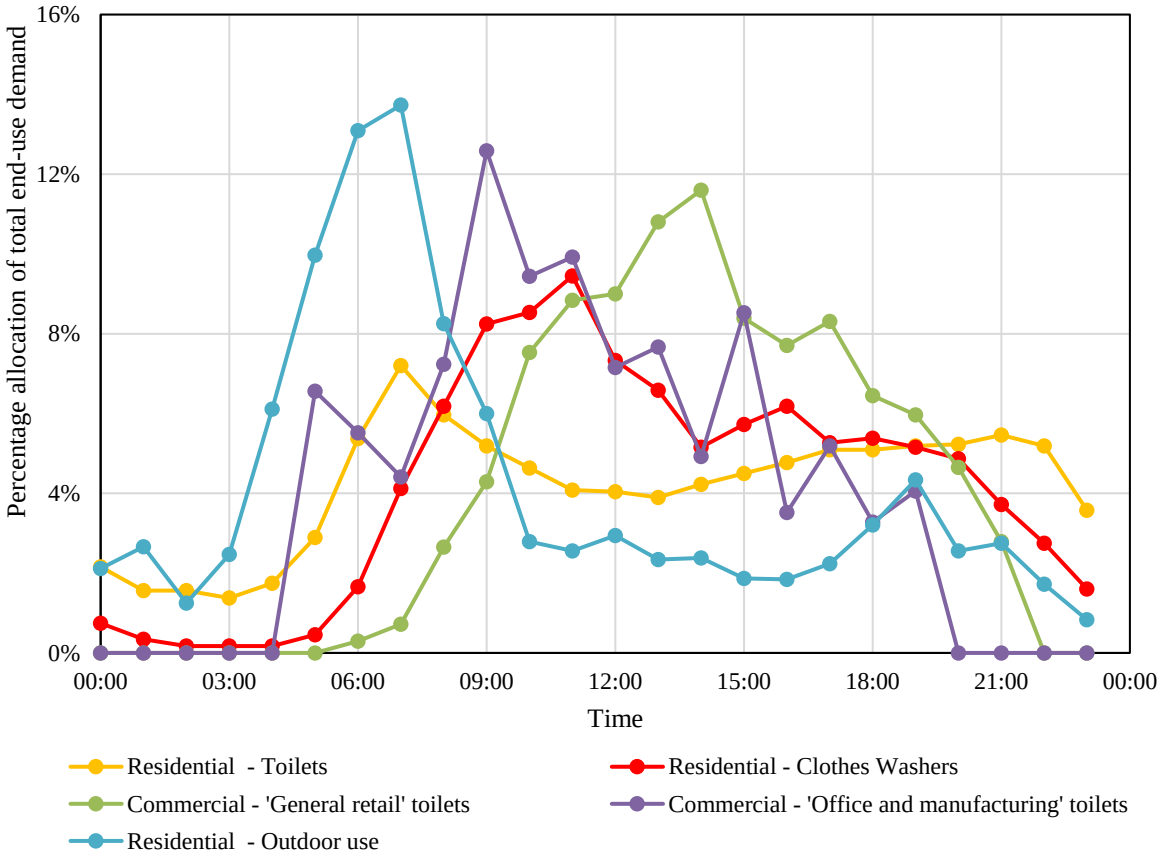

Figure 4-8: Diurnal pattern for various end-uses (Funk & DeOreo, 2011)

4.1.9 Economic data

The following section details the sources from which economic data was collected. Economic data was critical to the study as it was used to establish the monetary cost of each stormwater harvesting scenario. All costs used in this study included Value Added Tax (VAT). In South Africa, VAT is 14% and has been so since 1993 (PATC, 2016).

4.1.9.1 Capital costs

For this study, capital data was obtained from the following sources:

- Like Fisher-Jeffes (2015), the cost of each stormwater harvesting scenario's non-potable water reticulation network (described in Section 4.3.3) was based on the work presented by Bester *et al.* (2010). Bester *et al.* (2010) developed simple cost-functions that can be used to predict the typical cost of gravity and pressure pipelines of various materials or diameters (costs includes all common components) in South Africa. Additionally, the cost-functions presented in Marchionni *et al.* (2015) were used in order to verify the capital cost determined by Bester *et al.* (2010). As Marchionni *et al.* (2015) cost-functions were developed based on Portuguese water infrastructure, the calculated cost (Euros) was converted into South African Rands (ZAR) using a Purchasing Power Parity conversion factor (World Bank, 2016). After adjusting the prices to represent the ZAR value at the same time period, it was found the cost determined by Marchionni *et al.* (2015) differed from Bester *et al.* (2010) by a maximum of 2%.
- Several recent tender documents (post 2013) for water services projects in South Africa.
- Several quotes (post 2015) from companies who specialise in the UV disinfection, water level logging and Real Time Control systems.
- Personal communication with professionals who work in the water service production and supply industry
- South Africa's Department of Cooperative Governance and Traditional Affair's (DoCOGTA) manual on Municipal Infrastructure – 'An industry guide to infrastructure service delivery levels and unit costs' (DoCOGTA, 2010).

As the capital costs collected for this study are from several different years (between 2013 and 2016), all costs were inflated to represent the effective value as of April 2016 using the contract price adjustment provisions (CPAP) work group indices (StatsSA, 2016). Additionally, a contingency cost of 10% was added to all capital costs.

4.1.9.2 Maintenance costs

To ensure that a stormwater harvesting system is successful, it is essential that it receives frequent maintenance. Regular maintenance activities include: litter and vegetation removal; pipe cleaning and corrective maintenance to repair damages or failures to the system. Regular and irregular maintenance costs as well as the inspection costs were established after consulting both the DoCOGTA (2010) and the *South African SuDS Guidelines* (Armitage *et al.*, 2013). In regards to corrective maintenance, Armitage *et al.* (2013) state "*The City of Cape Town spends approximately 10% of its annual maintenance budget on 'repairs";* thus, corrective maintenance costs were determined based on this rough recommendation. Lastly, the frequency of

maintenance required was determined based on the recommendations of Armitage *et al.* (2013) as well as from recommendations from suppliers.

4.1.9.3 Operation costs

Naturally, a stormwater harvesting system will incur operational costs during its lifespan. For this study, operational costs consisted predominantly of the energy costs required to run the system. To determine the energy costs, the power consumption of each component that required energy, was multiplied by the CoCT's electricity tariff (kWh) and the time for which the system was in operation (determined through model simulations). The power consumption of each component was obtained from its supplier. Additionally, a Global System for Mobile Communication (GSM) contract and hosting charge required to operate telemetric services was imposed on the Real Time Control (RTC) systems – similarly, this cost was obtained from the supplier.

As pumping accounts for a substantial portion of the energy consumption of a distribution system, using a poorly sized pump can inflate cost estimates (OIT *et al.*, 2001; Rautenbach, 2005). Currently, there are various pumps on the markets, so rather than limiting the study's results to a particular pump, it was decided that pump operating costs for this study would be estimated using an idealised pump. Further, based on the recommendation of Van Zyl (2016), the most efficient method to operate a pressurised reticulation system, such as the non-potable water reticulation networks designed in this study, would be to use a variable speed pump. Consequently, the following equation was used to determine the pump operating costs:

$$ZAR = \sum_{t=1}^{T} \frac{0.25\rho g Q_t h}{1000\eta} \times E \qquad 4\text{-}6$$

Where: ZAR = pump operating cost; T = total number of time-steps; t = time-step; ρ = density of water (kg/m^3); g = gravitational constant (m/s^2); Q = water demand of system per time-step (m^3/s); h = pumping head (m); η = efficiency factor; E = CoCT electricity tariff (ZAR/kWh).

The following conditions were imposed when calculating the pump operating cost:

- A fifteen-minute time-step was used.
- The required operating flow rate of the pump at each time-step was represented by the water demand of the entire system at the corresponding time-step.
- The pump was assumed to operate at 75% efficiency based on the recommendations of Van Zyl (2016), Bester *et al.* (2010), Marchi *et al.* (2012) and Georgescu *et al.* (2014).

- The required pumping head equalled the sum of the minimum required network static head, the estimated frictional head losses and maximum elevation difference between the pump and a serviced property.
- As a fifteen-minute time-step was used, each term was multiplied by a factor of 0.25 (15 minutes = 0.25 hours) (seen in Equation 4-6) to convert calculated power to kilo-watt hours (kWh).

4.2 Stormwater model construction

At the beginning of this research, it was envisioned that a previously developed computer based stormwater model of the Diep River subcatchment carried out on behalf of the CoCT that was acquired could be adapted for this study. It became apparent after scrutinising this model that it had several severe limitations. The model was supposedly calibrated against four significant historic events although this calibration was apparently not successful as the model's peak flows were not comparable to observed peak flows. Additional limitations of the model were: it did not model the subterranean stormwater network for much of the upper reaches of the catchment; there was considerable uncertainty over the accuracy of the invert levels since the majority of them were estimated using a 5 m contour map; and there were discrepancies in the model's spatial reference system. Due to these limitations, it was decided that it would be simpler to construct a new stormwater model from scratch and to refer to the previously developed model when necessary.

After investigating the applicability and availability of several of the stormwater modelling software listed in Section 2.5.1, it was decided that *PCSWMM 6.2* – the latest version of *PCSWMM* released at the time of the study – was the most appropriate to use for this study. *PCSWMM 6.2* makes use of the USEPA's internationally recognised and freely available hydraulic and hydrological software, Storm Water Management Model Version 5 (*SWMM 5*), as its simulation engine. *SWMM 5* enables the dynamic rainfall-runoff modelling for long term or single storm events. Furthermore, *PCSWMM 6.2* also includes a GIS interface as well as tools that streamline model development and help users to efficiently optimise and analyse their models (CHI, 2016).

4.2.1 Subcatchment delineation

The hydrological catchment – the Diep River subcatchment – that was under investigation for this study was delineated using *ArcGIS's* watershed tool. By specifying the common drainage outlet, this tool determines the entire contributing area upstream of the drainage outlet using a DTM – described in Section 4.1.1. Additionally, within this process, the watershed tool will identify the smaller contributing areas (subcatchments) from which the greater catchment area is composed (ESRI, 2016a). However, the contributing subcatchment areas that this tool delineated had to be manually adjusted to account for the drainage paths caused by existing infrastructure.

As a result of this process, the Diep River subcatchment was divided into 225 smaller subcatchments. The accuracy and number of subcatchments this process identified was considered reliable due to the high resolution of the DTM (0.5 m vertical and horizontal resolution) and manual inspection. Towards the upper reaches of the catchment, where development was less dense, these subcatchment ranged on average between 10 – 20 ha. In the more densely developed areas, the typical subcatchment size was between 1 – 12 ha.

4.2.2 Stormwater conveyance network

The Diep River subcatchment's stormwater conveyance network was modelled using GIS data obtained from the CoCT. This data provided a spatial representation of the stormwater network, detailing the network's geographical layout as well its topographical and geometric properties (e.g. cover levels, invert levels, pipe diameters). The geographical layout was regarded as sound as it met the layout criteria specified in CSIR (2005). However unfortunately, much of the data's topographical and geometric information was missing. Therefore, it was necessary to perform a data patching exercise in order to fill in the gaps in the data.

4.2.2.1 Manhole depths

After inspecting the stormwater network data, it was discovered that approximately half of the manholes' data were missing cover and invert levels. To overcome this, cover levels were estimated from the DTM for manholes that were inadequately identified. It was established that the datum of the CoCT data was MSL, like the DTM, by comparing the cover levels estimated using the DTM to the cover levels that were provided by the CoCT data set. Once all cover levels were confirmed, the missing invert levels were estimated by deducting 1.6 m from the manhole's respective cover level. The estimated inverts levels were adjusted using *PCSWMM 6.2* functions to ensure the conduits minimum slope requirements, as recommended by CSIR (2005), were met. If the diameter of the upstream and downstream conduits of the respective manhole were not known, the invert level was only adjusted after the pipe was sized using the process described in Section 4.2.2.2. Overall, adjusting the invert level proved to be an iterative process; however, by the end of this process the model was deemed to reasonably represent the actual cover and invert levels of the stormwater network.

4.2.2.2 Pipe sizing

This study decided only to model pipes with a diameter equal to or greater than 375 mm. However, like the manhole information, a significant portion of the CoCT data were missing – approximately 40% of pipes did not have an assigned diameter. It was therefore necessary to estimate these pipe diameters. This was performed using *PCSMM's* pipe sizing tool; this tool computes pipe diameter using the Manning's formula and the computed peak flow through each pipe. The pipes were sized to accommodate peak flows generated from a design storm with a

two-year recurrence interval, based on the design recommendation of CSIR (2005) for residential areas (the majority of catchment is residential area – see Chapter 3). The South African SCS 24-hour Type 2 rainfall distribution curve (SANRAL, 2013) with 24-hour rainfall depth for a design storm of a two-year recurrence interval was used. The total rainfall depth that was specific to the catchment (51.2 mm) was obtained from CoCT data.

After the pipes with missing data had been sized using *PCSWMM's* pipe sizing function, the network was then checked for the following:

- A pipe's diameter was equal to or greater than the diameters of its upstream contributing pipes.
- Each pipe's slope met the minimum slope criterion for its assigned diameter as specified by CSIR (2005).
- That all pipes had a sufficient cover depth as specified by CSIR (2005).
- The flow direction of all pipes was towards the catchment's drainage point.

4.2.2.3 Open channel section profiles

The GIS data obtained from the CoCT did not provide any information on the section details of the open channels – natural or constructed – that were part of the stormwater conveyance network. In order to acquire this information, a process which combined a desktop investigation and a field survey was performed. This process is summarised in Figure 4-9.

As part of the desktop investigation, open channels were categorised as either natural or constructed using an orthorectified 8 cm high resolution image. If the open channel was classified as natural, its section profile was determined from the DTM using *PCSMM's* 'transect creator' tool. To account for the variability of a natural channel's profile, a section profile was taken approximately every 100 m. The orthorectified 8 cm high resolution image was then also used to locate the position of culverts along open channels. Additionally, the various positions in which constructed open channels' section profiles changed were identified and recorded.

Following the completion of the desktop investigation, a field survey was performed. The field survey visited the positions that were recorded during the desktop investigation to determine culvert sizes and the section profiles of constructed open channels. Each of these elements was manually surveyed with the use of a laser range finder and measuring staff. The laser range finder was used to measure the width of the culverts and channels whilst the measuring staff was used to measure their depths. The laser range finder was not used to measure depths as water in the channel/culvert disrupted the measurement. Once all of the recorded positions had been surveyed, the measurements were collated and input into the stormwater conveyance model.

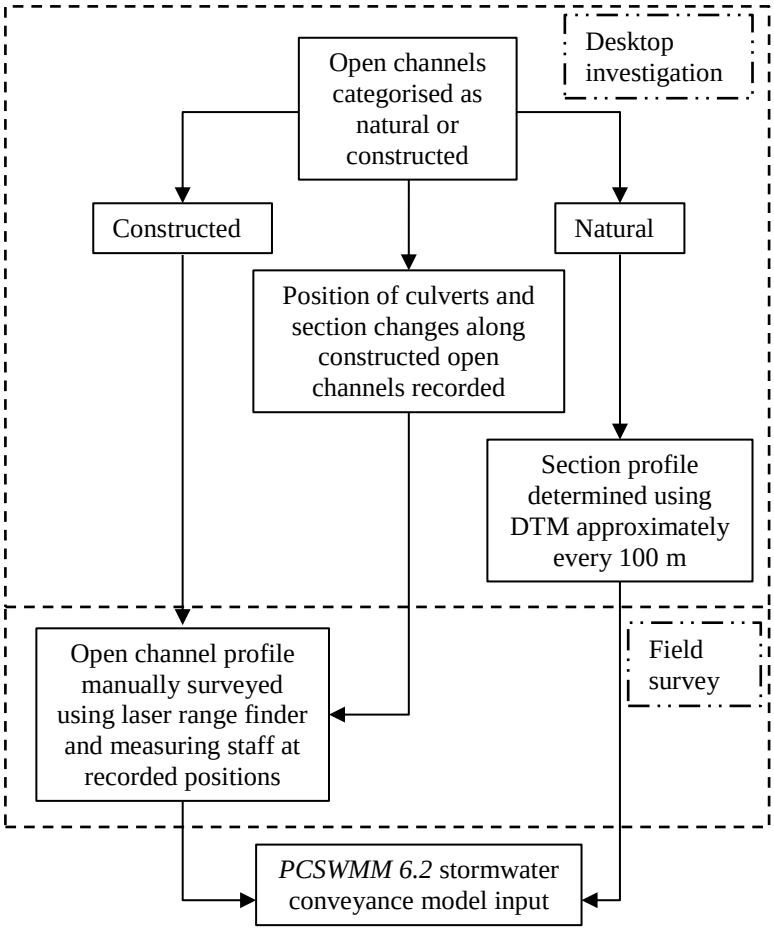

Figure 4-9: Process for determining open channel section profiles

4.2.3 Stormwater ponds

It was important to reliably model the Diep River subcatchment's stormwater ponds because of their significant role in this research. This meant information that detailed the geographical location, storage capacity, outlet structure and infiltration parameters of each pond was required. This information was derived from several sources which included: the DTM; reports on pond outlet structures; as-built drawings of each detention pond; and GIS data obtained from the CoCT that noted each pond's geographical location.

SWMM 5 uses an algorithm that requires a storage curve (depth versus area) to model storage capacity of a stormwater pond. As the Diep River subcatchment's stormwater conveyance network included both detention ponds and a retention pond, the storage curves of different types of ponds were determined in slightly different manners. In regards to detention ponds, the storage curves were generated in the following manner:

i) The maximum depth in each pond was determined from its respective as-built drawing.

ii) The storage curve was developed for the storage volume below the pond's maximum depth using the DTM. The storage curve used a depth interval of 0.5 m, at which each interval's corresponding area was determined.

iii) The storage curves for the detention ponds were considered accurate due to the DTM's high resolution. Furthermore, these storage curves were comparable to the storage curves for the same detention ponds that were previously developed for the model that was developed on behalf of the CoCT – Section 4.2.

Unfortunately, due to the limitations of topographical LiDAR data – described in Section 4.1.1 – it was not possible to determine the entire storage curve for the retention pond, *viz.* Little Princess Vlei. Therefore, the curve was derived in two parts according to the following method:

i) After inspecting the DTM, it was discovered that the water level in Little Princess Vlei was below the crest of the pond's outlet weir (level obtained from Element Consulting Engineers (2002)) at the time the LiDAR was captured.

ii) It was assumed that the Little Princess Vlei's pond floor was 2 m below the crest of pond's outlet weir. No information could be found on the average water depth in Little Princess Vlei and so the assumption was based on the average depth of neighbouring *vleis* – Princess Vlei and Zeekoevlei (Harding, 1992a, 1992b).

iii) Based on the information provided by the two aforementioned points, it was decided to represent the storage volume below the crest of the pond's outlet weir by a linear storage curve. The curve started from zero depth and zero area and rose to a maximum point at the depth of the crest of the pond's outlet weir (2 m) and its corresponding sectional area.

iv) The subsequent section of the storage curve represented the remaining storage volume of Little Princess Vlei (i.e. between the crest and maximum operational elevation of the Little Princess Vlei outlet weir). This section of the storage curve was derived from the DTM.

The outlet structures of the Diep River subcatchment's detention ponds were modelled according to the information provided by their as-built drawings. These as-built drawings were invaluable to this research as they provided the type of outlet structure and its dimensions. Unfortunately, the as-built drawings did not provide the discharge coefficients of the outlet structures. Therefore, these values were estimated from literature based on the type of outlet structure in each detention pond (Finnemore & Franzini, 2002; James *et al.*, 2010). In regards to Little Princess Vlei, its outlet structure was modelled by a flow formula that was obtained from Element Consulting Engineers (2002).

Each detention pond was modelled to account for infiltration using the Green-Ampt infiltration model. The infiltration parameters were determined from details on the boreholes that were drilled at each detention pond – described in Section 4.1.3.

4.2.4 Catchment parameter estimation

To model the hydrological processes of the Diep River subcatchment, it was necessary to approximate a number of catchment parameters. Whilst these parameters were subjected to calibration, the method in which their initial estimates were determined is described in Table 4-6.

