The prospects for stormwater harvesting in Cape Town, South Africa using the Zeekoe Catchment as a case study

Prepared by:

John Okedi, BSc. Civil Eng, MSc Water Res. Eng

Supervised by:

Professor Neil P Armitage, Pr. Eng, PhD

Dissertation submitted in fulfilment of the requirements for the degree of Doctor of Philosophy (PhD)

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

Declaration

I, **John Okedi**, declare that this thesis is my work. I have appropriately referenced sections taken from other sources with the Harvard – University of Cape Town convention for citation and referencing. I have worked with some BSc(Eng), and MSc(Eng) students and their contributions are referenced and attributed to the respective individuals. Some sections of this thesis have been submitted to the Water Research Commission of South Africa as a requirement for the provision of research funding. Other than these reports, I have not permitted anyone to copy my thesis with the intention of passing it off as their work.

Signature: Signed by candidate

Acknowledgements

I would like to thank the almighty God for the grace and strength to complete this PhD research programme at the University of Cape Town. For in him we live, and move, and have our being (Acts 17:28). I truly appreciate my family, especially my wife Sarah, and children Gerald and Daniella for the prayers, patience and understanding the need for me to work long hours including weekends and public holidays. They provided a social support structure that enabled me to complete the PhD programme. God bless you all, and I believe your patience and support was not in vain. To my parents, Mr and Mrs Epiyu, siblings Paul, Susan, Juli, Christine and Stella who have not seen me for five years, my absence was for a good cause. The wait is over, and we can now meet more often and enjoy each other's love and affection.

I would like to register my sincere appreciation to my supervisor and mentor Professor Neil Philip Armitage for the unwavering faith in me and support (both academically and financially) during this PhD research programme and report writing. Without his support, this work would not have even started. I am a significantly improved person academically and professionally due to his guidance and mentorship. This PhD research programme was made possible with funds from the Carnegie Corporation of New York and the Water Research Commission of South Africa. The financial support was invaluable, and I honestly believe that the training in the PhD programme has greatly enhanced my career as a researcher and lecturer. The future is truly bright for me, and I plan to discharge my professional responsibilities diligently as trained, for the benefit of the people in the communities that I will work. I also appreciate the colleagues and friends associated with the Urban Water Management research group that supported me in the PhD programme including Dr Kirsty Carden, Dr Lloyd Fisher Jeffes, Alastair Rohrer, Carina Doll, Nesre Reddy and Paul Mavundla. Special thanks to friends including Dr Enoma, Dr Arito, Dr Matongo, Patrick Bukenya, Dr Kiliswa and Gladwell Nganga for the encouragement during the PhD journey. I am grateful and cannot take for granted the cordial and pleasant environment you provided at our workplace. I also appreciate the Ugandan community in Cape Town that made us feel at home. Kisakye, Chebet, Prof Kalumba, Dr Sanya, Yoga, Musungu, Stella and Akampurila deserve mention. I am also thankful to Every Nation Church at Baxter in Rosebank, Cape Town for providing a spiritual home for my family. Exceptional thanks to Marc, Lana, Joseph, Rachel, Steve, Afolake, Mashekwa, Tabitha, Cathy, Patience, Siyanda and Mary-Ann.

Last, but not least, I appreciate Computational Hydraulics International for providing free software (PCSWMM), and the City of Cape Town (CCT) and South African Weather Service (SAWS) for the data used in this study. The modelling was carried out using the High-Performance Computing platform provided by the UCT eResearch. Some staff members at CCT, SAWS and UCT eResearch deserve mention including: Doug Austin, Gugulethu Quma, Candice Haskins, Rod Arnold, Nina Viljoen, Musa Mkhwanazi, Charlynne Arendse, Andrew Lewis, Ashley Rustin and Heine De Jager. Lisa van Aardenne of Climate System Analysis Group at the University of Cape Town provided rainfall and temperature data used to model the impact of climate change. The software, data and support provided were very valuable and contributed significantly to the completion of the PhD programme. I thank you all.

Abstract

The City of Cape Town in South Africa faced the possibility of taps running dry in 2018 because of a prolonged drought that commenced in 2015. With such droughts expected to reoccur frequently in future, this study investigated the prospect for stormwater harvesting (SWH) and use as an alternative water resource – selecting the 89 km^2 Zeekoe Catchment situated on the southern part of Cape Town as a case study. The study assessed potential to supply partially treated stormwater to non-potable water needs or fully treated water to potable water demands. The study determined that there was temporal mismatch between the identified non-potable water demands and the stormwater that is seasonally available from winter rainfall. Due to the mismatch, balancing storage was required in the range of 20 – 30% of the mean annual stormwater volume estimated at 18 Mm^3. The 61 stormwater ponds in the Zeekoe Catchment were assessed to determine the capacity to provide the required storage. Since the stormwater ponds were mainly for flood-control, the opportunity for multi-use was investigated using Real-Time-Control (RTC). RTC would allow for extended storage in the stormwater ponds for water supply and pre-emptive draining before storm events for flood control. The application of RTC on the stormwater ponds provided a capacity of 1 Mm^3 (about 5.5% of the mean annual stormwater volume). The capacity was inadequate as the stormwater supplied from the storage would only meet 44 – 60% of the demands, with a spill (water lost as overflow) of 35 – 51%. To provide additional capacity, an assessment was undertaken in stepwise increments of 1 Mm^3 to determine the optimal storage required from the shallow lakes (vleis) in the study area *i.e.* Zeekoevlei and Rondevlei. It was determined that after 4 Mm^3, there was limited increase in demand met and reduction in spill.

Stormwater could also be abstracted, fully treated to potable water standards and injected into the local potable water distribution system. Alternatively, the abstracted water from the vleis could be pre-treated at a new proposed WTP in the study area and then pumped to one of the existing water treatment plants. The assessment to determine influence of storage on quantity of stormwater supplied as discussed for non-potable water was undertaken for potable water. It was determined that for potable water that is required all the year round, the supply was not sensitive to changes in storage volume. Since the influence of balancing storage was limited, optimisation of SWH system was based on plant capacity to maximise supply and minimise spillage.

The available surface water storage *i.e.* stormwater ponds and vleis are currently used for other purposes such as flood control, recreation – including sailing – and to maintain ecology that require a permanent pool of water. The sensitivity of these activities was the driver for the study to consider alternative storage options such as groundwater aquifers. The study area had an unconfined aquifer with sandy soils that could support Managed Aquifer Recharge. The physical characteristics in the Zeekoe Catchment, *i.e.* largely flat terrain, pervious sandy soils and a relatively deep unconfined aquifer (20 – 50 m) would support typical borehole abstraction rates of 3.5 – 8.1 L/s per borehole from some 140 boreholes. This would provide a mean annual groundwater yield of 29 – 33 Mm^3 (about 15% of Cape Town water demand in 2018). Overall,

stormwater harvesting from groundwater storage was more advantageous than surface water as it provided larger water quantities and improved water quality.

This study has contributed towards identification of an alternative water resource by considering the possibilities of SWH from surface and groundwater storage for supply to potable and non-potable demands at a catchment scale. The study determined that the optimal use of stormwater requires a shift in the use to potable water uses. Treatment to potable standards would also eliminate the potential public health risks from cross connections. It was also determined that the treatment to potable water standards is more cost-effective for SWH at a catchment scale (centralised system) than using the water for non-potable purposes as it eliminates the need for the costly dual reticulation. Accordingly, this study recommends SWH and reuse to be for potable water needs where the abstraction is from a single location and the distribution through the existing potable water system. The factors that were determined to be important for the efficacious application of SWH included *inter alia* the availability of storage (surface or groundwater), the catchment characteristics (terrain, soil types, level of development, population density), and seasonal availability of the stormwater resource (winter or all year rainfall). The study also assessed the impact of land use and climate changes on the quantity of the stormwater. In terms of wider application, the study has provided insight into opportunities for stormwater use with partial or full treatment for non-potable or potable water demands respectively. The study has also provided a useful understanding of the potential scale and magnitude of the available non-potable water needs. It was also noted that reliability of the SWH was a function of storage capacity especially for supply to non-potable demands and the local rainfall regime. These issues need to be assessed to determine the prospects and viability of SWH and reuse in the given area.