Table 4-6: Catchment hydrological parameters

Parameter	Determined by
Subcatchment area	Subcatchment areas were determined through the watershed delineation process performed using *ArcGIS* – described in Section 4.2.1.
Subcatchment width	The width of a subcatchment was the quotient of its area and flow length. Each subcatchment's flow length was estimated through a combination of the stream link function in *ArcGIS* and manually adjusting the flow length to account for the sewer network. This value was adjusted through calibration.
Slope	Subcatchment slopes were determined from the DTM
Impervious area	The imperviousness of each subcatchment was determined using an area weighting tool that accounted for the subcatchment's land use composition. The typical imperviousness percentage of various land uses were established from James *et al.* (2010) and SRK Consulting (2012).
Manning's roughness coefficient for overland flow/stormwater conveyance network	Typical Manning's roughness coefficients for overland flow and various materials were acquired from James *et al.* (2010), and Finnemore & Franzini (2002). For overland flow, these values were assigned based on the subcatchment's land use composition using an area weighting tool. For the stormwater conveyance network, all constructed sewers/channels were assumed to be concrete whilst the material of natural channels was determined through inspection using the high resolution orthorectified image. Consequently, the appropriate Manning's roughness coefficient was assigned.
Depression storage	Depression storage was assigned using an area weighting tool that accounted for the subcatchment's land use composition. The typical depression storage for various land uses was established using James *et al.* (2010) and SRK Consulting (2012).
Subarea routing	Runoff from impervious areas was assumed to run onto pervious areas.
Percent of runoff routed between pervious and impervious area	This value was estimated based on the imperviousness of the subcatchment. The following values were used for different percentages of subcatchment imperviousness: $Si < 20\% = 50\%$ routed, $20\% > Si > 40\% = 35\%$ routed; $Si > 40\% = 20\%$ routed. Where Si = Percentage of subcatchment impervious.
Subcatchment outlet	The subcatchment outlet was assigned to the manhole with the lowest elevation in the subcatchment
Evaporation	Evaporation was modelled using the method described in Section 4.1.5. All open water was subjected to full evaporation potential (i.e. no cover reducing evaporation)
Rainfall	Subcatchments were assigned to their closest rainfall record. The rainfall records used for model simulation and calibration are described in Section 4.1.4.
Baseflow	Assumed to be negligible for this study – described in Section 4.1.6.

4.2.5 Calibration

As is evident from the preceding sections, modelling both the hydrological and hydraulic capacity of a catchment requires a substantial amount of parameter estimation. Appropriate parameters can be derived from data on the catchment's conditions; however, it is essential to calibrate these parameters to provide the most 'useful' representation of the catchment (Sangal *et al.*, 1994; Singhofen, 2001; James, 2005). It has long been recognised that it is important to calibrate stormwater models, and so a number of different techniques have developed in order to optimise the process. This study made use of *PCSWMM's* 'SRTC calibration tool' to assist the calibration process.

The 'SRTC calibration tool' is a tool provided by *PCSWMM 6.2* that streamlines the calibration process. It can be used to calibrate a model against observed data or to test parameter sensitivity. Based on uncertainty percentages given to each parameter, it completes two model runs for both the high and low values of the specified uncertainty range – this study used uncertainty percentages for each parameter recommended by James (2005). For each parameter tested, *PCSWMM 6.2* linearly interpolates between the two values. This enables each parameter to be calibrated as well as to check its sensitivity.

It should be noted that there were a number of limitations faced during the calibration of the stormwater model used in this study. These limitations were caused due to the following:

- There was limited flow data available. The only working flow gauge was positioned in the upper to middle reaches of the catchment (Figure 4-3) and it did not record 'dry' weather flows – Section 4.1.6. Therefore, only a portion of the Diep River subcatchment could be calibrated against observed data.
- The catchment's highly variable rainfall was only represented by two rainfall gauges (DIEP05AR and DIEP05ER – Figure 4-1). This was because only two of the collected rainfall records that provided continuous five-minute rainfall data were located in the gauged portion of the catchment.

As this study focussed on harvesting stormwater from existing stormwater ponds in one of the CoCT's catchments, there were several similarities between the calibration procedure performed by Fisher-Jeffes (2015) and the one required for this research. Therefore, it was decided the calibration procedure for this study should be consistent with the one performed by Fisher-Jeffes (2015) when possible. This procedure can be summarised by the following points:

- In order of importance, the calibration focused on total runoff, peak flows and visually representing hydrograph.
- 37 storm events were identified from the observed data. Two-thirds of these events (22 events) were used for calibration and one-third were used for verification (15 events).
- The parameters that were calibrated were: catchment width (width), percent of impervious area (Imperv.) 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), percent of impervious area with no depression storage (Zero Imperv.) percentage routed to pervious (Percent Routed); and Green and Ampt infiltration parameters.

- The parameters that were not calibrated were catchment slope and catchment area. Both of these parameters were accurately determined from the DTM using tools provided by *ArcGIS*. Also, the delineated subcatchment areas were manually inspected to ensure they accommodated the existing stormwater infrastructure.
- Calibrated parameters were validated to ensure they fell within an acceptable range defined by James (2005). However, for depression storage, the upper limit of this range was increased to accommodate interception storage. Interception storage can be significant for highly vegetated areas during large storm events and *SWMM 5* does not explicitly account for it (Guo & Adams, 1998).

A summary of the results of the calibration can be found in Table 4-7, Table 4-8 and Table 4-9. The runoff continuity errors (<0.7%) and routing continuity errors (<0.7%) were deemed to be acceptable.

Table 4-7: Calibration of total runoff

Error Function	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error	11.8	17.6
Integral Square Error rating	Fair	Fair
Nash Sutcliff Efficiency	0.81	0.748
R^2	0.83	0.768

Table 4-8: Calibration of peak flow

Error Function	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error	19	12
Integral Square Error rating	Fair	Fair
Nash Sutcliff Efficiency	0.764	0.606
R^2	0.825	0.909

Table 4-9: Calibration of hydrograph

Error Function	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error	0.715	1.87
Integral Square Error rating	Excellent	Excellent
Nash Sutcliff Efficiency	0.383	0.571
R^2	0.556	0.663

It is important to note that the aforementioned calibration results were only representative of the gauged portion of the Diep River subcatchment. After the parameters of the gauged portion were calibrated, the parameters belonging to the remaining portion of the catchment – the ungauged portion – still had to be adjusted. Before calibration, the parameters for the entire catchment were determined according to methods described in Section 4.2.4. Following calibration, only the parameters for the gauged portion of the catchment had changed. Thus, the percentage change in each parameter was determined and used to adjust the uncalibrated parameters accordingly (i.e. the parameters belonging to the ungauged portion of the catchment were scaled up/down according to the percentage change of the parameters in the gauged portion of the catchment). By the end of the calibration, it was deemed that the model provided a reasonable representation of the actual catchment conditions.

4.3 Constituents of the stormwater harvesting scenarios

This study devised sixteen different stormwater harvesting scenarios in order to assess whether stormwater could be harvested viably from the Diep River subcatchment's stormwater ponds. Initially, only fourteen scenarios were created, but after acquiring the results of these initial scenarios, a further two scenarios were developed by modifying two of the initial scenarios. The sixteen stormwater harvesting scenarios modelled various non-potable demands from residential and commercial properties as well as different storage and harvesting arrangements of the existing ponds. The scenarios also included RTC systems – systems that enable dynamic control of the operation facilities – and considered the systems used to distribute and treat harvested stormwater.

4.3.1 Stormwater harvesting scenarios

The following section describes the initial fourteen stormwater harvesting scenarios that were modelled in this study. As stated in Section 4.3 however, based on the results of these scenarios, an additional two scenarios were developed; these scenarios are described in Section 4.3.5. This section specifically describes the various non-potable demands and storage and harvesting arrangements of the stormwater ponds that the scenarios modelled.

4.3.1.1 Water Demand Alternatives

As stated in the Section 2.3.2, at present, the affordability of reliable treatment techniques is currently a hindrance to stormwater harvesting. Advance treatment and disinfection methods that can treat stormwater to a potable standard on-site are generally considered too expensive, whilst it is often too costly to pump stormwater to the nearest municipal water treatment works, as was the case in this study (Fisher-Jeffes, 2015). Both Wu *et al.* (2012) and Mitchell *et al.* (2006) have reported that there is greater public acceptance for using stormwater for non-contact uses than contact uses. Hence, this study focussed on harvesting stormwater to supply non-potable water demands. The study modelled four different non-potable demands that originated from various end-uses and property types. For ease of reference, the different types of non-potable demands modelled were referred to as water demand alternatives and are described in Table 4-10.

Table 4-10: Water Demand Alternatives

Water Demand Alternative	Type of end-use/s	Type of property end-use derived from
1	Toilet demands	Commercial properties
2	Toilet demands	Residential flat blocks
3	Toilet demands	Single residential properties (SRPs)
4	Toilet, clothes washing and irrigation demands	Single residential properties (SRPs)

The properties from which each water demand alternative extracted its end-use demands are described further by the following and are shown in Figure 4-10:

- **Water Demand Alternative 1** – represents toilet demands of two separate commercial centres that are located 'close' to Pond 4B and Little Princess Vlei, respectively.
- **Water Demand Alternative 2** – represents toilet demands of selected residential flat blocks that are located within the Diep River subcatchment.
- **Water Demand Alternatives 3 and 4 –** represent non-potable end-use demands of SRPs located in the immediate proximity of the harvesting storage facility.

As community acceptance is stated to be crucial to the success of a stormwater harvesting system (Wu *et al.*, 2012), it was decided that only end-uses that have been shown to have considerable public acceptance should be modelled. It has been shown in literature that there is a willingness to use stormwater to flush toilets. Since there are a variety of property types in the catchment whose toilet demands could be supplemented with harvested stormwater, toilet demands became the dominant non-potable end-use that was modelled in this research. Irrigation is another end-use that has excellent public acceptance; however, harvesting stormwater for irrigation purposes

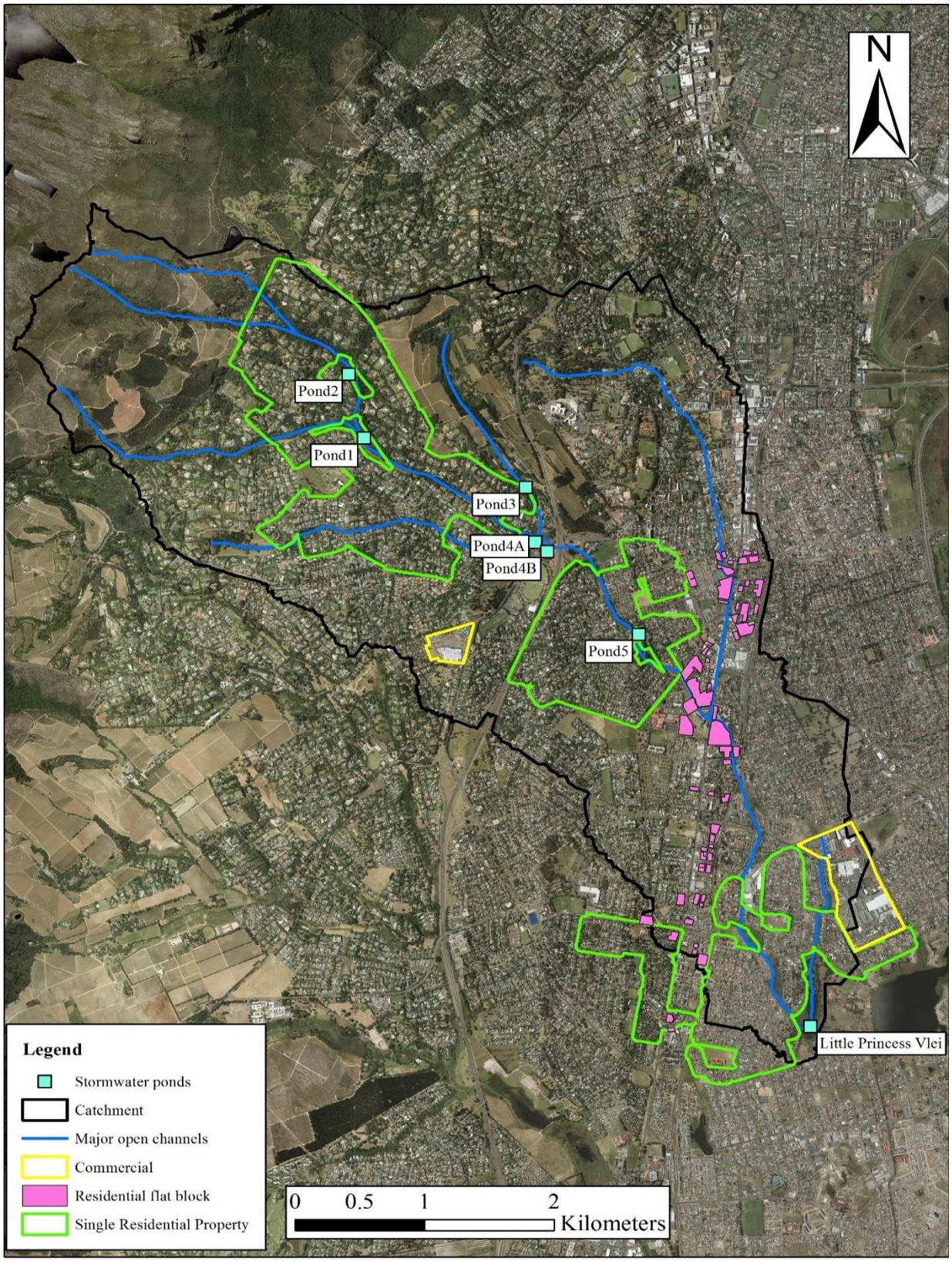

Figure 4-10: The properties that were serviced by the catchment's stormwater ponds in the stormwater harvesting scenarios

is typically ineffective in areas that experience seasonal rainfall such as the Diep River subcatchment and so it was considered much less than toilet demands in this study. It has also been shown in literature that the most economically viable stormwater harvesting systems occur in areas with spatially condensed, high water demands. So for this reason, it seemed pertinent to include a water demand alternative which resulted in high, concentrated demands. This led to the formation of Water Demand Alternative 4 that considered irrigation and clothes washing water demands along with toilet demands.

4.3.1.2 Storage configurations

Due to space constraints in urban areas, storage requirements are often a common concern to developers when implementing a stormwater harvesting system (Fletcher *et al.*, 2007). Brodie (2008) however, demonstrated that using dual storage systems in a stormwater harvesting system is a viable method to optimise storage volumes in areas where storage availability is limited. Since the Diep River subcatchment already has a number of potential storage facilities in its stormwater ponds, it was decided to investigate various storage and harvesting arrangements of the ponds to discover which would lead to the most viable stormwater harvesting system. For ease of reference, these arrangements will be referred to as storage configurations.

Table 4-11: Pond storage capacity and other attributes

Storage facility	Little Princess Vlei	Pond 1	Pond 2	Pond 3	Pond 4B	Pond 5
Pond type (existing)	Retention	Detention	Detention	Detention	Detention	Detention
On-line/off-line (existing)	On-line	On-line	On-line	On-line	Off-line	Off-line
Maximum pond depth (m) (existing)	3.72	6.5	8.1	4.4	4.25	2.3
Harvestable volume (Mℓ) (modelled)	22.3	18.5	11.9	4.7	8.9	3.8
Dead storage depth (m) (modelled)	1.2	3.3	1.8	1.8	1.2	1.2
Active storage depth (m) (modelled)	2	5.5	6.6	3	2.9	1.8

It is important to note the following in regards to the Diep River subcatchment's stormwater ponds whose location and capacity can be seen Figure 4-10 and Table 4-11, respectively:

- All ponds are unlined open storage facilities meaning that they are susceptible to infiltration, evaporation and incident precipitation.
- The harvestable volume for each pond is equal to the total storage volume minus both the flood mitigation and dead storage volumes.

- In order to prevent the growth of reeds and algal blooms as well as to uphold the aesthetics of the pond, the minimum dead storage depth was set at 1.2 m above the pond floor (Woods-Ballard *et al.*, 2007; CoCT, 2013). If a river passes through the pond (i.e. Pond 1, 2 and 3), the dead storage depth was set equal to 1.2 m plus the depth of the river section passing through the pond (Figure 4-11). The depth of the river section passing through the pond was measured during a site inspection.
- The maximum depth of each pond was determined from as-built drawings obtained from the CoCT.
- It was assumed all detention ponds could be converted into retention ponds by installing outlet control.

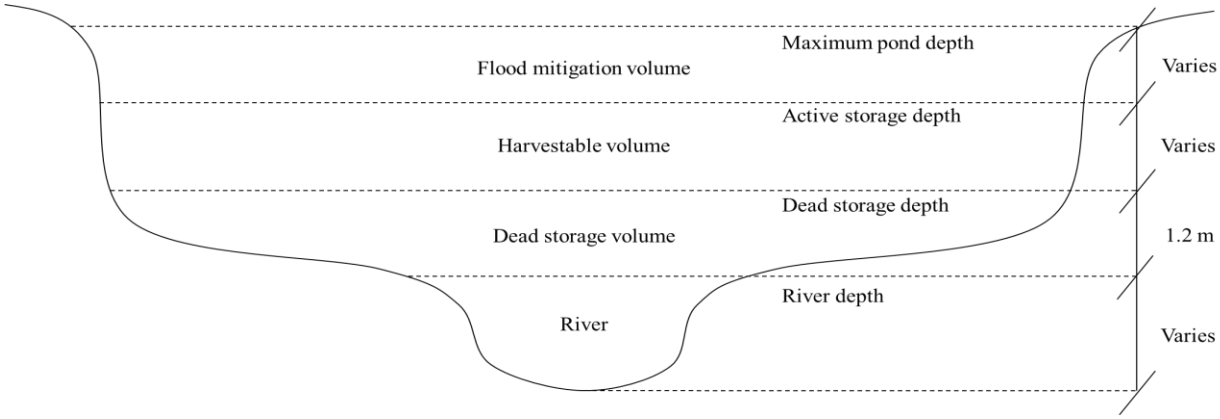

Figure 4-11: Cross-sectional representation of the various depths in the modelled stormwater ponds

Different storage configurations were achieved by changing the role that each pond performed within the stormwater harvesting system. The different roles that a pond could serve are described as follows:

- **Storage and harvesting of runoff –** In addition to capturing and storing stormwater, stormwater is also extracted directly from the pond. The pond is connected to distribution infrastructure.
- **Storage of runoff** – In addition to capturing and storing stormwater, the pond serves as a 'balancing storage pond'. When required, the pond will release captured stormwater to a downstream pond, where it can be distributed to users of the system. No stormwater is directly extracted from the pond and it is not connected to distribution infrastructure.
- **Not used for stormwater harvesting –** The pond will not serve any function within a stormwater harvesting system. It will maintain its current functionality, which is to attenuate peak flows.

Assuming that Little Princess Vlei, the most downstream pond in the catchment, was always to be used for the storage and harvesting of runoff, there are 243 different possible storage configurations that could be devised from the combination of pond roles and the existing stormwater ponds. This study however, only modelled four storage configurations (Storage Configuration A, B, C and D). Many of these storage configurations, however, share considerable similarities to one another. Modelling all of them was thus unnecessary as long as the most representative ones were considered. The choices behind which of the 243 possible storage configurations were modelled are described by the following:

i) Storage Configuration A assumed only harvesting stormwater from Little Princess Vlei without making use of upstream storage ponds, thus limiting the potential harvestable volume of stormwater, but also limiting cost implications. It was developed as it required the smallest financial investment. By modelling this storage configuration, it was possible to assess if other storage configurations that maximised the available storage capacity in the catchment were worth the increased financial investment.

ii) Storage Configuration B was developed in contrast to Storage Configuration A to assess the viability of harvesting stormwater from every pond. This storage configuration would investigate if the benefits of maximising the storage capacity in the catchment would outweigh the significantly greater financial investment.

iii) Both Storage Configuration C and Storage Configuration D were developed to investigate the effect of largely using upstream ponds for storage. Using upstream ponds to store stormwater provides the opportunity to maximise the storage capacity within the catchment whilst limiting cost implications (i.e. pumping and treatment costs). Further, Storage Configuration C was developed to determine if it is more viable to make use of upstream storage and simultaneously harvest from two ponds, creating smaller separate water distribution networks. Storage Configuration D was developed to determine if it is most viable to maximise upstream storage with a single extensive water distribution network.

It was decided that no modifications would be made to the existing ponds that altered their storage capacity. One of the aims of this study was to investigate if it is possible to optimise a stormwater harvesting system's storage capacity by altering its storage configuration. Additionally, it was decided not to operate Little Princess Vlei's outlet using RTC. The average water level in Little Princess was 7.78 m above MSL whilst the lowest point on any property neighbouring Little Princess Vlei was 9.40 m above MSL. Currently, Little Princess Vlei's water level increases to approximately 8.50 m during a ten-year flood event. Thus, taking these factors into consideration, it was decided that increasing the water level in Little Princess Vlei by operating its outlet using rudimentary RTC rules such as those used in this study (explained in Section 4.3.2) may lead to potential flooding issues. The role that each pond in the catchment performed in each of the storage configurations (Storage Configuration A, B, C and D) that were modelled in this study is depicted in Figure 4-12.