Table of Contents

List of Tables

List of Figures

Symbols

A	Cross-sectional conduit area (m^2)
A_r/A_i	The ratio of runoff area catchment to infiltration area
C_o	Initial quantity at the start of the assessment (t=0)
Cost	Cost is the value in Rand
C_s/t	Annual accumulation rate of metal mass/soil mass (mg/kg per year)
C_t	Concentration or quantity at time t (units depend on pollution)
C_w	Concentration of metal in the runoff water (mg/m^3)
CWR	Crop Water Requirement (mm)
D	Derivative controller (dimensionless)
d	Thickness of the soil layer (mm)
d_1	Depth of water in the surface layer (mm)
D_2	Thickness of the soil layer in the infiltration cell (mm)
d_3	Depth of water in the storage layer (mm)
dh	Hydraulic head (m)
D_t	Demand in each time-step t (m^3)
dx and dy	Spatial steps in x and y-direction
e(t)	error (the difference between desired and actual water level)
e(τ)	Integral time error (s)
e_1	Surface layer evapotranspiration rate (mm/hr)
e_2	Soil layer evapotranspiration rate (mm/hr)
e_3	Storage layer evapotranspiration rate (mm/hr)
e_a	Actual vapour pressure [kPa]
E_{pan}	Class A pan evaporation data (mm/day)
e_s	Saturation vapour pressure [kPa]
ET_o	Evapotranspiration (mm/day)
f	Infiltration rate at any time t (cm/hr)
F	Cumulative infiltration (cm)
f_1	Infiltration rate of surface water into the soil layer (mm/hr)
f_2	Percolation rate of water through the soil layer into the storage layer (mm/hr)
f_3	Exfiltration rate of water from the storage layer to the in-situ soil (mm/hr)
f_c	Final infiltration rate (after equilibrium at steady state) at $t = t_c$ (cm/hr)
f_d	Total number of time-steps in which demand is not fully met
f_o	initial infiltration rate at $t - 0$ (cm/hr)
f_s	Number of time-steps in which demand is not fully met
FV	Future Value (Rands)
G	Soil heat flux density (MJ m^{-2} day^{-1})
g	Acceleration of gravity (m^3/s)

H	Hydraulic head of water in the conduit (m)
H_L	Local energy loss per unit length of conduit
i	Precipitation rate falling directly on the surface layer (mm/hr)
I	Integral controller (dimensionless)
i	Interest rate (%)
I_t	Inflow into the storage at current time t (m^3)
k	Darcy's hydraulic conductivity (cm/h)
k	Parameter of the Green & Ampt model determined as the intercept
$K(x, y)$	Hydraulic conductivity in 2-dimensions (cm/h)
k_c	Crop coefficient (dimensionless)
K_p	Factor of the proportional coefficient (dimensionless)
k_p	Class A pan coefficient (dimensionless)
L	Total length of the pipeline (m)
MAP	Mean Annual Precipitation (mm)
n	Parameters of the Green & Ampt model determined as the slope
n	Number of years
P	Proportional controller (dimensionless)
p	Mean daily percentage of annual daytime hours (dimensionless)
P_t	Current incident precipitation volume (m^3)
PV	Present Value (Rand)
Q	Flow rate through the conduit (m^3/s)
Q	Total volume of water pumped (L/s)
q_1	Surface layer runoff or overflow rate (mm/hr).
q_o	Inflow to surface layer from runoff captured from other areas (mm/hr)
R_a	Extra-terrestrial radiation (MJ m^{-2} day^{-1})
R_{eff}	Effective rainfall (mm)
R_n	Net radiation at the crop surface (MJ m^{-2} day^{-1})
S_c	Capillary suction at the wetting front (cm)
S_f	Friction slope (dimensionless)
T	Air temperature at 2 m height (°C)
t	Simulation time (s)
T	Total number of time-steps in the simulation period
T_d	Factor of the derivative time (dimensionless)
T_i	Factor of the Integral time (dimensionless)
T_{max}	The mean daily maximum temperature (°C)
T_{mean}	Daily mean temperature (°C)
T_{min}	Mean daily minimum temperature (°C)
TR	Daily temperature range (°C) (*i.e.* T_{max} - T_{min})

t_{years}	Time (years)
u_2	Wind speed at 2 m height (m s^{-1})
V_{cap}	Maximum storage capacity (m^3)
V_d	Dead storage volume (m^3)
VR	Volumetric Reliability (ratio)
V_t	Storage volume at the end of the current time-step t (m^3)
V_{t-1}	Storage volume at the end of the previous time-step (t – 1) (m^3)
v_x *and* v_y	Velocity (flux) in x-direction and y-direction
w.l (t)	Water level (m)
x	Length of the conduit (m)
Y_t	Yield in each time-step t (m^3)
Γ	Psychrometric constant [kPa $^oC^{-1}$]
Ψ	Porosity of the soil (%)
Δ	Slope of the vapour pressure curve [kPa $^oC^{-1}$]
Θ_2	Soil layer moisture content (fraction)
Λ	Pollution decay rate (day^{-1})
A	Horton's decay coefficient (hr^{-1})
P	Soil bulk density (kg/m^3)
Φ	Resilience (ability to recover from a period of failure)
Φ_1	Fraction of freeboard above the surface not filled with vegetation
Φ_3	Voids fraction of storage layer (fraction)

Acronyms and abbreviations

AADD	Mean Annual Daily Demand
CCT	City of Cape Town
CFA	Cape Flat Aquifer
CMIP	Coupled Model Inter-comparison Project
CSAG	Climate Systems Analysis Group
DEM	Digital Elevation Model
DO	Dissolved Oxygen
DWAF	Department of Water Affairs and Forestry
DWS	Department of Water and Sanitation
EC	Electrical Conductivity
E. Coli	Escherichia coli
FAO	Food Agriculture Organisation
GCM	General Circulation Model
GFS	Global Forecast System
GIS	Geographic Information System
LID	Low Impact Development
LIDAR	Light Detection and Ranging
MAR	Managed Aquifer Recharge
MAR&R	Managed Aquifer Recharge and Recovery
PCSWMM	Personal Computer Storm Water Management Model
PHA	Philippi Horticulture Area
RCP	Representative Concentration Pathways
RTC	Real-Time Control
SAWS	South Africa Weather Services
StatsSA	Statistics South Africa
SWH	Stormwater Harvesting
SWMM	Storm Water Management Model
UCT	University of Cape Town
WHO	World Health Organisation
WRC	Water Research Commission
WTP	Water Treatment Plant
WWTW	Waste Water Treatment Plant
UWM	Urban Water Management
YAS	Yield After Spillage
YBS	Yield Before Spillage

1. Introduction

South Africa is a semi-arid and water-stressed country that heavily relies on surface water from unevenly distributed rainfall with a Mean Annual Precipitation (MAP) of 450 mm (about 50% of the world MAP) (DWA, 2004; Pitman, 2011). The corresponding streamflow in the rivers is relatively low with a Mean Annual Runoff (MAR) of 50,000 Mm^3 (which is 50% of the Zambezi and 3% of the Congo Rivers) (Pitman, 2011). With the surface water resources almost fully developed and utilised, it has been projected that there would be a gap between water demand and supply of some 17% by 2030 unless there is a meaningful change in water supply and use patterns (DWA, 2008). In the City of Cape Town (CCT), the major reservoirs are projected to provide for demand up to between 2020 (based on a high annual demand growth scenario of 3.4%) and 2023 (based on a low annual demand growth scenario of 2.3%) (DWS, 2014). To mitigate the impact of the limited surface water resource, the CCT has implemented various measures, *e.g.* pressure reduction and pipe replacement to minimise loss through leakage, rising block tariffs to discourage wastage, the installation of water management meters to curtail excessive demand and provision of treated sewage effluent to selected users as an alternative to potable water. The measures have been effective in successfully maintaining water withdrawal and supply from the major reservoirs at a mean annual value of about 300 Mm^3/year over a period of over ten years (2003 – 2017) (GreenCape, 2017; DWS, 2014). The water supply from the major reservoirs was, however, severely tested by the prolonged drought of 2015 – 2018, which exposed the considerable limitations of the available surface water resources. The prolonged drought threatened the CCT with the possibility of taps running dry in 2018. With such droughts expected to reoccur frequently in future, coupled with increasing population and raising living standards, CCT is now considering alternative water resources such as seawater, wastewater effluent, augmentation of existing surface water systems and groundwater.

Seawater consists 97% of total global water and would ideally provide a sufficient and reliable water source for a coastal city such as CCT. Nevertheless, the costly water treatment, high energy requirement from a constrained sector and by-products such as brine that can negatively affect the environment typically limit the widespread utilisation of seawater as a water resource (El Saliby *et al.*, 2009).