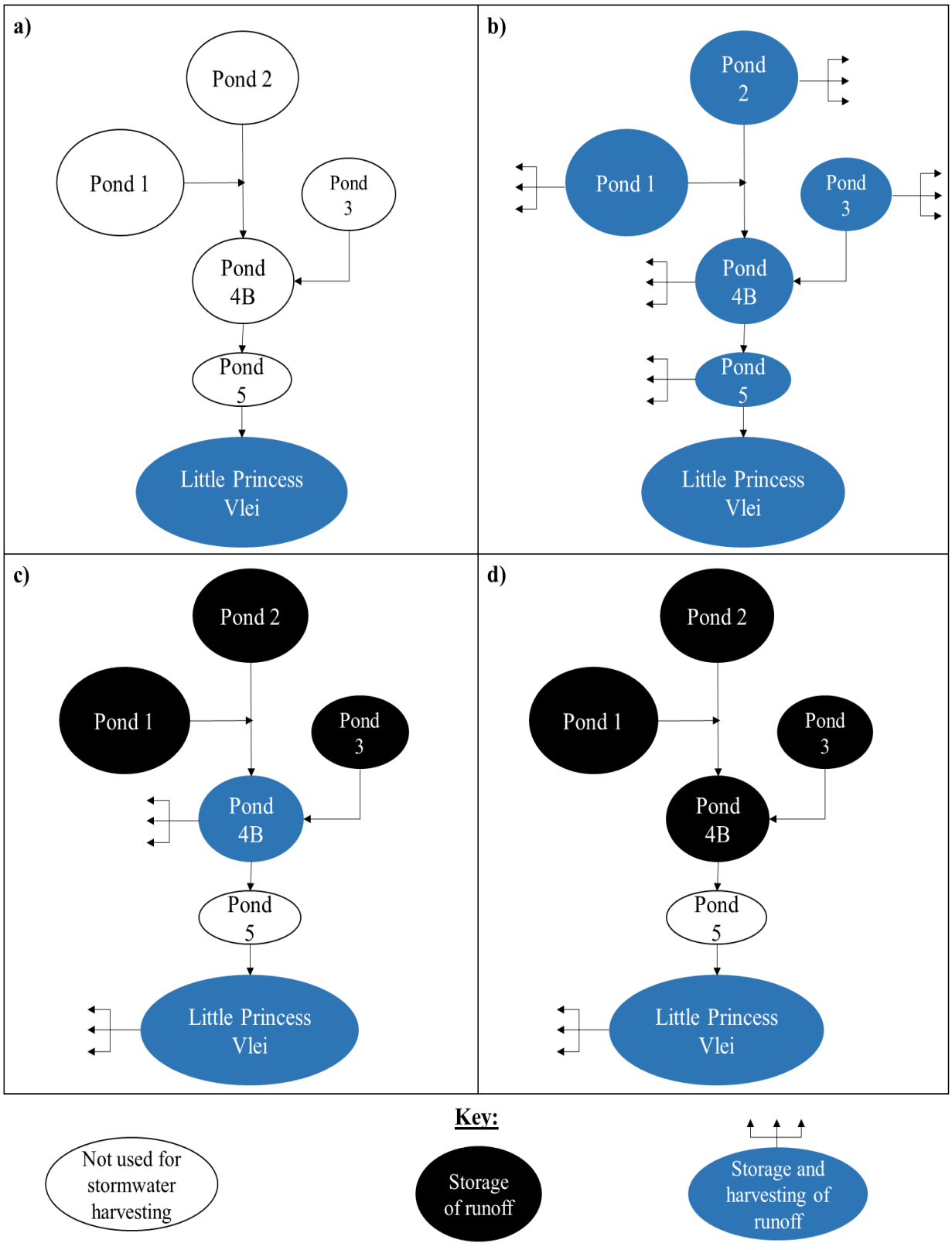

Figure 4-12: Schematic of storage configuration; a) Storage Configuration A; b) Storage Configuration B; c) Storage Configuration C; d) Storage Configuration D

4.3.1.3 Modelled stormwater harvesting scenarios

The initial fourteen stormwater harvesting scenarios that were modelled in this study were each assigned a water demand alternative (Section 4.3.1.1) and a storage configuration (Section 4.3.1.2). The combinations are listed in Table 4-12a and in Table 4-12b. It should be noted that no scenario was developed by combining Storage Configuration B with either Water Demand Alternative 1 or Water Demand Alternative 2. This was because there is no commercial centre or residential flat block within the immediate vicinity of each stormwater pond in the Diep River subcatchment. Furthermore, the two additional scenarios that were developed based on the results of the initial fourteen scenarios are not described in Table 4-12a or Table 4-12b, but in Section 4.3.5.

Table 4-12a: Stormwater harvesting scenarios and their composition

Stormwater harvesting scenario	Scenario composition:		Description of stormwater harvesting scenario
	Storage Configuration	Water Demand Alternative	
Scenario A1	A	1	• Harvesting only occurs at LPV* • Does not make use of upstream storage • Supplies toilet demands to the commercial properties located in close vicinity to LPV
Scenario A2	A	2	• Harvesting only occurs at LPV • Does not make use of upstream storage • Supplies toilet demands to residential flat blocks
Scenario A3	A	3	• Harvesting only occurs at LPV • Does not make use of upstream storage • Supplies toilet demand to SRPs
Scenario A4	A	4	• Harvesting only occurs at LPV • Does not make use of upstream storage • Supplies toilet, clothes washing and irrigation demands to SRPs
Scenario B3	B	3	• Harvesting occurs at all ponds • Does not make use of upstream storage • Supplies toilet demand to SRPs
Scenario B4	B	4	• Harvesting occurs at all ponds • Does not make use of upstream storage • Supplies toilet, clothes washing and irrigation demands to SRPs

*Little Princess Vlei (LPV)

Table 4-12b: Stormwater harvesting scenarios and their composition

Stormwater harvesting scenario	Scenario composition:		Description of stormwater harvesting scenario
	Storage Configuration	Water Demand Alternative	
Scenario C1	C	1	• Harvesting only occurs at LPV and Pond 4B • Pond 1, Pond 2 and Pond 3 are used for upstream storage • Supplies toilet demands to the commercial properties located in close vicinity to LPV and Pond 4B
Scenario C2	C	2	• Harvesting only occurs at LPV and Pond 4B • Pond 1, Pond 2 and Pond 3 are used for upstream storage • Supplies toilet demands to residential flat blocks
Scenario C3	C	3	• Harvesting only occurs at LPV and Pond 4B • Pond 1, Pond 2 and Pond 3 are used for upstream storage • Supplies toilet demands to SRPs
Scenario C4	C	4	• Harvesting only occurs at LPV and Pond 4B • Pond 1, Pond 2 and Pond 3 are used for upstream storage • Supplies to toilet, clothes washing and irrigation demands to SRPs
Scenario D1	D	1	• Harvesting only occurs at LPV • Pond 1, Pond 2, Pond 3 and Pond 4B are used for upstream storage • Supplies toilet demands to commercial properties located in close vicinity to LPV and Pond 4B
Scenario D2	D	2	• Harvesting only occurs at LPV • Pond 1, Pond 2, Pond 3 and Pond 4B are used for upstream storage • Supplies toilet demands to residential flat blocks
Scenario D3	D	3	• Harvesting only occurs at LPV • Pond 1, Pond 2, Pond 3 and Pond 4B are used for upstream storage • Supplies toilet demands to SRPs
Scenario D4	D	4	• Harvesting only occurs at LPV • Pond 1, Pond 2, Pond 3 and Pond 4B are used for upstream storage • Supplies toilet, clothes washing and irrigation demands of SRPs

*Little Princess Vlei (LPV)

Mitchell *et al.* (2006) stated that it is acceptable for a stormwater harvesting system to provide a less reliable level of service than primary water supply networks. A less reliable level of service (i.e. poor volumetric reliability) however, can simply be a result of over designing the serviceable

demand capability of a stormwater harvesting system (i.e. attempting to service too many properties). Thus, this would unnecessarily increase the cost of the required water distribution infrastructure. Unlike residential flat blocks and commercial centres, there are an abundance of SRPs situated in the Diep River subcatchment. However, it was clear that the ponds did not have the storage capacity to meet the demands of all SRPs in the catchment. It was therefore decided to limit the number of SRPs whose demand would be supplied by each scenario to ensure all scenarios achieved at least 70% volumetric reliability – the minimum level of service a stormwater harvesting system should be able to provide (Mitchell *et al.*, 2006).

The number of SRPs that were serviced in each scenario was based on the demand that could be sustained by the harvestable volume (i.e. effective active storage volume) of its harvesting pond/s in between major storm events that would fully replenish this volume ('dry' periods). For scenarios that made use of upstream storage ponds, the harvestable volume of the scenario's harvesting pond/s was increased by the volume of active storage of each upstream pond that would be used for storage. Some initial modelling of the catchment indicated that a 'dry' period, on average, lasted approximately 106 days. The total number of SRPs considered in each scenario was approximated according the following process:

i) Each harvesting pond was assigned to the suburb in which it was located.

ii) The total water demand of a SRP during the average length of a 'dry' period was then determined using Equation 4-7.

$$TWD_{\text{'dry' period}} \approx DWD_{\text{Pond's suburb}} \times \text{Average 'dry' period length} \qquad 4\text{-}7$$

Where: $TWD_{'dry'\ period}$ = total water demand of a SRP during average 'dry' period length; $DWD_{Pond's\ \text{suburb}}$ = daily water demand of a SRP in the harvesting pond's assigned suburb (Section 4.1.8).

iii) Using the result of Equation 4-7, the number of SRPs a harvesting pond can supply during the average 'dry' period length was determined using Equation 4-8.

$$No.\ of\ SRPs \approx \frac{V_{\text{harvesting pond}}}{TWD_{\text{'dry' period}}} \qquad 4\text{-}8$$

Where: *No. of SRPs* = number of SRPs a harvesting pond can supply during average 'dry' period; V = harvestable volume; $TWD_{'dry'\ period}$ = total water demand of a SRP during average 'dry' period.

iv) Using the result of Equation 4-8, the total number of SRPs supplied in a scenario was determined using Equation 4-9.

$$(Total\ no.\ of\ SRPs)_{scenario} \approx \sum \text{SRPs supplied by scenario's harvesting pond}/s \quad \text{4-9}$$

Where: *Total no. of SRPs* = total number of SRPs considered in a scenario.

Based on this method of approximation, the number of SRPs that could be serviced by each of the residential scenarios (i.e. Scenario A3, A4, B3, B4, C3, C4, D3 and D4) ranged between 277 and 1685 properties.

4.3.2 Real Time Control rules

Introducing Real Time Control (RTC) – described in Section 2.5.2 – into urban drainage systems is becoming an increasingly popular method to optimise their performance. By incorporating dynamic controls that respond to online field measurements, RTC can limit the redundancy imposed on urban drainage systems by pre-designed static controls (US EPA, 2006; Vallabhaneni & Speer, 2011; Gaborit *et al.*, 2012; Beeneken *et al.*, 2013; Muschalla *et al.*, 2014; García *et al.*, 2015). Hence, the ability of RTC to manipulate flow through an outlet structure would allow the existing detention ponds in the Diep River subcatchment to potentially retain runoff long enough for it to be harvested. This ability was key to the formulation of the storage configurations described in Section 4.3.1.2.

RTC provided the opportunity to create different storage configurations by controlling how the outlet structures of different ponds operated. This enabled the study to investigate the viability of increasing effective storage volumes by maximising the available storage throughout the entire catchment. Different storage configurations were created to determine the most viable manner in which this could be performed. It was assumed that operating RTC rules of low complexity could adequately answer this research question and that using RTC rules of a high complexity would only provide a more optimal stormwater system, but ultimately lead to the same answer. As this study was looking into viability of using RTC for stormwater harvesting, it was not necessary to find the most optimal solution. For this reason, the RTC systems used in this study were rudimentary including local automatic controls (LAC) and/or regional controls (RC) – explained in Section 2.4.2. Each storage configuration made use of a different RTC system which are described in the following sections.

4.3.2.1 Storage Configuration A

Figure 4-13 provides a schematic of the RTC system employed in Storage Configuration A. This RTC system is described as follows:

- The only pump in the system, situated at Little Princess Vlei, operates under a LAC. The LAC logic is as follows: during periods in which there is a demand for harvested stormwater, if the water level in Little Princess Vlei is above the dead storage depth

(1.2 m), the pump is turned on and it extracts water from the pond. If there is no demand for harvested stormwater or the water level in Little Princess Vlei is below the dead storage depth, the pump is turned off.

- No outlet structure in the system is controlled using RTC.

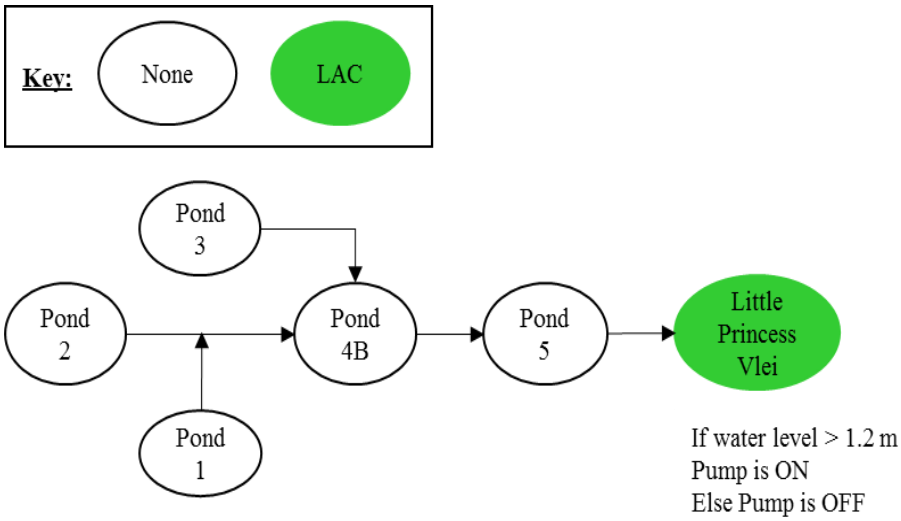

Figure 4-13: Schematic of RTC system used for Storage Configuration A

4.3.2.2 Storage Configuration B

Figure 4-14 provides a schematic of the RTC system employed in Storage Configuration B. This RTC system is described as follows:

- Water may be pumped out of every pond in the catchment. Each pond has its own pump, which is operated, independently of the other ponds' pumps in the system, based on the local conditions of the pond it serves (i.e. LAC). The LAC logic used to control each pump is as follows: if there is a demand for harvested stormwater and there is water available above the dead storage level, the pump is turned on and water is extracted. If there is no demand for harvested stormwater or the water level in a pond is below the pond's dead storage depth (Figure 4-14), the pump is turned off and water cannot be extracted.
- LACs are used to control the outlet structures of Ponds 1, 2, 3, 4B and 5. The LAC logic is as follows: the outlet is only open when the water level exceeds the active storage depth (Figure 4-14) in its respective pond; otherwise, the outlets remain closed.

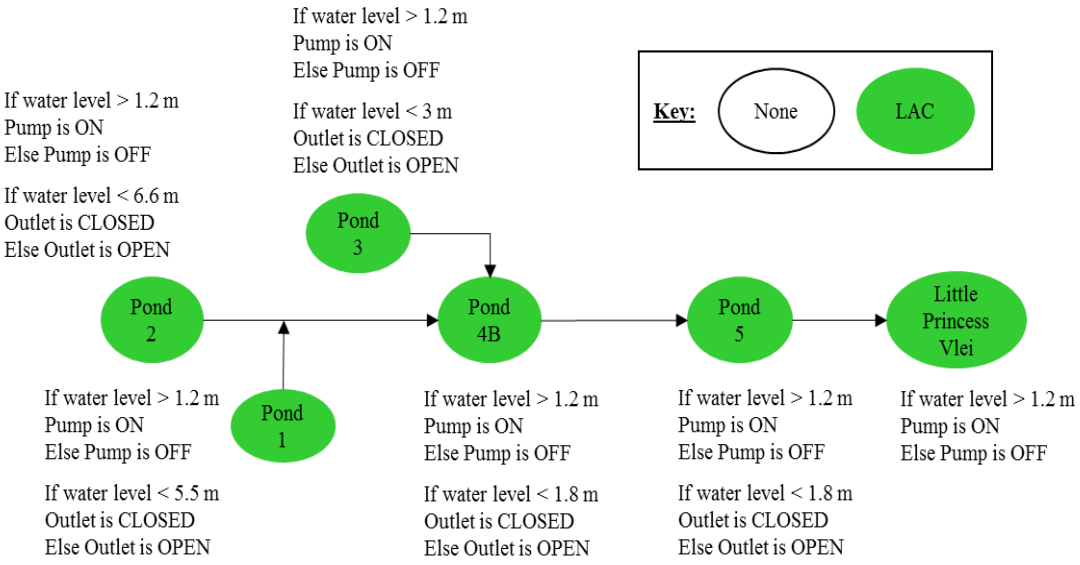

Figure 4-14: Schematic of RTC system used for Storage Configuration B

4.3.2.3 Storage Configuration C

Figure 4-15 provides a schematic of the RTC system employed in Storage Configuration C. This RTC system is described as follows:

- The outlet structure of Ponds 1, 2 and 3 are controlled using both LAC and RC. The outlets of these ponds are individually opened if the water level of the respective pond exceeds its active storage depth (Figure 4-15) (LAC). The outlets of Pond 1, 2, and 3 are also opened if the water level in Pond 4B falls below its dead storage depth (1.2 m) whilst the water of the respective pond exceeds its dead storage depth (RC). The outlets remain closed under all other operating conditions.
- The outlet structure of Pond 4B is controlled using a LAC. The LAC logic is as follows: the outlet is only open when the water level in Pond 4B exceeds its active storage depth (3.1 m), otherwise the outlet remains closed.
- There are two pumps in the system, situated at Little Princess Vlei and Pond 4B, that both operate under a LAC. The LAC logic used to control both pumps is as follows: during periods in which there is a demand for harvested stormwater, if the water level at a pump's respective pond exceeds the dead storage depth (1.2 m), the pump is turned on and water is extracted from the respective pond. If there is no demand for harvested stormwater or the water level in the respective pond is below the dead storage depth, the respective pump is turned off and water cannot be extracted from the pond.

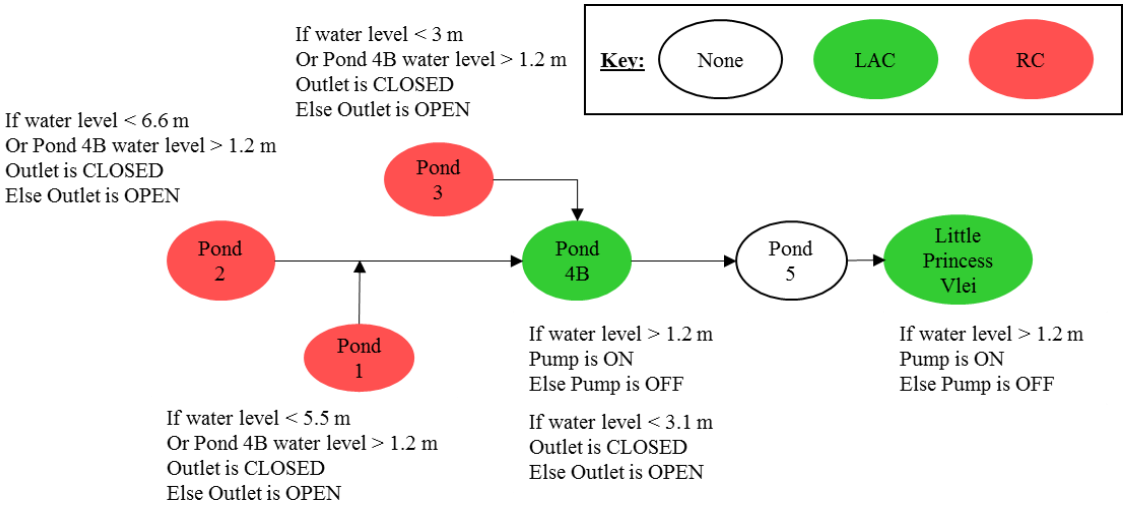

Figure 4-15: Schematic of RTC system used for Storage Configuration C

4.3.2.4 Storage Configuration D

Figure 4-16 provides a schematic of the RTC system employed in Storage Configuration D. This RTC system is described as follows:

- LACs and RCs are used to control the outlet structures of Ponds 1, 2, 3 and 4B.
- The outlet structure of Ponds 1, 2 and 3 are controlled using both LAC and RC. The outlets of these ponds are individually opened if the water level of the respective pond exceeds its active storage depth (Figure 4-16) (LAC). The outlets of Pond 1, 2 and 3 are also opened if the water level in Pond 4B falls below its dead storage depth (1.2 m) whilst the water level of the respective pond exceeds its dead storage depth (1.2 m) (RC). The outlets remain closed under all other operating conditions.
- The outlet structure of Pond 4B is controlled using both LAC and RC. If the water level in Pond 4B exceeds its active storage depth (3.1 m), its outlet is opened (LAC). The outlet of Pond 4B is also opened if the water level in Little Princess Vlei falls below the dead storage depth whilst the water level in Pond 4B exceeds its dead storage depth (1.2 m) (RC). The outlet remains closed under all other operating conditions.
- The only pump in the system is situated at Little Princess Vlei and is operated under a LAC. The LAC logic is as follows: during periods in which there is a demand for harvested stormwater, if the water level in Little Princess Vlei is above the dead storage depth (1.2 m), the pump is turned on and water is extracted from the pond. If there is no demand for harvested stormwater or the water level in Little Princess Vlei is below the dead storage depth, the pump is turned off and water cannot be extracted from the pond.

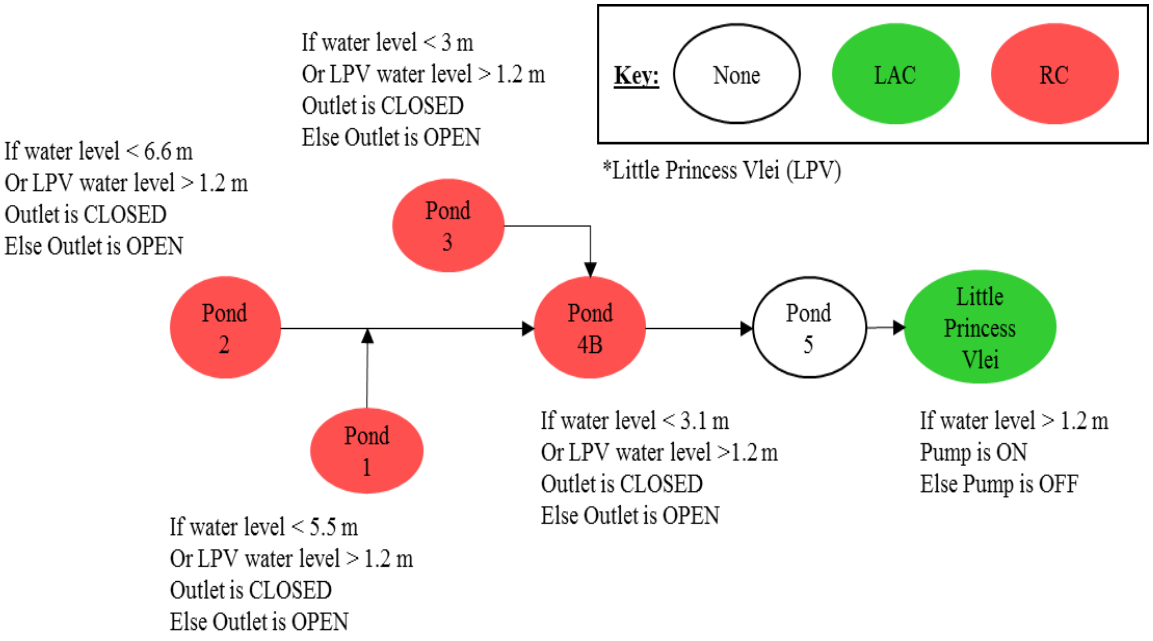

Figure 4-16: Schematic of RTC system used for Storage Configuration D

The actual RTC rules that were applied for each storage configuration can be seen in Appendix E.

4.3.3 Dual reticulation networks

Stormwater harvesting systems typically make use of a secondary reticulation network to supply non-potable water demands to multiple users (i.e. neighbourhood sized schemes); thus, creating a dual reticulation network – described in Section 2.3.4. Secondary reticulation networks however, can incur a significant proportion of the total financial cost of a stormwater harvesting system, and so the viability of a stormwater harvesting system is sensitive to the cost implications of this infrastructural investment (Armitage *et al.*, 2014; Fisher-Jeffes, 2015). For this reason, it was necessary to determine approximate cost estimates of each scenario's distribution network. Swamee & Sharma (2008) state that a cost-efficient water reticulation network is designed through balancing the network costs and its reliability. With this in mind, the non-potable water distribution network for each scenario was individually designed using *EPANET 2.0* – a water distribution modelling software package developed by US EPA (US EPA, 2017).

The secondary non-potable water reticulation networks were designed to meet the design criteria specified in CSIR (2005) – the accepted design guidelines for water distribution systems in South Africa. However, the following exceptions were made to the recommended design criteria:

i) The networks were not designed for fire flow conditions. It was assumed the additional network capacity required to handle fire flow was accounted for by the primary (potable) water supply network.

ii) The networks were only able to provide a minimum residual head of 15 m during peak demand – chosen because this is the accepted minimum residual head normally allowed in the potable water supply network (during fire flow conditions, i.e. a worst case scenario) (CSIR, 2005). Since, the network would only supply non-potable water demands, it was deemed acceptable to lower the level of service in favour of the financial benefits it produced.

The total daily water demand that was used to design each scenario's network was derived according to the water demand alternative that the scenario incorporated and the properties that the scenario serviced. This demand was then multiplied by an instantaneous peak factor (IPF) recommended by CSIR (2005) to model peak demand conditions. The networks were then optimised using an iterative process to generate the most cost efficient design that could meet the demand.

As stated in Section 4.1.9.3, the non-potable water reticulation networks were designed so treated stormwater would be pumped directly into the network creating a pressurised system. It was decided to use a pressurised system, as opposed to a gravity-fed system, as pressurised systems allow for greater operational flexibility and are easier to extend. Most importantly though, pressurised systems can pump water directly from source, thus, not requiring an elevated service reservoir (Trifunovic, 2006). Whilst the networks would be pressurised using a variable speed pump (described in Section 4.1.9.3), to model this condition, an idealised pump curve was generated by *EPANET 2.0* based on the pump's required operating point. The operating point for each pump's system was determined for peak flow conditions according to the following method:

- The pump's operating point flow rate was equal to the total daily water demand of the respective scenario multiplied by the IPF.
- The pump's operating head was the summation of the following terms: the minimum required residual head, frictional head losses in the network and the maximum elevation difference of a serviced property and the pump elevation.

4.3.4 Treatment of harvested stormwater

As stated in Section 4.1.7, the low resolution of the acquired water quality data meant that it was merely used to determine the degree of treatment that harvested stormwater required. After analysing this data, it was apparent that the Diep River contained highly variable levels of Total Suspended Solids (TSS) (18.9 and 31.4 mg/ℓ, mean and standard deviation, respectively) and *E.coli* (18 629 and 110 994 count/100mℓ, mean and standard deviation, respectively), that on average, exceeded the allowable standards for non-potable domestic, recreational and industrial

uses at all monitoring stations (DWAF, 1996a, 1996b, 1996c, 1996d, 1996e). However, other water quality parameters, listed in Section 4.1.7, were within an acceptable standard – further detail on the water quality is provided in Appendix G. Although TSS removal would occur due to the extended retention time of runoff in the stormwater ponds (Papa *et al.*, 1999; Shammaa *et al.*, 2002; Gaborit *et al.*, 2012), the high levels of *E.coli* suggested that the harvested stormwater would require treatment before distribution.

The *E.coli* concentration dictated the treatment that harvested stormwater required before distribution. It was decided that harvested stormwater would be best disinfected using a UV treatment system to ensure the *E.coli* concentration was reduced to an acceptable standard for non-potable water. Chlorination, a method of disinfection, was avoided as chlorinated water can have harmful effects on plants if the water is used for irrigation purposes. Since the required dose of UV disinfection is dependent on the UV transmittance (UVT) of the water, the harvested stormwater would require filtration beforehand to ensure a consistent quality (Fisher-Jeffes, 2015). The inclusion of slow sand filters – a low cost treatment device requiring relatively low-levels of maintenance that can tolerate wide fluctuations in flow range whilst still effectively removing organic matter and turbidity (Visscher, 1990; Papa *et al.*, 1999) – was assumed for this purpose.

Figure 4-17: A CoCT sign warning of polluted water in Little Princess Vlei

The same method for the post-treatment of harvested stormwater (i.e. a slow sand filter and a UV treatment system) was incorporated in all of the stormwater harvesting scenarios. However, the required size of each treatment device varied per scenario since different scenarios supplied different demands. For costing purposes, the slow sand filter and disinfection systems were designed for the peak hourly water demand of the pond and scenario in question. All costs for sand filters and UV disinfection equipment were obtained from recent tenders or local suppliers respectively. Figure 4-18 provides a schematic of the treatment process used within each stormwater harvesting scenario.

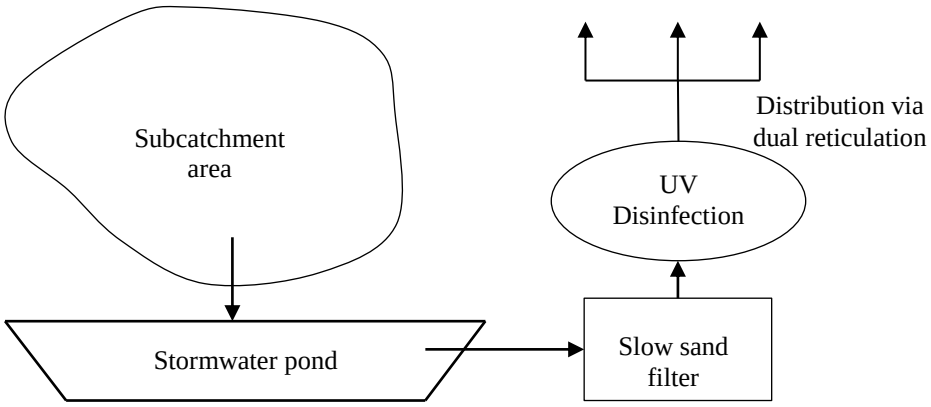

Figure 4-18: Proposed treatment system for harvested stormwater

4.3.5 Flood mitigation assessment

The CoCT (1986) states that the stormwater ponds along the Diep River were introduced to protect downstream areas of the catchment from flood events of up to 1-in- 20 year magnitude. Degradation of this flood mitigation ability due to stormwater harvesting would likely mean that additional flood mitigation measures would be required – thereby reducing the economic viability of using the ponds to harvest stormwater. Thus, the impact that each stormwater harvesting scenario had on the existing pond system's flood mitigation ability was assessed by comparing the percentage of storm events attenuated as well as the average reduction in peak flow that is provided by the pond system when used for stormwater harvesting to what is currently provided for the storm events that occurred within the ten-year period of analysis. The storm events were classified by comparing them to CoCT Intensity-Duration-Frequency (IDF) curves for the catchment. Unfortunately, the continuous ten-year rainfall record under which the stormwater harvesting scenarios would be modelled did not include a 1-in-20 year event. It was therefore decided to model the stormwater harvesting scenarios and existing system under worst case conditions – i.e. full active storage volume in all ponds and saturated soil conditions – for a design storm with a twenty-year return period. The South African SCS 24-hour Type 2 rainfall

distribution curve was used (SANRAL, 2013); the total rainfall depth for this storm that was specific to the catchment (110.1 mm) was obtained from CoCT data.

4.3.6 Modified stormwater harvesting scenarios

Two additional stormwater harvesting scenarios were developed after obtaining the results of the initial fourteen scenarios that were described in Section 4.3.1.3. The results of the initial scenarios, which are described in detail in Chapter 5, showed that the majority of runoff entering Little Princess Vlei was being released. The initial stormwater harvesting scenarios did not increase Little Princess Vlei's active storage volume by operating its outlet weir using RTC due to concerns it may increase the potential flood risk of properties neighbouring the pond. It was assumed therefore, that incorporating RTC at Little Princess Vlei's outlet would allow it to yield a substantially greater volume of stormwater. Since individual properties have a limited demand however, stormwater harvesting systems that have a greater volumetric yield naturally may be able to service a greater number of properties. This results in a more extensive distribution network, which equates to a larger cost investment. As a result, harvesting a greater volume of stormwater may not necessarily increase it economic viability.

Despite potential flooding concerns, it was decided to investigate if increasing Little Princess Vlei's active storage volume would lead to a more economically viable stormwater harvesting system. Two modified scenarios were created by increasing the active storage volume in Little Princes Vlei. The additional scenarios modified the two initial scenarios that produced the lowest unit cost of harvested stormwater – Scenario A4 and Scenario D4 (described in Section 5.1.1). The two modified scenarios modelled the same non-potable end-uses and used the same storage configuration as the initial scenarios from which they were created. These scenarios modelled toilet, clothes washing and irrigation demands of SRPs. Whilst both scenarios only harvested stormwater from Little Princess Vlei, Scenario D4 made use of upstream storage whilst Scenario A4 did not. The two modified scenarios were referred to as Scenario A4_modified and Scenario D4_modified.

The RTC introduced at Little Princess Vlei in the two modified scenarios operated under Local Automatic Control (LAC). The outlet was modelled as an adjustable weir. The rules that governed the weir were as follows:

i) If the water depth in Little Princess Vlei was below 3.22 m, the weir was non-operational and its crest elevation is set at 3.22 m above the pond floor (i.e. does not release water)

ii) If the water depth in Little Princess Vlei was above 3.22 m, the weir was operational and its crest is dropped to a depth of 3.02 m above the pond floor.

Using these RTC rules, the active storage depth in Little Princess Vlei was increased from 2 m (in the initial scenarios) to 3.22 m (in the modified scenarios) above the pond floor. The other ponds in the catchment were operated using the same RTC strategies as the initial scenario from

which they were created. The other aspects of the stormwater harvesting system (i.e. properties serviced, water distribution network and treatment requirements) were determined using the same methods as the initial fourteen stormwater harvesting scenarios.

4.4 Life Cycle Cost Analysis

A Life Cycle Cost Analysis (LCCA) was performed on each stormwater harvesting scenario in order to establish the total cost of implementing each system. LCCA is a commonly used economic evaluation method that looks beyond initial capital costs and considers all costs incurred by an asset over its period of service (WERF, 2011; Armitage *et al.*, 2013). By considering all costs associated to the asset, LCCA allows developers to make informed comparisons about the cost implications – initial capital costs versus operation and maintenance costs – of various systems (Swamee & Sharma, 2008). Additionally, LCCA are beneficial as they can be performed as an economic appraisal (costs/benefits caused by environmental influences) or a monetary appraisal (direct/indirect costs) (Armitage *et al.*, 2013). Since economic appraisal can be difficult as it requires placing a monetary value on subjective factors such as ecological services or aesthetic amenity, it was decided that only direct costs (i.e. capital costs and operational and maintenance expenses) would be considered in the LCCA performed in this study – thus, disregarding indirect costs and non-monetary aspects.

During a LCCA, it is necessary to convert all costs incurred by the asset over its lifespan to an equivalent time period (i.e. convert future costs to a present value) in order to calculate the total Life Cycle Cost (LCC) of an asset. Future costs are converted to their present value by multiplying the cost by the relevant discount factor. This conversion is performed using Equation 4-10 whilst the relevant discount factor is calculated using Equation 4-11. The LCC represents the sum of all future costs reduced to a present value minus the present value of any residual costs. It was calculated using Equation 4-12 (Lampe *et al.*, 2005; Woods-Ballard *et al.*, 2007; WERF, 2011; Armitage *et al.*, 2013).

$$PV_n = FC_n \times DF_n \qquad 4\text{-}10$$

Where: PV = present value for year, n (ZAR); FC = total future monetary costs in year, n (ZAR); DF = discount factor in year, n; n = number of years from present year

$$DF_n = \frac{1}{(1+i)^n} \qquad 4\text{-}11$$

Where: DF = discount factor for year, n; i = real discount rate; n = number of years from present year

$$LCC = \sum_{n=0}^{No.\ years} (PV_n)_{costs} - \sum_{n=0}^{No.\ years} (PV_n)_{Res} \quad \text{4-12}$$

Where: *LCC* = Life Cycle Cost (ZAR); *No. years* = total number of years in life cycle analysis; *n* = number of years from present year; *i* = real discount rate; *PV* = present value for year, *n*; *Res* = residual cost (ZAR)

As is evident from the aforementioned equations, the results of the LCCA will be sensitive to certain parameters such as the discount rate or life span of the system components. Hence, the following aspects were considered during the LCCA of each stormwater harvesting scenario:

- **Discount rate** – The selection of an appropriate nominal discount rate for a LCCA is of the upmost importance (Lampe *et al.*, 2005). The real discount rate is the rate used to convert future values to their present day equivalents. Hence, it is critical in assessing the feasibility of assets that incur significant future costs (e.g. operation and maintenance expenses). Fisher-Jeffes (2015) defined the real discount rate as '*the difference between the rate of return on the open market* [nominal discount rate] *and inflation'*. Whilst the chosen discount rate is at the discretion of those who are undertaking the analysis, the South African National Treasury recommends that government bond yield for the period of analysis should be used to represent the nominal discount rate (National Treasury, 2004). Shown in Table 4-13, are the values used to determine the real discount rate.

Table 4-13: Interest rate and inflation values used for calculation of real discount rate

Analysis period	Government 10-year bond (%) - Nominal discount rate*	Inflation (%)**	Real discount rate (%)
2005 - 2015	8.74	5.32	3.25

*(Investing, 2016)**(HomeFinance, 2016)

- **Life cycle duration** – Unlike the hydrological period of analysis for this study, the LCCA was performed for a fifty-year duration. While annual estimates like stormwater yield and each system's energy cost had been estimated based on the hydrological analysis, it was decided to use a longer life cycle duration for the LCCA as this better represented the actual lifespan of a stormwater harvesting system.
- **Life span of system components –** It was necessary to know the life span of all components used in each system to establish future capital costs as well as to calculate residual values at the end of the life cycle duration. Information on a component's lifespan was obtained from its supplier when available, or it was based on the recommendations of Armitage *et al.* (2013).

- **Residual value –** Residual values were calculated for system components that had yet to reach the end of their life span by the end of the life cycle duration used for the LCCA in this study. Residual value – also known as the salvage value – represents the remaining value of the asset and so it should be deducted from the LCC – shown in Equation 4-12. For the sake of simplicity, each component's residual value was calculated using the straight line depreciation method, whilst assuming that it had a zero end-of-life value.

After the LCC had been determined from a fifty-year duration, it was used to calculate the Equivalent Annualised Cost (EAC). As stated in its name, EAC represents the cost per year of owning and operating an asset. By reducing the LCC to an EAC, it was possible to determine the unit cost (ZAR/kℓ) of harvesting stormwater per year for each stormwater harvesting scenario. The EAC was determined using the following equation:

$$EAC = \frac{i(1+i)^n}{(1+i)^n - 1} \times LCC \qquad 4\text{-}13$$

Where: EAC = equivalent annualised cost (ZAR); i = real discount rate; n = number of years from present year; LCC = life cycle cost (ZAR)

4.5 Summary of method

This chapter has presented the methods that were used in this dissertation. The chapter was separated into four sections that each focussed on a key component of the method used; these were: the acquisition of data, the construction of the stormwater model, the development of the stormwater harvesting scenarios and the Life Cycle Cost Analysis (LCCA) that was performed.

This research required a substantial amount of data in order to model the various aspects of a stormwater harvesting system. Unfortunately, obtaining data at the required level of detail proved to be a limitation. Reliable flow data was only available for a single flow gauge located in the middle of the catchment, whilst daily evaporation had to be estimated using the Hargreaves' method as daily evaporation data was not available for the catchment. Furthermore, extensive disaggregation processes that have been developed in previous studies were followed to disaggregate rainfall and water demand data to a finer time-step.

This chapter details the process that was followed to construct and calibrate a stormwater model of the Diep River subcatchment using *PCSWMM 6.2*. This model was created to simulate the collection and storage aspects of a stormwater harvesting system. The distribution of harvested stormwater was modelled using water distribution modelling software – *EPANET 2.0* – whilst the treatment that harvested stormwater required was determined according water quality data acquired from monthly grab samples. The scenarios' ability to mitigate flooding for a 1-in-20 year storm was assessed using a design storm under worst case conditions. Sixteen different stormwater harvesting scenarios – fourteen initial scenarios as well as two additional scenarios

developed based on the results of the initial scenarios – were developed using these methods. These scenarios modelled four types of non-potable demands as well as four different storage and harvesting arrangements of the catchment's existing ponds. The different storage and harvesting arrangements were achieved using rudimentary RTC rules. After the results of these scenarios had been determined (Chapter 5), the two additional scenarios were developed by modifying the two scenarios that had the greatest economically viability – Scenario A4 and Scenario D4. These scenarios increased the active storage volume in Little Princess Vlei, despite flooding concerns, to determine if yielding a substantially greater volume of stormwater would increase the economic viability of stormwater harvesting in the Diep River subcatchment.

An economic evaluation of each stormwater harvesting system was performed using a LCCA. All costs that were incurred by the stormwater harvesting systems over their life-span were reduced to a present value. The total present value of each system was then converted to an Equivalent Annualised Cost (EAC) to establish the annual cost of operating each system.

5. Results and discussion

This chapter presents the results that were obtained after modelling the stormwater harvesting scenarios. Additionally, the chapter will engage in a discussion of the significance of each result.

Section 5.1 will present and discuss the performance indicators of each stormwater harvesting scenario: unit cost, average annual yield, volumetric reliability, overflow ratio and resilience.

Section 5.2 discusses the effect that the stormwater harvesting scenarios will likely have on the stormwater management of the Diep River subcatchment.

Section 5.3 discusses the results of the scenarios that were modified to investigate if improvements could be made to the initial stormwater harvesting scenarios to enhance the viability of harvesting stormwater from the stormwater ponds in the Diep River subcatchment.

All of the results that will be discussed in this chapter were generated, in part, using the catchment stormwater model constructed in *PCSWMM 6.2*. This made use of dynamic wave flow routing. The scenarios were modelled using three sets of disaggregated rainfall records ten-years in length (January 2005 – December 2014) with a fifteen-minute timestep (Section 4.1.4). The model calculated runoff generated for wet and dry weather using a five-minute timestep whilst it used a flow routing timestep of five seconds. The runoff continuity error for all scenarios equalled – 0.1% whilst the routing continuity error ranged between -0.1 to -1.7%. These errors were considered acceptable according to James (2010) who states that a continuity error below 10% is tolerable.