Social acceptability associated with treated wastewater effluent is a limiting factor for this resource as shown in Ilemobade *et al.*, (2009) where 94% of respondents expressed support for recycling during a drought, but only 36% were willing to reuse the water themselves. The unwillingness was mostly intuitive as no specific cultural and religious grounds seemed to be preventing the possible reuse of treated effluent (Wilson & Pfaff, 2008). Due to the prolonged drought, the CCT implemented level 6B water restrictions in February 2018, where households were required to minimise domestic water use to 50 litres per person per day. To achieve the target, households were compelled to re-use greywater. Although the re-use of greywater contributed to a reduction in the total amount of water used at the household level, health risks during handling need to be assessed and guidance provided to safeguard the population.

With no additional major dam sites available near CCT, there are proposals to augment existing dams, *e.g.* raising the dam wall of the Voëlvlei to accommodate additional water brought in from various rivers and transfer of winter water from the Berg River (DWA, 2012). Although the augmentation of the Voëlvlei dam has been determined to be a feasible water intervention option, the system would only provide an additional 20 Mm^3/year, *i.e.* 3.3% of system yield, also equal to 6.7% of the mean annual withdrawal rate from the major dam reservoirs (DWA, 2012).

The CCT has also considered groundwater extraction from the Table Mountain Group (TMG) and Cape Flats Aquifer (CFA). Groundwater resources are widely used globally, however, there are potential severe irreversible environmental impacts such as ground subsidence. In Mexico City, excessive abstraction of groundwater over a long period of time (since the 1950s and greatly increased in the 1980s) has resulted in subsidence of 0.4 m/year since 1984, reaching 8 m in some areas by 2010 (Ortiz-Zamora & Ortega-Guerrero, 2010). On the other hand, Managed Aquifer Recharge and Recovery (MAR&R) with treated wastewater effluent and stormwater would mitigate the negative effects of the ground subsidence (Tredoux *et al.*, 1980; Tredoux & Cain, 2010).

It was noted that among the various alternative water resources considered in Cape Town *i.e.* seawater, wastewater effluent, augmentation of existing surface water systems and groundwater, stormwater harvesting and reuse was not included. This was identified as a missed opportunity and the study aims to investigate the prospects and viability of stormwater harvesting and reuse in Cape Town. In the field of urban hydrology, a water cycle management approach aimed at environmental protection has recently emerged since 1990 variously called *inter alia* Water Sensitive Urban Design (WSUD) in Australia, Low Impact Development (LID) in the United States and Sustainable Drainage Systems (SuDS) in the United Kingdom (Fletcher *et al*., 2014). It is has a more holistic water cycle management philosophy that aims to minimise net outflow of water from an urban catchment (Fletcher *et al.*, 2014). In South Africa, the application of these approaches have been the subject of research by the Urban Water Management research unit at the University of Cape Town that has culminated in the publication of guidelines to assist in the design and management of SuDS in South Africa (Armitage *et al.,* 2013), as well as a framework and guidelines for the implementation of WSUD (Armitage *et al.*, 2014). *Inter alia*, the new approach specifically promotes interventions such as stormwater harvesting (SWH) and re-use (Fisher-Jeffes, 2015; Rohrer, 2017; Rohrer & Armitage, 2017). Wong, (2007) notes that cities are potential catchment areas in their own right which, if well managed, would meet a substantial proportion of their water needs. Marsden & Pickering (2006) determined that the mean cost per kilolitre of SWH was lower than many other sources including *inter alia* seawater desalination, rainwater harvesting and long-distance pipelines.

In this study, the overall goal and objective was to investigate the prospects for stormwater harvesting (SWH) utilising surface and groundwater storage using the Zeekoe Catchment of Cape Town, South Africa as a case study. The focus and contribution of this study was the identification and determination of the opportunity and prospects for SWH and reuse as an alternative water resource. This included:

i. Open surface water storage in stormwater ponds enhanced through 'Real Time Control' (RTC), *i.e.* dynamic management of water levels in the ponds so that they are as empty as possible before a storm event and full afterwards;

ii. The use of stormwater ponds functioning as infiltration cells to transfer stormwater to aquifer storage for augmentation of groundwater resources;

iii. Determination of the extent and volumetric reliability of harvested stormwater supply for non-potable water uses such as urban agriculture, irrigation of residential gardens, open parks, and toilet flushing; and

iv. Economic analyses to determine the cost-effectiveness of SWH and use for potable and non-potable water. The costs of desalination were also determined for comparison.