5.1 Exploration of stormwater harvesting viability

The following section analyses the results of the stormwater harvesting scenarios using different assessment parameters. Each assessment parameter will be discussed individually in its own subsection. The assessment parameters that were chosen to assess the viability of the stormwater harvesting scenarios include: unit cost of harvested stormwater (ZAR/kℓ), average annual yield of harvested stormwater (Mℓ/yr.), volumetric reliability, overflow ratio and resilience.

5.1.1 Unit cost of harvested stormwater (ZAR/kℓ)

The unit cost of harvested stormwater for each scenario was calculated by dividing the scenario's Equivalent Annualised Cost (EAC) (described in Section 4.4) by its average annual yield of harvested stormwater. The average annual yield of harvested stormwater was calculated from the annual yields of harvested stormwater over the ten-year hydrological analysis period. The unit cost of harvested stormwater for each scenario can be seen in Figure 5-1.

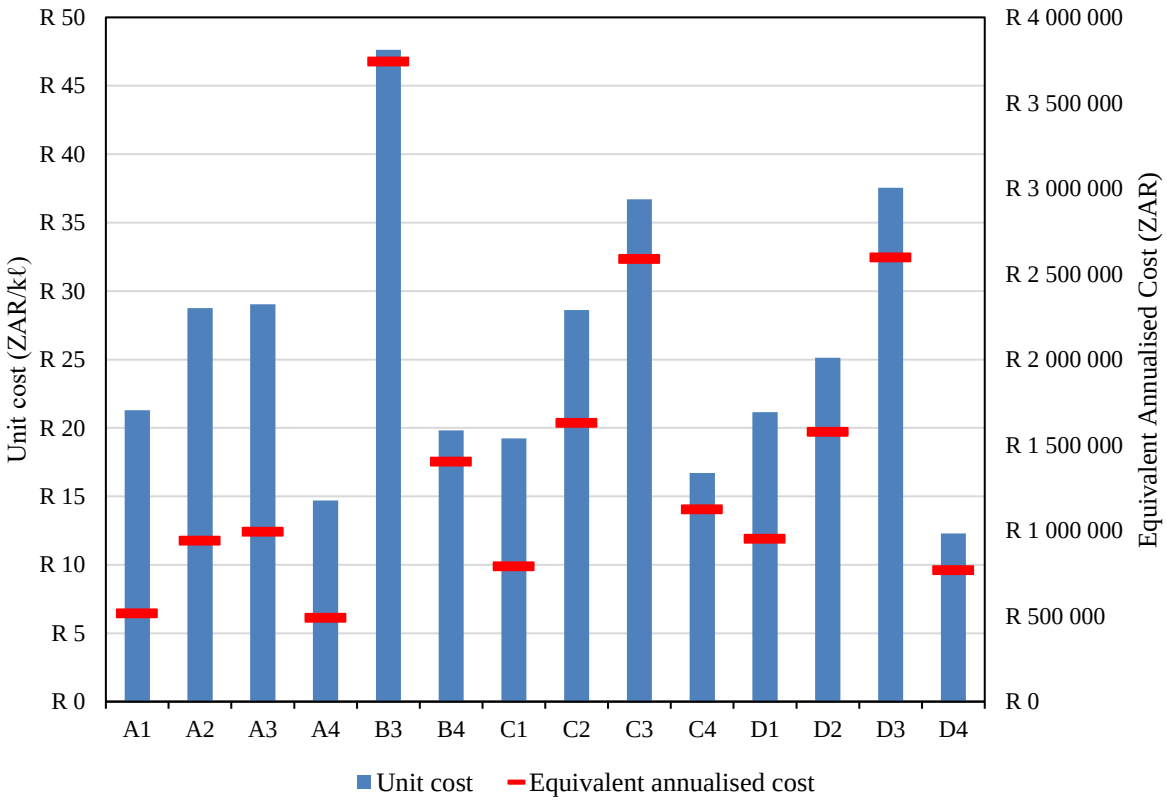

Figure 5-1: The unit cost of harvested stormwater from each of the modelled stormwater harvesting scenarios

In an effort to encourage water sensitive behaviour amongst domestic properties, the CoCT bills residential water users according to a stepped tariff. These tariffs as at July 2016 can be viewed in Appendix B. The CoCT provides residential water users 6 kℓ of water per month for free. If monthly water usage exceeds 6 kℓ per month, the next lowest tariff that the CoCT currently charges residential water users is 11.07 ZAR/kℓ. According to the average monthly water usage data (Section 4.1.8), the average water usage per month of all the suburbs modelled in this study ranged between 19 to 44 kℓ of water per month. The residents within the catchment were therefore typically billed either 15.87, 23.51 or 29.03 ZAR/kℓ (Step 2, 3 or 4 CoCT step tariff structure) for their water usage. Water usage by commercial properties in the CoCT is billed however, according to a flat rate of 17.10 ZAR/kℓ. The CoCT increases these tariffs for residential and commercial water usage during water scarce periods.

Figure 5-1 reveals that the majority of scenarios harvested stormwater at a unit cost that is comparable to the cost per kilolitre that residents and business owners in the Diep River subcatchment typically pay the CoCT for their water usage. The cost of supplying harvested stormwater to residential properties varied greatly. Naturally, the scenarios that modelled the greatest percentage of the total non-potable demand per property (i.e. toilet, clothes washing and irrigation demands – Water Demand Alternative 4) harvested stormwater at the lowest unit cost.

Two of these scenarios (Scenario D4 and A4) could harvest stormwater at a unit cost lower than what the residents in the catchment are typically charged by the CoCT, whilst the other two scenarios (Scenario C4 and B4), along with the other residential demand scenarios (Scenario A2, A3, C2 and D2), could harvest stormwater at a cost within the typical billing range. The remaining residential demand scenarios would all harvest stormwater at costs far greater than what residents in the catchment are currently charged. All of the scenarios that considered commercial demands (Scenarios A1, C1 and D1) harvested stormwater at a cost that was comparable to the rate that the CoCT charges for water usage in commercial properties.

It can be argued that an economically efficient stormwater harvesting system manages to maximise the volume of stormwater it harvests whilst minimising its costs. Reaching this optimal state can be problematic though, as it cannot be obtained by focusing solely on either maximising stormwater yields or minimising cost. This was apparent based on the undesirable result of the Scenario B3, which harvested stormwater from every pond in the system, but supplied it to the most properties. Whilst this scenario maximised the volume of harvested stormwater, harvesting from multiple ponds increases the cost of the system due to added capital, operation and maintenance costs. In contrast, the scenarios that harvested stormwater from a single pond (i.e. Little Princess Vlei) were able to limit these costs leading to stormwater harvesting systems with greater economic viability. Furthermore, the results illustrated in Figure 5-2 show that the economic viability of a stormwater harvesting scenario is highly susceptible to the extent of its water distribution infrastructure.

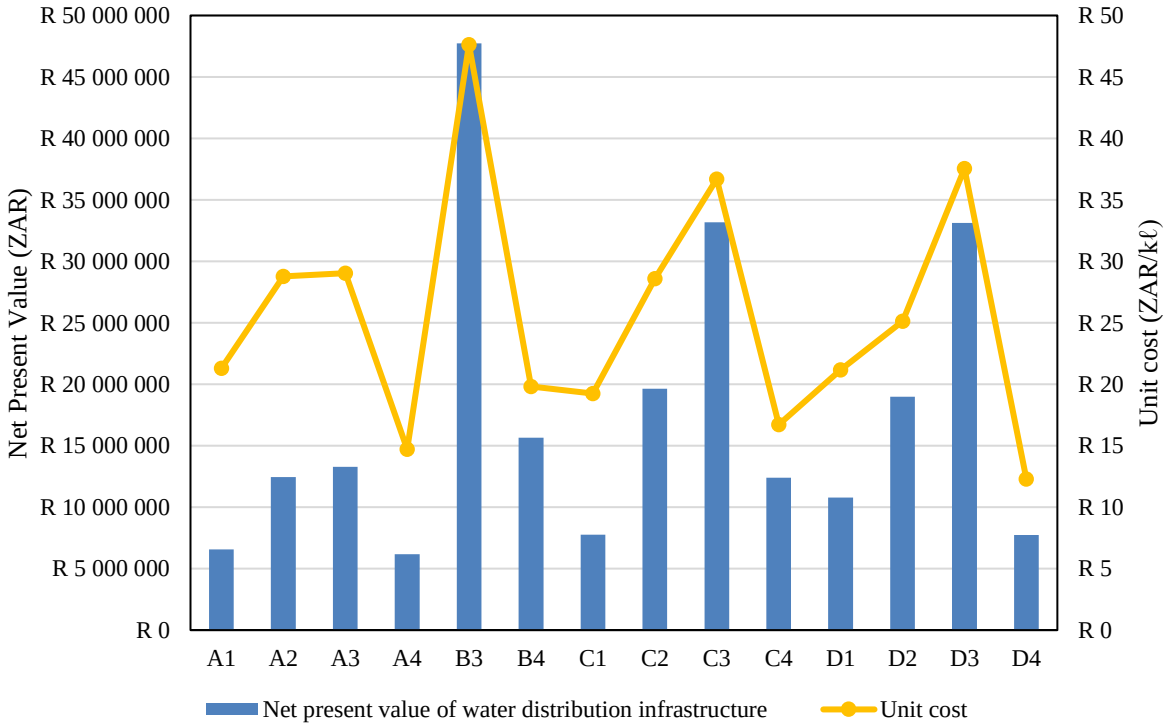

Figure 5-2: The net present value of each stormwater harvesting scenario's water distribution infrastructure compared to its unit cost of harvested stormwater

The Net Present Value (NPV) of each scenario's water distribution infrastructure is compared to its unit cost of harvested stormwater in Figure 5-2. As expected, this comparison reveals the significant impact that the cost of the water distribution infrastructure has on the cost to harvest stormwater. Figure 5-2 shows that scenarios with lower water distribution infrastructure costs are generally capable of harvesting stormwater at a lower unit cost. The capital and maintenance costs of the distribution infrastructure ranged between 40 – 54% of the total cost per scenario.

Since the cost of water distribution infrastructure has a significant influence on the economic viability of stormwater harvesting systems, successful stormwater harvesting systems typically minimise their water distribution infrastructure investment by:

i) Supplying harvested stormwater to areas with concentrated water demands (i.e. high water demand in a small area)

ii) Limiting the distance between their harvesting pond and the area they service.

Generally, the scenarios that would harvest stormwater at the lowest unit cost were those that considered the greatest non-potable water demand per household (i.e. Scenarios A4, B4, C4 and D4). Based on this finding, residential flat blocks should be an ideal property for a stormwater harvesting system to service since residential flat blocks create high water demands within a small area by concentrating households. However, the results of this study found that the scenarios that serviced residential flat blocks (Scenarios A2, C2 and D2) were not the most economically viable. This was due to the significant distance that residential flat blocks were located from harvesting ponds – 2.3 km on average – thus requiring extensive water distribution infrastructure, which increased the costs of the scenarios. In comparison, the scenarios that serviced commercial properties (Scenarios A1, C1 and D1) were located, on average, 1.1 km from their harvesting pond, but would supply harvested stormwater at a lower unit cost than the scenarios that serviced residential flat blocks despite the demand concentration being far lower in commercial properties.

5.1.2 Average annual yield of harvested stormwater

It is important to note that a scenario's average annual yield of harvested stormwater was heavily influenced by the storage configuration that it incorporated (Section 4.3.1.2). Since the storage capacity of the different storage configurations were not equal, the scenarios that had a greater storage capacity were capable of producing higher stormwater yields and thus, supplying greater demands. Consequently, to avoid misinterpreting the results presented in this section, when comparing the average annual yield of harvested stormwater per scenario it is important to consider the demand that the scenario aimed to supply. These results are shown in Figure 5-3.

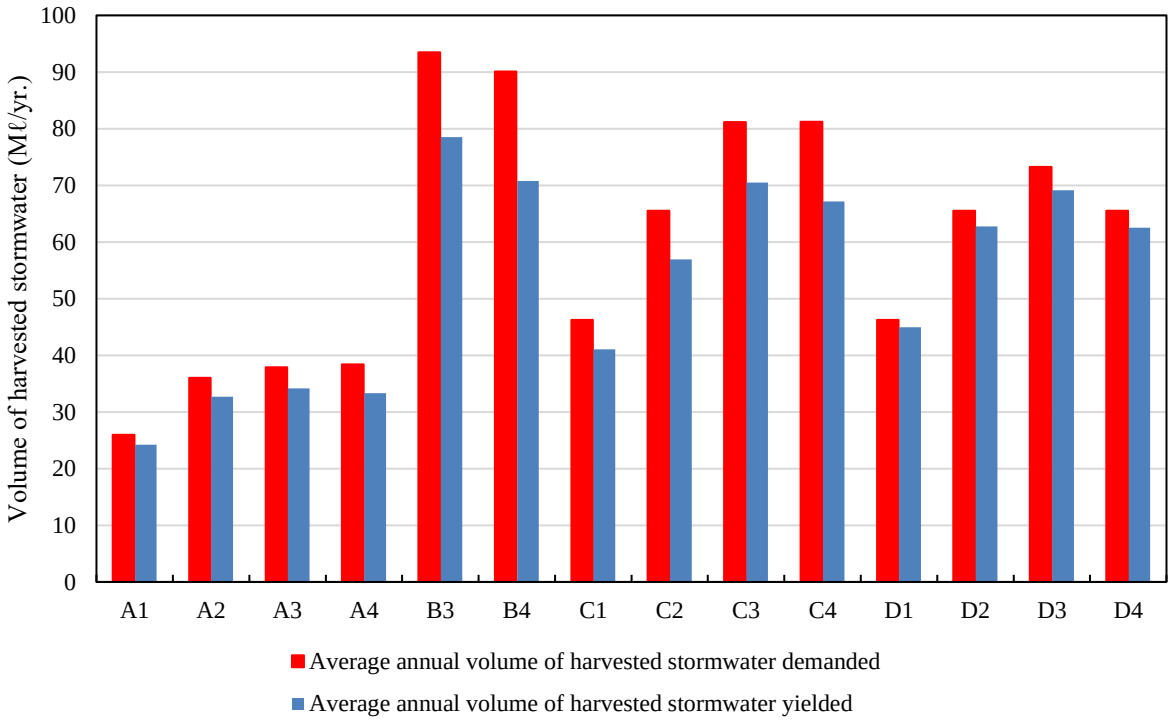

Figure 5-3: Average annual yield per stormwater harvesting scenario

As expected, Figure 5-3 shows that in general, increasing a stormwater harvesting system's storage capacity enables it to service greater demands and ultimately yield larger volumes of harvested stormwater. The scenarios that harvested stormwater from every pond (Scenario B3 and B4) had the greatest storage capacity, which led to the highest stormwater yields. In contrast, the scenarios that only harvested stormwater from Little Princess Vlei without using upstream storage (Scenarios A1, A2, A3 and A4) had the smallest storage capacity and yielded the lowest stormwater volumes. Interestingly though, whilst Figure 5-3 shows that using upstream storage ponds is an effective method to increase storage capacity, it also indicates that using upstream storage ponds is most efficient when upstream ponds are only used for storage and stormwater is only harvested from the most downstream pond (Scenarios D1, D2, D3 and D4). The scenarios that made use of upstream storage facilities all had equal storage capacity. Of these scenarios however, those that only harvested from Little Princess Vlei (Scenarios D1, D2, D3 and D4) yielded larger volumes of stormwater on average compared to those that harvested stormwater from Little Princess Vlei and Pond 4B (Scenario C1, C2, C3 and C4).

The type of property a scenario serviced also influenced the volume of stormwater it yielded. The results presented in Figure 5-3 show that the scenarios that serviced commercial properties (Scenario A1, C1 and D1) serviced the smallest average annual demand and subsequently harvested lower volumes of stormwater compared to the scenarios that serviced residential properties. This was because there was a limited number of commercial properties in the catchment, whereas there was an abundance of residential properties. The results presented in Figure 5-3 assumed each scenario would have 100% community participation, which may be

difficult to obtain in reality. Community acceptance is critical to the success of a stormwater harvesting system (Section 2.3.2). Mitchell *et al* (2005) stated that there is greater community acceptance of large scale (more than 10 000 properties) centralised systems compared to small scale (single neighbourhoods) decentralised systems. Whilst the scenarios that harvested stormwater from every pond would harvest the highest average annual yield, in reality these scenarios may not receive enough community acceptance to be successful. Instead, it is likely that the scenarios that only consider stormwater harvested from Little Princess Vlei, especially those that include upstream storage pond since they serviced a greater number of properties, would receive greater community acceptance compared to the other scenarios.

5.1.3 Analysis of harvesting performance indicators

The following section describes the results of the stormwater harvesting scenarios in terms of the storage facility performance indicators; these being: volumetric reliability, overflow ratio and resilience. It is important to note that the performance indicators were determined based on the results of a continuous ten-year hydrological analysis with a fifteen-minute time-step. These performance indicators are typically used to assess the performance of an individual storage facility. In this study, however, the performance indicators were used to describe the performance of the scenario as a whole, which could include multiple storage facilities that were parallel and/or sequential to one another. The performance indicators for scenarios that included multiple storage facilities were the average of each performance indicator for each storage facility in the scenario at which harvesting occurred.

5.1.3.1 Volumetric reliability

Similar to the average annual yield results discussed in Section 5.1.2, the results on the volumetric reliability that will be discussed in this section need to be carefully analysed to avoid misleading conclusions. It is important to note the following limitations of the result:

- It was intended that the total water demand each stormwater harvesting scenario serviced was limited so that each scenario had at least a 70% volumetric reliability – the minimum level of service recommended by Mitchell *et al.* (2006). This demand was approximated based on the demand that could be sustained by a scenario's full storage capacity during the average length of a 'dry' period. This method was explained in Section 4.3.1.3.
- Each scenario's volumetric reliability may have been improved by reducing its total water demand. Furthermore, the scenarios that had better volumetric reliability may have been a result of a better approximation of the total demand that they are capable of supplying.
- The majority of scenarios did not model the same total water demand. The only scenarios whose total water demand were equal to each other were scenarios C1 and D1 as well scenarios C2 and D2. This presented a degree of uncertainty in determining which scenario truly provided the greatest volumetric reliability.

Table 5-1: The volumetric reliability of each stormwater harvesting scenario

Scenario	Volumetric reliability	Scenario	Volumetric reliability	Scenario	Volumetric reliability	Scenario	Volumetric reliability
A1	0.93			C1	0.89	D1	0.97
A2	0.91			C2	0.87	D2	0.96
A3	0.90	B3	0.84	C3	0.87	D3	0.94
A4	0.87	B4	0.79	C4	0.83	D4	0.95

Notes:

- Scenarios A1, C1 and D1 modelled commercial property demands whilst the other scenarios modelled residential property demands.
- Scenarios A4, B4, C4 and D4 modelled demands with a seasonal component whilst the other scenarios modelled purely non-seasonal demands.
- Scenarios A1, A2, A3 and A4 only harvested stormwater from Little Princess Vlei.
- Scenario B3 and B4 harvested stormwater from every pond in the catchment.
- Scenarios C1, C2, C3 and C4 made use of upstream storage and harvested from two ponds (Pond 4B and Little Princess Vlei).
- Scenario D1, D2, D3 and D4 made use of upstream storage and only harvested stormwater from Little Princess Vlei.

The volumetric reliability of each scenario is presented in Table 5-1. From these results, it is evident that seasonal water demands affected a scenario's volumetric reliability. On average, the scenarios whose total demand included irrigation demands (Scenario A4, B4, C4 and D4) were found to have to the lowest volumetric reliability. The Diep River subcatchment experiences seasonal rainfalls, therefore, the scenarios whose demand included a seasonal demand component would experience their maximum demand during the driest period of the year. The system would often fail to meet these demands as water supplies would be at their lowest during this period. This result is consistent with the findings of Mitchell *et al.* (2007). It is unclear whether the type of property that a scenario serviced influenced its volumetric reliability. The results presented in Table 5-1 show that scenarios that modelled commercial property demands had marginally better volumetric reliability than scenarios that modelled residential property demand. However, as stated in Section 4.3.1.3, there are a limited number of commercial properties situated in the catchment, which thus limited the total demand the properties could supply.

Table 5-1 also showed that a scenario's storage configuration influenced its volumetric reliability. It was found that volumetric reliability decreased as the number of harvesting ponds in the scenario increased. By harvesting stormwater from multiple ponds, upstream harvesting ponds reduce the volume of runoff entering downstream harvesting ponds. This reduces the potential yield of downstream harvesting ponds. Consequently, this will reduce the downstream pond's volumetric reliability as well as the entire system's volumetric reliability if the total water demand has not been reduced to account for the decreased runoff volume entering the

downstream harvesting pond. This finding is apparent as the scenarios that only harvested stormwater from Little Princess Vlei (Scenario A1, A2, A3, A4, D1, D2, D3 and D4) would have the greatest volumetric reliability. However, if stormwater is only harvested from a single pond, using upstream ponds for storage will have a beneficial impact on a stormwater harvesting system's volumetric reliability. This is evident as the result of the scenarios that only harvested stormwater from Little Princess Vlei show that the scenarios that made use of upstream ponds for storage (Scenario D1, D2, D3 and D4) would have better volumetric reliability than the scenarios that did not make use of upstream ponds for storage (Scenario A1, A2, A3 and A4).