This thesis consists of eight chapters. Chapter 1 (this one) provides an introduction including a background, research focus and the anticipated impact of the study. Chapter 2 is a review of the available literature relating to modelling SWH, assessment of water supply reliability and economic analysis. Chapter 3 includes an overview and statement of the method, selection of a suitability study area, available data and a summary of the method used in modelling surface water, groundwater and SWH. Chapter 4 discusses the method used and results of SWH from surface water storage, *i.e.* stormwater ponds and vleis (shallow lakes). Chapter 5 discusses the method used and the results of SWH from groundwater storage *i.e.* aquifer in the study area. Chapter 6 discusses on the supply of harvested stormwater to potable and non-potable demand (*i.e.* urban agriculture, public open spaces, residential garden irrigation and toilet flushing). Chapter 7 provides an economic analysis to determine the viability of SWH. Chapter 8 presents the conclusions, study contribution and recommendations. A reference list and appendices are provided at the end of this thesis.

2. Literature review

2.1 Overview

In this chapter, the relevant published work on similar studies, *i.e.* new and innovative approaches of stormwater harvesting (SWH) are discussed in eight sections to provide context to the study. Section 2.1 highlights water scarcity as the primary factor driving the need for SWH and use. The section also includes the likely causes of water scarcity and opportunity to utilise stormwater as a water resource. Section 2.2 provides an overview of stormwater as an alternative water resource and available opportunity for SWH and use. Section 2.3 discusses the prospects of using the available capacity in surface water storage, *e.g.* stormwater ponds and shallow lakes (vleis) for the effective implementation of SWH. It also includes the likely impact of climate change, issues of stormwater quality and possible water treatment options. Section 2.4 provides a discussion on the potential for Managed Aquifer Recharge and Recovery (MAR&R), *i.e.* augmentation of groundwater storage with stormwater. It also provides a discussion on the available opportunity for stormwater quality improvement in groundwater storage and a summary of international and local case studies with successful implementation of MAR&R; Section 2.5 discusses the identification and quantification of appropriate demand to be supplied with the harvested stormwater; Section 2.6 highlights the methods and components essential for an economic analysis and identification of benefits from SWH; Section 2.7 presents the expected social issues associated with SWH, and Section 2.8 provides a summary of the opportunity for SWH and use.

2.2 Water scarcity

The limited water resources and threat of water scarcity in Cape Town was the main driver for this study aimed at contributing towards proposing an alternative water resource currently not considered for development in CCT. Water scarcity can be defined as circumstances where fresh and easily accessible water resources are inadequate to meet water demand in a given area (Steduto *et al.* 2012). It has been attributed to the rapid population growth that commenced in the second half of the 20th century resulting in severe depletion of water resources (United Nations, 1999). The inconsistency in the trends, *i.e.* the tripling of the global population compared to the six-fold increase in water use in the 20th century indicates that there are other influencing factors (Cosgrove and Rijsberman, 2000). Various studies, *e.g.* Shiklomanov (1998), Raskin *et al.* (1997), and Alcamo *et al.* (1997, 2000) have shown that urbanisation with the associated improving standards of living and rising incomes are also key factors in significantly increasing water use. Further, there is evidence that human-influenced climate change has increased temperatures and reduced rainfall in some areas (Gucinski *et al.*, 1992; Hansen & Dale, 2001; Reichle *et al.*, 2003). The impact has been increased with growing water demand, *e.g.* irrigation due to high evaporation and reduced water resources from low rainfall to the point where water resource limit has been reached. Some examples of constrained water resources

include in Cape Town, South Africa where the City faced the possibility of taps running dry in 2018 because of a prolonged drought that commenced in 2015. Other examples of areas with constrained water resources include Perth, Australia (WA State Water and Strategy, 2003) and Mexico City (Ortiz-Zamora & Ortega-Guerrero, 2010). In Australia, the drop in groundwater levels has been exacerbated by below-mean rainfall in prolonged droughts and these trends are predicted to continue in the future due to climate change and global warming (Toze, 2006).