5.1.3.2 Overflow ratio

As described in Section 2.5.2, the overflow ratio indicates the fraction of water that enters the storage unit and is then subsequently spilled during the simulation period (Palla *et al.*, 2011). This ratio is used to determine how often a storage facility exceeds its full capacity. However, in this study, the overflow ratio is used to express the total volume of water that was released from the storage facility (i.e. it included the volume of water that was purposely released from the storage facility outlet for flood control). Negating the volume of runoff lost to evaporation and infiltration as well as the volume used for dead storage, the overflow ratio provides an approximate indication of the percentage of collected runoff a scenario failed to harvest. The overflow ratio of each scenario can be seen in Figure 5-4.

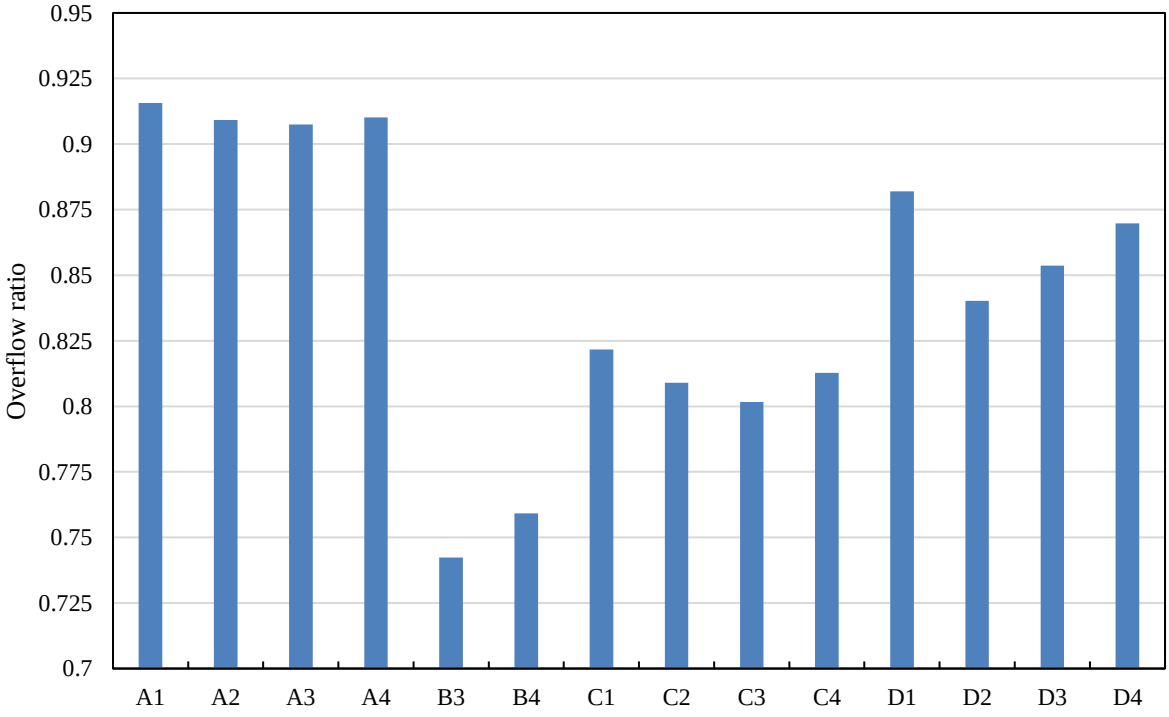

Figure 5-4: Overflow ratio per stormwater harvesting scenario

The results presented in Figure 5-4 show that the majority of stormwater that entered harvesting ponds was released in all of the scenarios that were modelled. Furthermore, it is apparent from Figure 5-4 that scenarios that had multiple harvesting ponds (Scenario B3, B4, C1, C2, C3 and C4) had lower overflow ratios compared to scenarios that had a single harvesting pond (Scenario A1, A2, A3, A4, D1, D2, D3 and D4). This finding can be explained by the outlet conditions of Little Princess Vlei since all scenarios used it as a harvesting pond. Due to concerns that increasing Little Princess Vlei's water level may lead to flooding its neighbouring properties (explained in Section 4.3.1.2), it was decided to model Little Princess Vlei's outlet weir based on its existing conditions rather than operate it using rudimentary Real Time Control (RTC) as modelled in this research. This meant that water was free to flow out of Little Princess Vlei whenever the water level exceeded the height of its weir crest. Consequently, this resulted in Little Princess Vlei spilling the majority of stormwater it collected. Since the other harvesting ponds operated their outlet using RTC, they were capable of retaining greater volumes of stormwater. As a result, the scenarios that had multiple harvesting ponds tended to have lower overflow ratios than the scenarios that only harvested at Little Princess Vlei. This finding indicates that it would be possible achieve greater stormwater yields by operating Little Princess Vlei's outlet weir using RTC to increase its active storage volume. To determine if this would enhance the economic viability of stormwater harvesting in the Diep River subcatchment however, two additional scenarios were modelled (Scenario A4_modified and D4_modified) that operated Little Princess Vlei's outlet weir using RTC. The results of the two additional scenarios are discussed in Section 5.3

5.1.3.3 Resilience

Resilience measures the rate that a storage facility recovers from a period in which it was in failure. It is the ratio of the number of continuous periods in which the system is in failure (i.e. unable to supply harvested stormwater) to the total duration that the system is in failure. Hashimoto *et al.* (1982) notes that resilience is equivalent to the average probability of the system recovering from failure in the next simulation time-step. Since this study used a simulation time-step of fifteen minutes, resilience would indicate the probability of a system recovering from failure within fifteen minutes. This was not considered useful however, so the total duration that each scenario was in failure was converted from minutes to days. By performing this conversion, it was possible to measure the probability of a system recovering from failure the following day. The resilience of each scenario is presented in Figure 5-5.

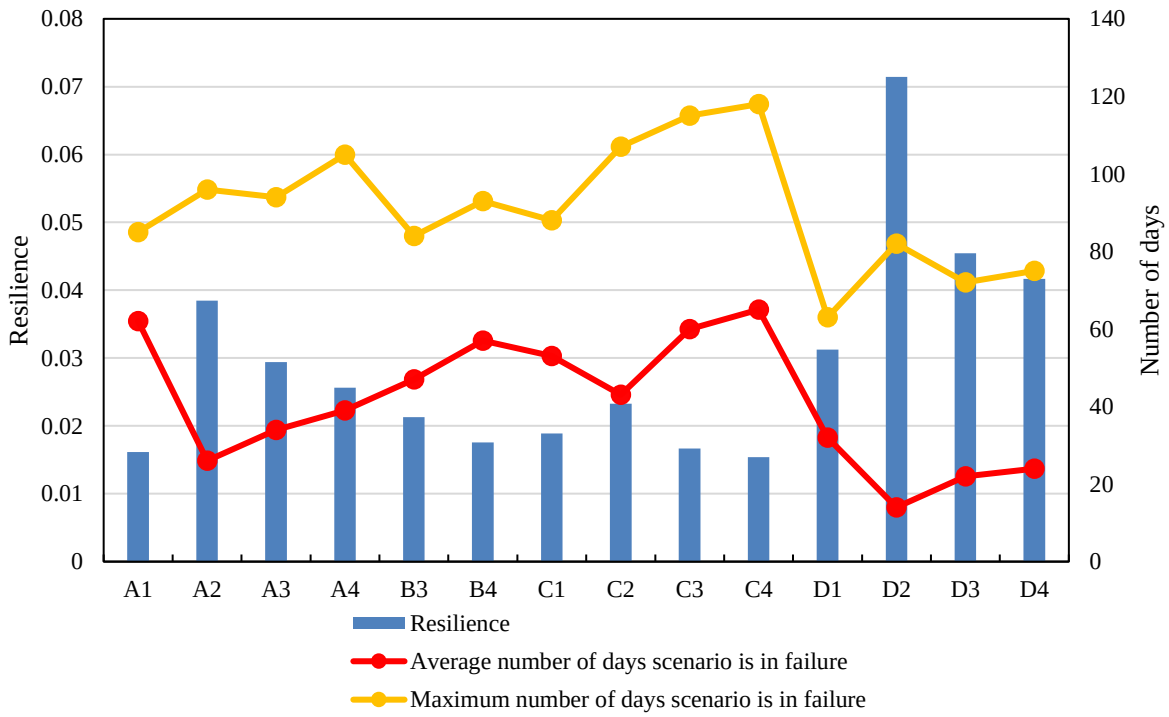

Figure 5-5: Resilience of each stormwater harvesting scenario

During a period of failure, the probability that a scenario would recover that day ranged between 2 and 7%. The low resilience of each scenario can partly be attributed to seasonal rainfalls that occur in the Diep River subcatchment. If a scenario entered a period of failure during the 'dry' period of the year, it was likely that this period would be prolonged due to the infrequency of rainfall occurring at this time of the year. This is substantiated by the average and maximum number of days a scenario took to recover from failure (Figure 5-5). Figure 5-5 shows that the average number of days a scenario took to recover from failure ranged between 14 and 65 days. Furthermore, it shows that the maximum duration of failure ranged between 63 and 118 days. These lengths of failure typically cover the driest months of the year (December to February). Despite the low resilience of all scenarios, it was apparent that scenarios that made use of upstream storage ponds and only harvested stormwater from Little Princess Vlei (Scenario D1, D2, D3 and D4), on average, had the greatest resilience. This was expected as upstream storage facilities are capable of replenishing the active storage zone of the harvesting pond when it is low (i.e. close to entering a period of failure). Surprisingly however, the scenarios that made use of upstream storage but harvested from two ponds (Scenario C1, C2, C3 and C4) on average, had the worst resilience. Presumably, this is because upstream ponds used for storage would only replenish the more upstream harvesting pond. Consequently, the volume of runoff entering the most downstream harvesting pond would be reduced. This would negatively affect the resilience of the most downstream harvesting pond and ultimately, the resilience of the system as a whole.

The type of water demand a scenario supplied also proved to influence the scenario's resilience. On average, the scenarios whose demand had a seasonal component (Scenarios A4,

B4, C4 and D4) had lower resilience than the scenarios that only modelled non-seasonal demands. It should be noted though, that resilience is a measure of the rate at which a storage facility recovers; hence, a storage facility that fails frequently but only for short durations will have a better resilience ratio compared to a storage facility that may fail on seldom occasions but for long durations. Thus, whilst the scenarios that serviced commercial properties (Scenarios A1, C1 and D1) were found to have the worst resilience, on average, they entered into a period of failure the least amount of times (shown in Table 5-2).

Table 5-2: The amount of times a scenario entered into failure during the period of analysis (January 2005 – December 2014)

Scenario	Failure occurrences	Scenario	Failure occurrences	Scenario	Failure occurrences	Scenario	Failure occurrences
A1	4			C1	8	D1	3
A2	10			C2	9	D2	7
A3	14	B3	15	C3	9	D3	15
A4	12	B4	14	C4	14	D4	7

5.2 Stormwater management impacts

If harvesting stormwater from the Diep River subcatchment's existing stormwater ponds is to be considered a viable opportunity, then stormwater harvesting should not detrimentally impact the ponds' existing function. As reported by CoCT (1986), the stormwater ponds situated along the Diep River were introduced to protect downstream regions of the catchment from the risk of flooding, which had substantially increased due to urban densification in the catchment. It has been cited in numerous studies that stormwater harvesting can have beneficial impacts on stormwater flows. Mitchell *et al.* (2006) however, commented that harvesting stormwater from existing stormwater ponds will not significantly increase flood attenuation benefits as, by design, stormwater ponds already attenuate peak stormwater flows. Furthermore, storing stormwater in ponds reduces the volume of storage that is available to attenuate storm events. For this reason, the rudimentary RTC considered in this study was designed so that water was released from each pond whenever the water level in the respective pond exceeded its maximum active storage depth. This was done to ensure that there was always an adequate storage volume available in each pond in the catchment for flood mitigation.

This section assesses how the Diep River subcatchment's existing stormwater ponds currently perform during various storm events to understand how this performance would be affected if the ponds were used for stormwater harvesting. This assessment was made by comparing: the percentage of storm events for various return periods that the ponds attenuate, the ponds' effect on peak flows and the ponds' ability to attenuate the peak flow generated from

a 1-in-20 year design storm under worst case conditions, under their existing conditions and when used for stormwater harvesting.

5.2.1 Effect on the percentage of storm events that are attenuated

At present, not all small storm events are attenuated by the Diep River subcatchment's stormwater ponds. These ponds were designed to handle larger storm events – up to a 1-in-20 year storm event – whilst smaller events are typically able to pass through the ponds without being attenuated. This is because the outlets of Pond 1, 2 and 3 are positioned at the same level as their pond floor, whilst Pond 4B and Pond 5 are off-line ponds that bypass flows associated with small events. Only Little Princess Vlei captures small flow events due to its outlet weir conditions, but if the pond's water level is above its outlet weir crest at the time of an event it provides minimal attenuation. In this study, to enable Pond 1, 2, 3, 4B and 5 to permanently store stormwater, their outlets were notionally modified to capture flows associated with small events. To ensure that this modelled modification did not inhibit their existing ability to attenuate storm events, the attenuation currently provided by the ponds was compared to the attenuation they provided whilst being used to harvest stormwater. This comparison is shown in Figure 5-6. The attenuation provided by the catchment's stormwater ponds under existing catchment conditions (i.e. not incorporating stormwater harvesting) is described by the 'black coloured bar' named 'As-is'.

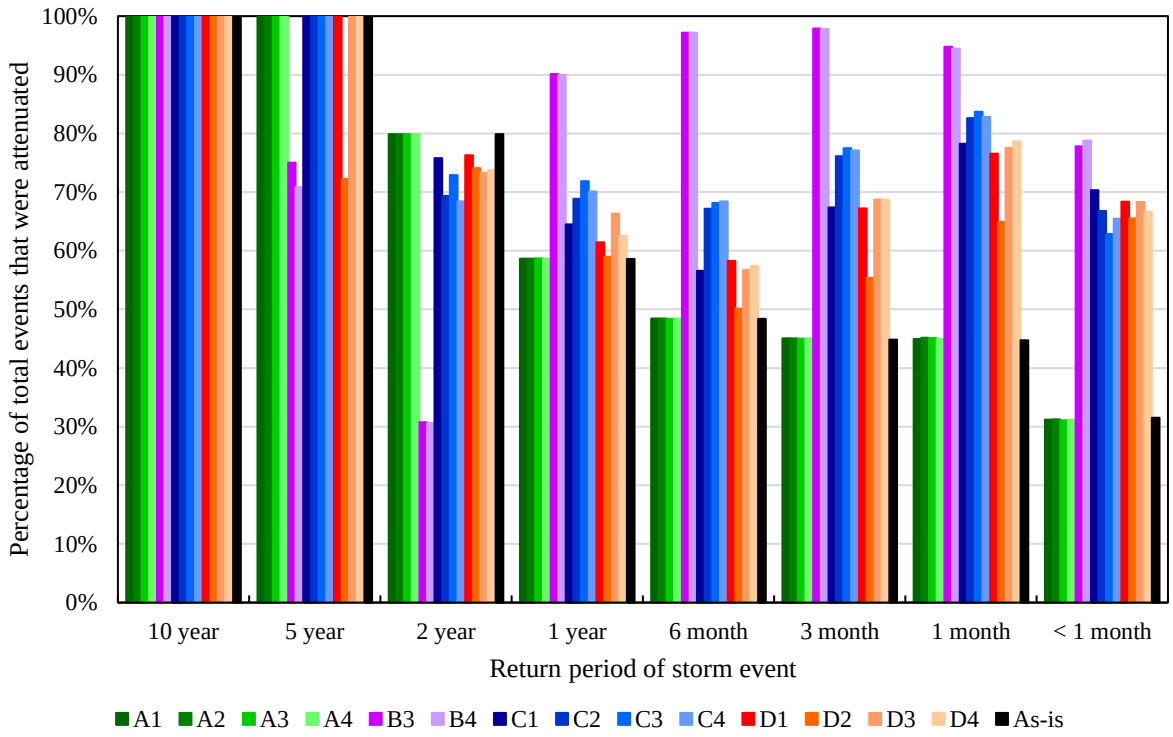

Figure 5-6: The percentage of the total number of storm events attenuated for various return periods by the ponds currently (As-is) and by each stormwater harvesting scenario

The flow through each pond in the catchment during each storm event that occurred during the period of analysis was inspected to determine if each pond had attenuated the associated flow. A pond was only deemed to have attenuated a storm event if the pond's peak outflow was less than its peak inflow during the event. Figure 5-6 represents the percentage of the storm events that were attenuated by all ponds in the catchment for each stormwater harvesting scenario as well as under the existing catchment conditions. For example, Figure 5-6 shows that all of the stormwater ponds under existing catchment conditions attenuated the peak flow associated with every ten-year storm event (i.e. peak flow was reduced). It also shows only 59% of peak flows through the catchment's ponds associated with a one-year storm event were attenuated. The fact that not all one-year events were attenuated under existing conditions may be because a single pond failed to attenuate any one-year event or because all ponds approximately failed to attenuate the same amount of one-year events. However, again, it should be noted that the existing pond were only designed to handle large storm events (i.e. up to a ten year return period or greater).

The purpose of Figure 5-6 is to determine if the percentage of the total number of storm events that the catchment's stormwater ponds attenuate increases or decreases when they are used for stormwater harvesting – the percentage attenuation to peak flows is discussed in Section 5.2.2. The results presented in Figure 5-6 show that using the Diep River subcatchment's stormwater ponds for stormwater harvesting would not impair their ability to attenuate storm events up to a 1-in-10 year return period throughout the catchment. The results show that the majority of scenarios attenuated the same percentage of storm events for large events (i.e. up to a ten year return period) as the ponds currently attenuate, whilst a greater percentage of small events was attenuated when the ponds were used for stormwater harvesting. This finding is consistent with Mitchell *et al.* (2006) who concluded that the small positive impacts achieved from harvesting stormwater from stormwater ponds will be more significant for smaller flood events.

It is evident from Figure 5-6 that a scenario's storage configuration influenced the attenuation provided by the catchment's stormwater ponds. The scenarios that harvested stormwater from every pond in catchment provided the greatest flow attenuation of all scenarios. These scenarios provided better flow attenuation than the catchment's stormwater ponds under existing conditions. The scenarios that made use of upstream storage also provided better flow attenuation than the catchment's stormwater ponds under existing conditions. Naturally, the scenarios that only harvested stormwater from Little Princess Vlei offered the least additional attenuation benefits since these scenarios had smallest storage capacity. These findings concur with Fisher-Jeffes (2015) who stated: "*The maximum attenuation of peak flows within the catchment would coincide with the largest storage volume and greatest consistent demand".*

The scenarios that harvested stormwater from every pond in the catchment provided greater attenuation to peak flows than the scenarios that used upstream storage ponds, despite having similar storage capacities and modelling the same type of non-potable demands. This is because harvesting at every pond in the catchment distributed water demands to all ponds. This meant that water was continuously withdrawn from every pond, which would lower each pond's water

level creating more available storage that could be used for flood control. When using the upstream ponds for storage, water was only withdrawn from one or two downstream harvesting ponds. As a result, the water level in upstream storage ponds was only lowered when water was needed to replenish a downstream harvesting pond and thus, these ponds would often have less available storage to attenuate flooding events.

5.2.2 Stormwater harvesting's effect on peak flows

The results of Section 5.2.1 reveal that harvesting stormwater from the Diep River subcatchment's stormwater ponds does not inhibit their ability to attenuate storm events with up to a 1-in-10 year return period. Section 5.2.1 however, did not reveal if peak flow rates are reduced to the same extent when using the ponds for stormwater harvesting as they are under their existing condition. For this reason, the results presented in this section will be used to determine if the percentage reduction in peak flow by the catchment's ponds when used for stormwater harvesting was comparable to the percentage reduction in peak flow by the ponds under their existing condition. The percentage reduction in peak flow was equal to the ratio between change in peak outflow and peak inflow through a pond to a pond's peak inflow. For example, a 100% reduction in peak flow would occur if a pond's inflow was fully captured by the pond (i.e. zero outflow). The percentage reduction in peak flow was determined for all ponds in the catchment for all storm events that occurred during the period of analysis. Since there are multiple ponds in the catchment as well as multiple peak flow rates with the same return period, the results that are presented in Figure 5-7 are the average percentage reduction in peak flow rate for the entire scenario. The attenuation provided by the Diep River subcatchment's stormwater ponds under current catchment conditions (i.e. not incorporating stormwater harvesting) is described by the 'black coloured bar' named 'As-is' in Figure 5-7.

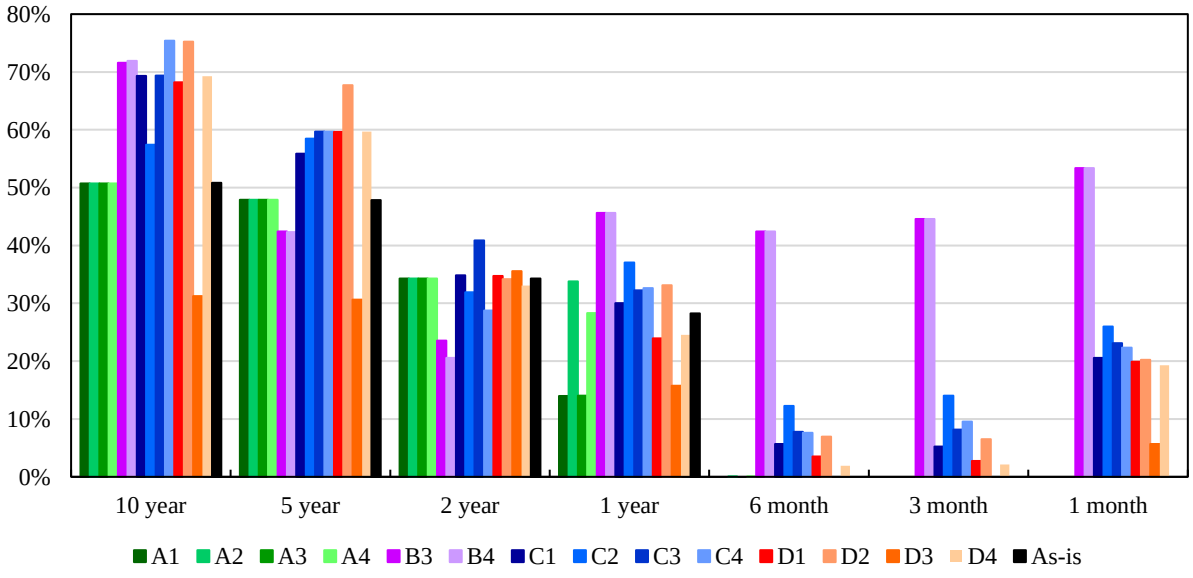

Figure 5-7: Average reduction in peak flow in each stormwater harvesting scenario

The results shown in Figure 5-7 indicate that harvesting stormwater from the Diep River subcatchment's stormwater ponds would not limit their existing ability to reduce peak flow rates for storm events with up to a 1-in-10 year return period. In fact, the majority of scenarios would, on average, provide a greater reduction in peak flow than what the catchment's ponds currently provide. While it is unclear if the type of water demand a scenario supplied effected its ability to reduce peak flows, it is apparent that the storage configuration a scenario incorporated had some influence. Storage configurations that included upstream storage appeared best at reducing the peak flow rate of larger events whilst the storage configuration that harvested from multiple ponds seemed most proficient at reducing the peak flow rate of smaller events.

5.2.3 Effect of 1-in-20 year storm event

As mentioned in Section 4.3.5, the ten-year continuous rainfall record under which the scenarios were modelled did not include a 1-in-20 year flood event. It was therefore necessary to assess the effect that a 1-in-20 year event may have on the ponds when used for stormwater harvesting using a design storm under the worst case conditions – i.e. all ponds' active storage volume full and saturated soil conditions. The analysis compared the percentage change in peak flows downstream of the pond system when it is used for stormwater harvesting to existing conditions for a 1-in-20 year storm event. Table 5-3 shows that centralised systems that did not make use of upstream storage (i.e. Scenario A1, A2, A3 and A4) had no impact on the downstream peak flow. This was to be expected as these scenarios implemented the least obtrusive changes on the existing pond system. Although marginal, the peak flow downstream of the pond system increased for decentralised systems and centralised systems that used distributed upstream storage (i.e. Scenario B3, B4, C1, C2, C3, C4, D1, D2, D3, and D4). This was because a portion of the ponds that was previously used to attenuate flow was taken up by stored runoff. The increase in peak flow however, was relatively minor in these systems because whilst the overall system was designed to attenuate storm events up to a 1-in-20 year event, the most upstream ponds (Pond 1, 2 and 3) had adequate storage to handle storms of a greater magnitude (CoCT, 1986).

Table 5-3: Percentage increase in peak flow downstream of the pond system for a 1-in-20 year event

Scenario	Percentage increase in peak flow	Scenario	Percentage increase in peak flow	Scenario	Percentage increase in peak flow	Scenario	Percentage increase in peak flow
A1	0%			C1	4%	D1	4%
A2	0%			C2	3%	D2	4%
A3	0%	B3	2%	C3	4%	D3	6%
A4	0%	B4	3%	C4	4%	D4	4%

5.3 Increasing the harvestable volume at Little Princess Vlei

The initial fourteen stormwater harvesting scenarios did not make use of RTC at Little Princess Vlei due to concern over raising the potential flood risk to properties neighbouring the pond. After analysing each scenario's overflow ratio (Section 5.1.3.2) however, it was clear that each scenario was releasing the majority of stormwater that entered their storage facilities. This suggested that increasing Little Princess Vlei's active storage volume using RTC could increase the volume of harvested stormwater. Since individual properties have a limited demand, stormwater harvesting systems that have a greater volumetric yield naturally have to service a greater number of properties. This results in a more extensive distribution network which equates to a larger cost investment. For this reason, it was decided to investigate whether increasing the active storage volume in Little Princess Vlei would create a more economically viable stormwater harvesting system (i.e. lower unit cost of harvested stormwater). The results of the modified scenarios, compared against their initial scenario equivalent, can be seen in Table 5-4.

Table 5-4: The results of the modified scenarios compared to their initial scenario equivalent

Scenario	Unit cost (ZAR/kℓ)	Equivalent annualised cost (ZAR/yr.)	Average annual yield (Mℓ/yr.)	Volumetric reliability	Overflow ratio	Resilience
A4_Modified	14.70	2 640 000	180	0.77	0.64	0.093
A4	14.69	490 000	33	0.87	0.91	0.026
D4_Modified	14.84	3 210 000	216	0.79	0.66	0.136
D4	12.29	770 000	63	0.95	0.87	0.042

The results shown in Table 5-4 reveal that both modified scenarios would harvest stormwater at a lower unit cost that what single residential properties (SRPs) in the Diep River subcatchment are typically billed for their water usage by the CoCT (15.87, 23.51 or 29.03 ZAR/kℓ). However, these unit costs are still greater than the lowest tariff that the CoCT currently charges for water usage (11.07 ZAR/kℓ), if monthly usage exceeds 6 kℓ per month – the CoCT provides 6 kℓ per month free of charge. Furthermore, neither of the modified scenarios harvested stormwater for a lower unit cost than their initial scenario equivalents (Scenario A4 and D4), although they were comparable. This is largely attributed to the substantially higher cost investment of the modified scenarios, which mostly resulted from added costs required to extend the water reticulation networks to distribute the additional available stormwater. Interestingly, the results of the modified scenarios indicated that the benefits of using upstream storage diminish as the storage capacity of the harvesting storage facility increases.

Using RTC to increase the active storage volume of Little Princess Vlei led to significantly larger annual stormwater yields. Whilst the increased annual yield did not lead to lower unit

costs, it did show that substantially larger volumes of stormwater can be harvested from Little Princess Vlei. The modified scenarios generally produced better performance indicators than their initial scenario equivalents. The modified scenarios had greater resilience and released a lower percentage of the volume of runoff entering the storage facility. Although, the modified scenarios had lower volumetric reliability, as noted in Section 5.1.3.1, results on volumetric reliability in this study can be misleading. The volumetric reliability results can be misleading as the demand a scenario serviced, which differed in most scenarios, was estimated based on its storage capacity. Furthermore, the modified scenarios considered seasonal demands, which meant that the system was typically in failure when its highest demand occurred.

By introducing RTC at Little Princess Vlei, the modified scenarios substantially increased the volume of stormwater that could be harvested from the pond. It should be noted though, that the rudimentary RTC used in these scenarios raised the average storage volume depth to 9 m above MSL. The maximum water level in the pond during the simulation period for either scenario was 9.24 m above MSL. As noted in Section 4.3.1.2, the lowest point on any property neighbouring Little Princess Vlei was 9.40 m above MSL. Thus, the modified scenarios substantially increased the risk of flooding these properties. Consequently, the results achieved by these scenarios could not viably be attained without implementing some method that would decrease the potential flood risk to existing properties (e.g. dredging Little Princess, using RTC of greater complexity to ensure the water level in Little Princess Vlei never surpassed 9 m above MSL, building dikes around Little Princess Vlei, etc.). The results of the modified scenarios are encouraging however, as they prove that far greater volumetric yields of harvested stormwater can be attained from Little Princess Vlei at a comparable unit cost to the most viable initial stormwater harvesting scenarios.

5.4 Summary of results

The performance of each stormwater harvesting scenario that was modelled in this dissertation was presented in this chapter. The performance of each scenario was assessed according to: unit cost of harvested stormwater, average annual yield, volumetric reliability, overflow ratio, resilience and its stormwater management implications.

The catchment's stormwater ponds would still offer a comparable attenuation of peak flows associated to large storm events (up to a 1-in-20 year return period), although slightly inhibited, when used for stormwater harvesting to what they currently provide. However, this was on the condition that RTC was used to maintain an adequate volume of storage for flood attenuation. RTC was also shown to be an effective method of optimising the available storage capacity within the catchment through the means of distributed storage. In particular, using RTC to manage the ponds used for storage that were positioned upstream of a harvesting pond increased a stormwater harvesting system's stormwater yield, volumetric reliability and resilience without negatively impacting its economic viability. The study also found that harvesting stormwater

from every pond in the catchment to increase stormwater yields was the least viable method of stormwater harvesting.

The majority of scenarios modelled in this dissertation were able to harvest stormwater at a unit cost that is comparable to what the CoCT typically bills catchment residents and business owners for their water usage. The scenarios that were considered most economically viable were those that modelled the greatest percentage of the total non-potable demand per residential property. These scenarios were the most economically viable as they supplied a high demand to areas situated in close vicinity to the system's harvesting pond, thus limiting the extent of their water distribution infrastructure. Ultimately, it was shown in this chapter that the economic viability of a stormwater harvesting system is most susceptible to the cost of its water distribution infrastructure. The results of the modified scenarios highlighted this finding. Whilst the modified scenarios harvested substantially greater stormwater volumes than the initial scenarios, they did not improve the economic viability of harvesting stormwater in the catchment. This was due to the extra cost required to extend the water reticulation network to distribute the additional harvested stormwater. This finding indicates that stormwater harvesting systems should be developed with equal consideration given to both the system's storage requirements and the urban densification of the area the system will service.

6. Conclusions

Like much of the world, South Africa is burdened with concerns over the future of its water security. This is largely attributed to the effects of urbanisation which has arguably pushed water demands to their sustainable limits. Furthermore, existing freshwater resources are being deteriorated by increased volumes of contaminated urban runoff. With the realisation that water scarcity poses a significant constraint to future human-development prospects, it has become widely recognised that a paradigm shift in conventional urban water management practices is required. This has prompted the development of alternative solutions to conventional water management practices that aim to reduce the vulnerability of existing water resources. One solution that is attracting a growing interest is using harvested stormwater to supplement water demands.

Due to the relative abundance of stormwater in urban areas, it is believed that stormwater can viably be harvested to substitute water demands. However, stormwater harvesting's applicability to all situations is questionable due to concerns over its storage requirements. Fortunately, stormwater ponds, a stormwater practice that is frequently used in urban areas, are a potential solution to these concerns as they are particularly well-suited for harvesting stormwater. Stormwater ponds are designed to provide temporary storage to attenuate peak surface runoff flows. Since flood attenuation is a periodic responsibility, the storage capacity in stormwater ponds if often underutilised. Thus, stormwater harvesting systems could take advantage of the available storage to store harvested runoff. In order to investigate if extending the use of existing stormwater ponds to include stormwater harvesting would be a viable option to improve the water security in South Africa, it will be necessary to develop reliable research. For this reason, this dissertation describes a case study on an appropriate existing urban catchment to determine if it is economically viable to use existing stormwater ponds for harvesting purposes. The catchment used in this study was the Diep River subcatchment, located in the Southern Suburbs of the City of Cape Town (CoCT). The catchment includes seven stormwater ponds that were primarily implemented for flood control, but could be retrofitted to harvest stormwater.

Several modelling tools were used to explore the viability of harvesting urban runoff from the stormwater ponds in the Diep River subcatchment; these included: a long-term continuous *PCSWMM 6.2* catchment stormwater model; *EPANET 2.0* water distribution network models; and a Life Cycle Cost Analysis (LCCA). These models required a substantial amount of data. Furthermore, as would be expected, the reliability of these models is dependent upon the reliability of the data from which they are developed. This proved to be a limitation of the research as reliable data at the level of detail required for a number of model components was not available, particularly in regards to observed data. Several methods were employed to overcome the data limitations, such as rainfall and water demand disaggregation.

This study initially modelled fourteen different stormwater harvesting scenarios to assess if it is economically viable to harvest stormwater from the Diep River subcatchment's stormwater

ponds. The scenarios modelled four different types of non-potable demands (referred to as water demand alternatives) since it is frequently stated in literature that the success of a stormwater harvesting system is influenced by the type of water demand it services. At the same time, the scenarios modelled four different harvesting and storage arrangements of the catchment's stormwater ponds (referred to as storage configurations), as again, it is stated in literature that the reliability of a stormwater harvesting system can be improved based on its storage facility layout. The various storage configurations were created by incorporating rudimentary Real Time Control (RTC) systems.

It was discovered that the majority of the modelled scenarios were able to supply harvested stormwater at a unit cost that is comparable to what residents and business owners are typically charged by the CoCT for their water usage. The type of non-potable demand a stormwater harvesting system supplied as well as the storage configuration it incorporated were both shown to have an effect on its economic viability. However, the results of this study show that a stormwater harvesting system's water distribution infrastructure has the greatest influence on its economic viability. The capital and operational cost of water distribution infrastructure ranged between 40 – 50% of the total cost in each scenario; hence, the scenarios that were capable of minimising these costs generally supplied harvested stormwater at the lowest unit cost.

The storage configuration that a stormwater harvesting incorporated proved to have a decisive role on the average volume of stormwater it harvested per annum. As would be expected, the scenarios that incorporated the storage configuration with the greatest storage capacity were able to service the greatest demand and ultimately, yield the greatest volume of stormwater. However, merely increasing a stormwater harvesting system's storage capacity does not necessarily lead to an economically viable system. Instead, it was shown in this study that the most economically viable method to increase harvested stormwater yields was to use upstream ponds for storage, but only harvest stormwater from the most downstream pond in the catchment. The scenarios that only harvested stormwater at the most downstream pond all harvested stormwater at an equivalent unit cost, however the scenarios that also made use of upstream ponds for storage substantially increased the volume of stormwater they harvested.

It was found that the storage configuration influenced how well a stormwater harvesting system performed in regards to the storage facility performance indicators used in this study (i.e. volumetric reliability, resilience and overflow ratio). The scenarios that made use of upstream ponds for storage, but only harvested stormwater at the most downstream pond in the catchment, on average, had the greatest volumetric reliability and resilience. The overflow ratio for all of the scenarios was high, which reveals that in each scenario, the majority of stormwater that entered the catchment's ponds was released. However, in comparison of scenarios' overflow ratios, the scenarios with the lowest overflow ratios were those that harvested stormwater from every pond in the catchment.

The catchment's stormwater ponds would still offer a comparable attenuation of peak flows associated to large storm events (up to a 1-in-20 year return period), although slightly inhibited, when used for stormwater harvesting to what they currently provide. Furthermore, it would

increase the percentage of small flow events that the ponds attenuate. It should be noted though, that the rudimentary RTC used in this study ensured that there was always an adequate volume of storage available in each pond for flood attenuation. Since the stormwater ponds were initially introduced to the catchment as a flood control measure, it was important to ensure that stormwater harvesting did not restrict the pond's ability to perform this function. Thus, this finding was critical to the viability of using the ponds for stormwater harvesting.

The results of the initial fourteen stormwater harvesting scenarios suggested that the Diep River subcatchment's stormwater ponds could yield greater volumes of stormwater by operating Little Princess Vlei's outlet weir using RTC to increase the pond's active storage volume. This was investigated by developing two additional scenarios based on the two initial scenarios considered to be most viable. The two initial scenarios were modified to include RTC at Little Princess Vlei's outlet weir – the pond that released the greatest fraction of runoff – to create the two additional scenarios. The results of the two additional scenarios proved that greater volumes of harvested stormwater could be attained at unit cost comparable to the most viable scenarios. Furthermore, the modified scenarios indicated that the benefits of using upstream storage diminish as the size of the downstream harvesting pond increases.

Harvesting stormwater from stormwater ponds in the Diep River subcatchment is potentially viable from an economic standpoint on the condition that it used to supply multiple non-potable end-use demands (i.e. irrigation, toilets and clothes washing) and that there is significant uptake amongst catchment residents. These conditions would limit the extent of the water distribution network and its associated costs, which was shown to have a substantial effect on the economic viability of a stormwater harvesting system. Furthermore, using RTC to manage upstream ponds that are used for storage was shown to be the most economically viable method to maximise the storage capacity of a stormwater harvesting system when storage availability is limited. Based on the results of this dissertation, stormwater harvesting systems of a similar economic viability as those developed in this study could be replicated in other South African catchments with similar development profiles and stormwater pond layouts. Furthermore, developments that have been designed to accommodate stormwater harvesting could increase stormwater yields by incorporating upstream stormwater ponds into their system.

7. Recommendations

The recommendations presented in this chapter were made so that future research can build on the results demonstrated in this dissertation:

i) Engage with stakeholders to understand the public perception towards stormwater harvesting in Cape Town. This study showed that stormwater harvesting systems that supplemented the greatest proportion of a property's non-potable demands were the most economically viable. It should therefore be understood if the public is willing to use stormwater for non-potable end-uses other than toilet demands or irrigation.

ii) Retaining stormwater in upstream facilities using Real Time Control (RTC) for the purpose of using it to later replenish a downstream storage and harvesting facility showed promising results. However, these promising results seem to diminish as the size of the harvesting pond increases. Further research should be made to determine if the benefits of upstream ponds for storage were unique to this dissertation.

iii) Further research should be directed towards investigating the balance between the complexity of a RTC system and the benefits it provides to a stormwater harvesting system.

iv) The number of properties that were serviced by the scenarios modelled in this study were estimated with the intention of obtaining an acceptable volumetric reliability (i.e. greater than 70%). However, this created uncertainty as to whether the system could yield greater demands or if the system was oversized. Due to the major influence that water distribution infrastructure costs have on the economic viability of stormwater harvesting systems, it is important that the extent of the network is estimated reliably. For this reason, future research should focus on creating a reliable method that will allow developers to estimate the optimum number of properties a stormwater harvesting system can service so as not to oversize the network.

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Appendix A : Collected rainfall records

Table A-1: Rainfall records that were collected for the study

Rain gauge	Source	Interval	Recorded Period	Reliable
Versfeld Avenue	Private Citizen	Daily	January 1995 - December 2015	Yes
Vineyard Road	Private Citizen	Daily	January 1979 - December 2015	Yes
Corsair Cresent	Private Citizen	Daily	January 2003 - December 2015	Yes
Wren way	Private Citizen	Monthly	January 1989 - November 2015	No
Glenwood Place	Private Citizen	Daily	July 2011 - December 2015	No
Groot Constantia	ARC*	Daily	January 2005 - December 2015	Yes
Princess Vlei Road	Private Citizen	Daily	January 2013 - December 2015	Yes
KEYS05DR	CoCT**	5 minutes	January 2012 - December 2015	No
DIEP05IR	CoCT	5 minutes	January 2012 - December 2015	No
DIEP05AR	CoCT	5 minutes	June 2013 - July 2015	Yes
DIEP05CR	CoCT	5 minutes	June 2013 - July 2015	No
DIEP05DR	CoCT	5 minutes	January 2012 - December 2015	No
DIEP05BR	CoCT	5 minutes	June 2013 - July 2015	Yes
DIEP05ER	CoCT	5 minutes	April 2013 - July 2015	Yes
Lysanders way	Private Citizen	Daily	January 2005 - December 2015	Yes
Violet Road	Private Citizen	Daily	June 2013 - December 2015	Yes
Woodley Road	Private Citizen	Daily	March 2014 - December 2015	Yes
Fairfield Close	Private Citizen	Monthly	January 1999 - December 2015	Yes
Great Constantia	DWA***	Daily	September 1967 - September 1990	Yes
Kirstenbosch	SAWS****	5 minutes	March 2010 - July 2015	Yes

*Agriculture Research Council (ARC)
**City of Cape Town (CoCT)
***Department of Water Affairs of South Africa (DWA)
**** South African Weather Services (SAWS)

Appendix B: Municipal electricity and water tariffs

Table B-1: City of Cape electricity tariffs July 2016

Tariff block	Tariff category	Tariff (including VAT)
Low consumption (< 1000 kWh/month)	ZAR/kWh	2.608
High consumption (> 1000 kWh/month)	ZAR/kWh	1.4735
High consumption (> 1000 kWh/month)	Connection fee (ZAR/day)	42.47

Table B-2: City of Cape Town water tariffs July 2016

Type of property	Tariff step	Level 1 tariffs – normally applied (consumptive 10% water savings)	Level 2 tariffs – applied during water restrictions (consumptive 20% water savings)	Number of suburbs whose average monthly water demand falls into particular step tariff
		(ZAR/kℓ)	(ZAR/kℓ)	Suburbs
Commercial	Constant tariff	R 17.10	R 19.88	-
Domestic – single residential properties and bulk metered flats	Step 1 (≤ 6 kℓ)	0	0	0
	Step 2 (> 6 kℓ ≤ 10.5 kℓ)	R 11.07	R 11.66	0
	Step 3 (> 10.5 kℓ ≤ 20 kℓ)	R 15.87	R 18.24	1
	Step 4 (> 20 kℓ ≤ 35 kℓ)	R 23.51	R 29.75	10
	Step 5 (> 35 kℓ ≤ 50 kℓ)	R 29.03	R 45.50	10
	Step 6 (> 50 kℓ)	R 38.30	R 85.09	0

Appendix C : Rainfall disaggregation parameters

Table C-1: 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	sigma_X σX mm d-1
January	0.401	0.751	0.044	2.761	0.003	80.946	80.946
February	0.265	0.890	0.067	99	0.254	87.570	87.570
March	0.132	2.159	0.076	99	0.861	45.893	45.893
April	0.004	0.201	0.002	82.349	6.177	99	99
May	0.419	0.235	0.052	3.917	0.080	99	99
June	0.832	0.581	0.102	2.806	0.028	99	99
July	0.674	0.762	0.115	2.745	0.028	82.666	82.666
August	0.582	0.350	0.056	2.521	0.039	79.740	79.740
September	0.673	0.449	0.237	4.104	0.127	62.868	62.868
October	0.228	0.192	0.038	3.478	0.062	47.804	47.804
November	0.301	0.001	1.552	9.967	1.160	77.757	77.757
December	0.008	0.037	0.001	77.724	3.453	73.532	73.532

Appendix D : Diep River subcatchment (2011 & 2016)

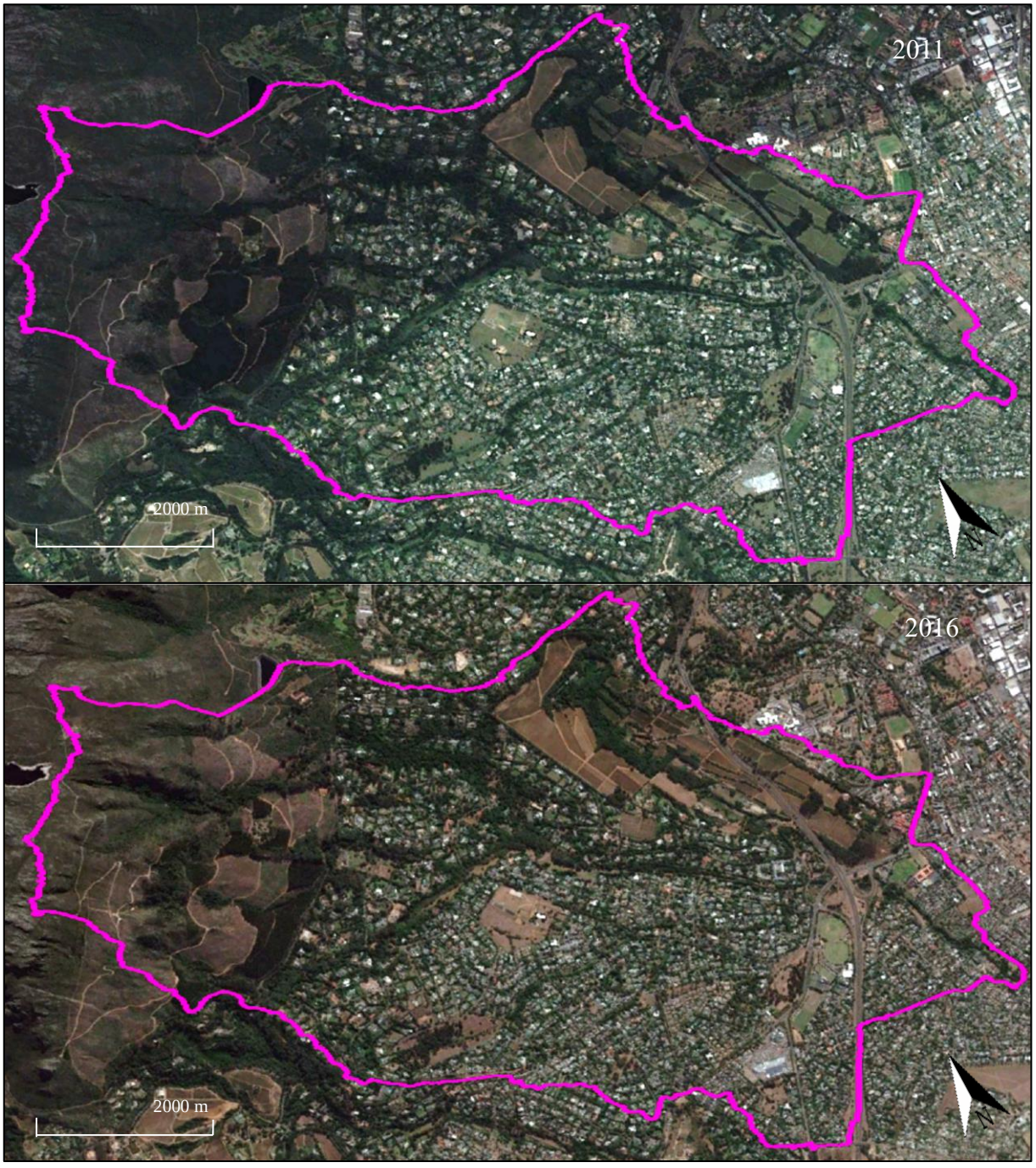

Figure D-1: Development changes in the Diep River subcatchment between 2011 and 2016

Appendix E: Real Time Control rules

The following appendix presents the actual Real Time Control (RTC) rules that were used to govern the various storage configurations. It should be noted that the depths listed are in meters. The orifice setting reflects the open/close status of the pond outlet; where: 0 = outlet is closed, 0.5 = outlet is half-open, and 1 = outlet is fully open.

Storage Configuration A

RULE 1. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV_SCENARIO#

RULE 2. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 3A
IF SIMULATION TIME > 0
AND ORIFICE C349_3 SETTING = 0.5

RULE 3B
IF SIMULATION TIME > 0
AND ORIFICE C1103_1 SETTING = 0.5

RULE 3C
IF SIMULATION TIME > 0
AND ORIFICE C353_2 SETTING = 0.5

Storage Configuration B

RULE 1. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV

RULE 1. POND1
IF SIMULATION TIME > 0

AND NODE SU3 DEPTH > 1.2
THEN PUMP P2 STATUS = ON
AND PUMP P2 SETTING = TIMESERIES POND1_SCENARIO#

RULE 1. POND2
IF SIMULATION TIME > 0
AND NODE SU13 DEPTH > 1.2
THEN PUMP P3 STATUS = ON
AND PUMP P3 SETTING = TIMESERIES POND2_SCENARIO#

RULE 1. POND3
IF SIMULATION TIME > 0
AND NODE SU2 DEPTH > 1.2
THEN PUMP P4 STATUS = ON
AND PUMP P4 SETTING = TIMESERIES POND3_SCENARIO#

RULE 1. POND4B
IF SIMULATION TIME > 0
AND NODE SU8 DEPTH > 1.2
THEN PUMP P1 STATUS = ON
AND PUMP P1 SETTING = TIMESERIES POND4B_SCENARIO#

RULE 1. POND5
IF SIMULATION TIME > 0
AND NODE SU1 DEPTH > 1.2
THEN PUMP P6 STATUS = ON
AND PUMP P6 SETTING = TIMESERIES POND5_SCENARIO#

RULE 2. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 2. POND1
IF SIMULATION TIME > 0
AND NODE SU3 DEPTH < 1.2
THEN PUMP P2 STATUS = OFF

RULE 2. POND2
IF SIMULATION TIME > 0
AND NODE SU13 DEPTH < 1.2
THEN PUMP P3 STATUS = OFF

RULE 2. POND3
IF SIMULATION TIME > 0
AND NODE SU2 DEPTH < 1.2
THEN PUMP P4 STATUS = OFF

RULE 2. POND4B
IF SIMULATION TIME > 0
AND NODE SU8 DEPTH < 1.2
THEN PUMP P1 STATUS = OFF

RULE 2. POND5
IF SIMULATION TIME > 0
AND NODE SU1 DEPTH < 1.2
THEN PUMP P6 STATUS = OFF

RULE 3. POND1
IF NODE SU3 DEPTH > 5.5
THEN ORIFICE C1103_1 SETTING = 1
ELSE ORIFICE C1103_1 SETTING = 0

RULE 3. POND2
IF NODE SU13 DEPTH > 6.6
THEN ORIFICE C349_3 SETTING = 1
ELSE ORIFICE C349_3 SETTING = 0

RULE 3. POND3
IF NODE SU2 DEPTH > 3
THEN ORIFICE C353_2 SETTING = 1
ELSE ORIFICE C353_2 SETTING = 0

RULE 3. POND4B
IF NODE SU8 DEPTH > 3.1
THEN ORIFICE C635_3 SETTING = 1
ELSE ORIFICE C635_3 SETTING = 0

RULE 3. POND5
IF NODE SU1 DEPTH > 1.8
THEN ORIFICE OR6 SETTING = 1
ELSE ORIFICE OR6 SETTING = 0

Storage Configuration C

RULE 1. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV_SCENARIO#

RULE 1. POND4B
IF SIMULATION TIME > 0
AND NODE SU8 DEPTH > 1.2
THEN PUMP P1 STATUS = ON
AND PUMP P1 SETTING = TIMESERIES POND4B_SCENARIO#

RULE 2. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 2. POND4B
IF SIMULATION TIME > 0
AND NODE SU8 DEPTH < 1.2
THEN PUMP P1 STATUS = OFF

RULE 3. POND1
IF NODE SU3 DEPTH > 5.5
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C1103_1 SETTING = 1
ELSE ORIFICE C1103_1 SETTING = 0

RULE 3. POND2
IF NODE SU13 DEPTH > 6.6
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C349_3 SETTING = 1
ELSE ORIFICE C349_3 SETTING = 0

RULE 3. POND3
IF NODE SU2 DEPTH > 3
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C353_2 SETTING = 1
ELSE ORIFICE C353_2 SETTING = 0

RULE 3. POND4B
IF NODE SU8 DEPTH > 3.1

THEN ORIFICE C635_3 SETTING = 1
ELSE ORIFICE C635_3 SETTING = 0

Storage Configuration D

RULE 1.LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV_SCENARIO#

RULE 2.LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 3. POND1
IF NODE SU3 DEPTH > 5.5
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C1103_1 SETTING = 1
ELSE ORIFICE C1103_1 SETTING = 0

RULE 3. POND2
IF NODE SU13 DEPTH > 6.6
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C349_3 SETTING = 1
ELSE ORIFICE C349_3 SETTING = 0

RULE 3. POND3
IF NODE SU2 DEPTH > 3
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C353_2 SETTING = 1
ELSE ORIFICE C353_2 SETTING = 0

RULE 3. POND4B
IF NODE SU8 DEPTH > 3.1
OR NODE S_LPV DEPTH < 1.21
THEN ORIFICE C635_3 SETTING = 1
ELSE ORIFICE C635_3 SETTING = 0

Storage Configuration A_modified

RULE 1.LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV_SCENARIO_MODIFIED#

RULE 2.LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 3.LPV
IF NODE S_LPV DEPTH > 2.72
THEN WEIR W14 SETTING = 1
ELSE WEIR W14 SETTING = 0.84

RULE 3A
IF SIMULATION TIME > 0
AND ORIFICE C349_3 SETTING = 0.5

RULE 3B
IF SIMULATION TIME > 0
AND ORIFICE C1103_1 SETTING = 0.5

RULE 3C
IF SIMULATION TIME > 0
AND ORIFICE C353_2 SETTING = 0.5

Storage Configuration D_modified

RULE 1. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH > 1.2
THEN PUMP P5 STATUS = ON
AND PUMP P5 SETTING = TIMESERIES LPV_SCENARIO_MODIFIED#

RULE 2. LPV
IF SIMULATION TIME > 0
AND NODE S_LPV DEPTH < 1.2
THEN PUMP P5 STATUS = OFF

RULE 3. LPV
IF NODE S_LPV DEPTH > 2.72
THEN WEIR W14 SETTING = 1
ELSE WEIR W14 SETTING = 0.84

RULE 3. POND1
IF NODE SU3 DEPTH > 5.5
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C1103_1 SETTING = 1
ELSE ORIFICE C1103_1 SETTING = 0

RULE 3. POND2
IF NODE SU13 DEPTH > 6.6
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C349_3 SETTING = 1
ELSE ORIFICE C349_3 SETTING = 0

RULE 3. POND3
IF NODE SU2 DEPTH > 3
OR NODE SU8 DEPTH < 1.22
THEN ORIFICE C353_2 SETTING = 1
ELSE ORIFICE C353_2 SETTING = 0

RULE 3. POND4B
IF NODE SU8 DEPTH > 3.1
OR NODE S_LPV DEPTH < 1.21
THEN ORIFICE C635_3 SETTING = 1
ELSE ORIFICE C635_3 SETTING = 0

Appendix F: Stormwater model parameters

Table F-1: Summary of calibrated subcatchments parameters

Parameter	25% Percentile	Average	75% Percentile
Area (ha)	4.016	9.099	11.359
Width (m)	115.078	212.845	256.318
Flow Length (m)	247.398	438.705	557.554
Slope (%)	0.775	3.819	5.583
Impervious (%)	5.031	8.206	9.059
N Impervious	0.017	0.019	0.019
N Pervious	0.238	0.274	0.260
Depression storage Imperviousness (mm)	2.095	2.676	3.390
Depression Storage Perviousness (mm)	7.730	8.448	8.842
Zero Imperviousness (%)	21.394	21.868	21.929
Percent Routed (%)	35.162	48.445	70.324
Suction Head (mm)	108.677	128.416	138.156
Conductivity (mm/hr)	5.993	11.492	8.138
Initial Deficit (fraction)	0.298	0.306	0.324

Table F-2: Summary of calibrated constructed conduit parameters

Parameter	25% quartile	Average	75% quartile
Length (m)	26.01	72.03	89.99
Roughness (Manning's coefficient)	0.015	0.015	0.015
Geometry (m)	0.60	0.81	1.00

Table F-3: Summary of calibrated manning's coefficients for natural channel sections

Parameter	25% Percentile	Average	75% Percentile
Bank roughness (Manning's coefficient)	0.038	0.045	0.039
Channel roughness (Manning's coefficient)	0.021	0.031	0.035

Appendix G: Diep River water quality

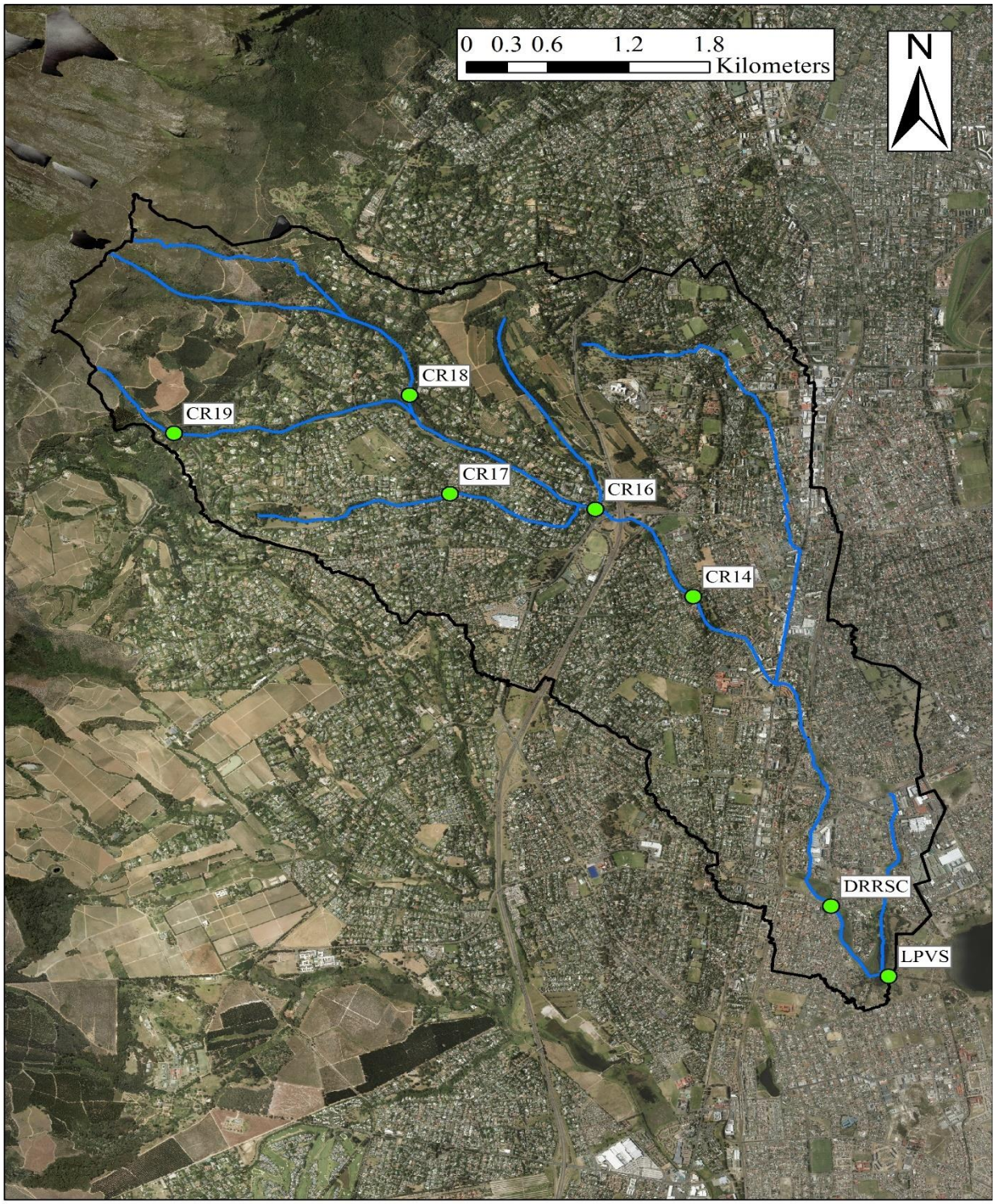

Figure G-1: The location of the CoCT water quality sampling points in the Diep River subcatchment

Table G-1: The percentage of water quality samples taken from each sampling point that meet the water quality requirements specified by the South African Water Quality Guidelines

	Target parameter	**CR16 (1993 – 2016)**	**DRRSC (1991 – 2016)**	**LPVS (1989 – 2016)**	**CR14 (1993 – 2003)**	**CR17 (1993 – 2003)**	**CR18 (1993 – 2003)**	**CR19 (1993 – 2002)**
Dissolved Oxygen (mg/l)	5 - 8	49%	26%	48%	41%	45%	39%	27%
Suspended Solids 105°C (mg/l)	0 - 20	78%	84%	72%	78%	67%	81%	81%
pH	6 - 9	99%	89%	98%	100%	98%	96%	91%
Conductivity (mS/m)	0 - 75	98%	98%	65%	100%	97%	100%	100%
COD as O (mg/l)	0 - 75	97%	98%	51%	100%	96%	97%	100%
Ammonia as N (mg/l)	0 - 2	100%	96%	61%	100%	100%	100%	100%
Nitrate + Nitrite as N (mg/l)	0 - 10	100%	100%	54%	100%	100%	100%	93%
Total Phosphorus as P (mg/l)	0 - 1	99%	98%	63%	100%	97%	99%	100%
E.coli (count/100ml)	0 - 130	31%	7%	43%	20%	52%	81%	86%

* Samples were taken on a monthly basis for the time period specified below the name of the monitoring station

Appendix H: Ethics clearance form

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: Alastair Rohrer Department: Civil Engineering

Preferred email address of the applicant: RHRALA001@myuct.ac.za

If a Student: Degree: MSc Engineering Supervisor: Professor Neil Armitage

If a Research Contract indicate source of funding/sponsorship:

Research Project Title: Stormwater ponds as an important component of a sustainable urban drainage system (SuDS) in Cape Town

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:		03/06/15

This application is approved by:

Supervisor (if applicable):		5/6/15
HOD (or delegated nominee): *Final authority for all assessments with NO to all questions and for all undergraduate research.*		
Chair : Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the above questions.		17/07/2015

Appendix I: Data sources for figures

Table I-1 provides the data sources that were used for figures that were not referenced in the text. This was because either more than one source was used to create the figure and/or the raw data was edited.

Table I-1: Data sources used in figures where reference is not provided

Figure	Source	Details
Figure 3-1	(CoCT, 2009b) (CoCT, 2009c) (CoCT, 2009d) (CoCT, 2009e)	Orthorectified image Shapefile of stormwater network layout Shapefile showing suburb boundary Shapefile showing city boundary
Figure 4-1	(CoCT, 2009b) (CoCT, 2009f)	Orthorectified image Shapefile of rainfall monitoring stations
Figure 4-3	(CoCT, 2009b) (CoCT, 2009g)	Orthorectified image Shapefile of flow monitoring stations
Figure 4-10	(CoCT, 2009a) (CoCT, 2009b) (CoCT, 2009c)	Shapefile showing land uses Orthorectified image Shapefile of stormwater network layout
Figure G-1	(CoCT, 2009b) (CoCT, 2009h)	Orthorectified image Shapefile of water quality monitoring stations