Developing countries such as South Africa with rapidly growing domestic, commercial and industrial demands are amongst the most affected by water scarcity (Rijsberman, 2006). There are about four billion people (more than half of the world's population), mainly in the developing countries, that are affected by water scarcity (Alcamo *et al.,* 1997, 2000). Water scarcity is also anticipated to increase due to water use escalation in some sectors, *e.g.* agriculture as a result of predicted future high temperatures (Rijsberman, 2000; Rijsberman and Molden, 2001). Another factor that contributes to water scarcity is that the most substantial volumes of water on earth are in a form not readily usable by humans, *e.g.* sea water and frozen water at the poles (IWMI, 2000). Seawater accounts for 97% of total global water, while 2.25% is trapped in glaciers and ice in the north and south poles, leaving only 0.75% as freshwater in groundwater aquifers, rivers and lakes (Turner, 2006). Low income and developing countries that can only afford to access water in readily and easily usable forms will be most affected by acute scarcity as water supplied will increasingly be inadequate to meet their needs or too costly (Rijsberman, 2006).

A new approach to stormwater management has emerged that allow for the preservation of the environment, and simultaneously promote the reuse of the resource that would have otherwise been lost in the traditional methods. This approach is variously termed *inter alia* Water Sensitive Urban Design (WSUD) in Australia, Low Impact Development (LID) / Green Infrastructure (GI) in the United States and Sustainable Drainage Systems (SuDS) in the United Kingdom (Fletcher *et al*., 2014). In South Africa, these were incorporated in guidelines to assist in the local design and management of SuDS (Armitage *et al.,* 2013), as well as a framework and guidelines for the implementation of WSUD (Armitage *et al.*, 2014). A key benefit that emerges from the new approach is the use of stormwater as a supply through what is termed stormwater harvesting (SWH). In SWH, stormwater is transformed from a 'threat to be managed' to a 'resource to be exploited' through means such as the extended detention of stormwater in ponds and/or Managed Aquifer Recharge and Recovery (MAR&R). In the implementation of MAR&R, the stormwater is transferred to ground aquifers for later extraction – while simultaneously fulfilling the function of flood protection (Mauck, 2017).

2.3 Stormwater as an alternative water resource

2.3.1 Overview

The conventional management of urban stormwater is to promptly collect and convey runoff from a storm event away from locations of rainfall incidence to avoid flooding and minimise inconvenience. However, disposal of stormwater in this manner results in the loss of a potentially

valuable water resource from an urban area. Some studies, *e.g.*(Wong, 2007; Brown *et al.*, 2008; Dotto *et al.*, 2014) have established that cities are potentially important catchment areas in their own right which, if well managed, can meet a substantial portion of its water demand. Many countries around the world are beginning to consider stormwater as an alternative water resource to augment existing water supplies due to the current and projected future water scarcity. A survey in Australia showed that about 92% respondents support the non-potable use of stormwater for municipal and industrial purpose, 79% believe that treated stormwater can be safe for potable supply and 65% accept that it is a cost-effective source of potable water for Australian cities (Hoban *et al.*, 2015). The successful implementation of SWH typically requires storage, *e.g.* in Singapore with the utilisation of a Coastal Reservoir (Tortajada *et al.*, 2013). Locally in Atlantis – a township located in the north of Cape Town, South Africa. SWH is implemented as MAR&R where groundwater storage is augmented with a mixture of stormwater and treated wastewater. There are two main challenges of SWH, *i.e.* the stormwater is only available in certain periods of the year (*i.e.* rainfall seasons), and it is typically associated with physical, chemical and biological pollutants (CCT, 2005; Brown & Magoba, 2009; Haskins, 2012). Studies that have assessed the potential for SWH as a resource for water supply (*e.g.* Mitchell *et al.*, 2007; Fletcher *et al.*, 2004; Philp *et al.*, 2008; Tortajada *et al.*, 2013; Fisher-Jeffes, 2015, 2015; Hoban *et al.*, 2015; Rohrer & Armitage, 2017) considered the following:

- Short to medium term storage of flood runoff – to balance the mismatch in the temporal availability of stormwater (rainfall seasons) and demands such as irrigation (highest in the dry season);
- Catchment management to reduce pollutant load – essential for reducing the cost of water treatment and allowing for reuse by a wide range of demands including potable and non-potable water purposes;
- Treatment of stormwater to a standard appropriate for the desired purpose using the so-called '*fitness-for-purpose*' principle – in the *'fitness-for-purpose'* principle, stormwater is typically treated to the minimum standard required by the end-user. Water that is treated to standards lower than potable water is restricted to non-potable water purposes such as irrigation of residential gardens and open parks, toilet flushing, car washing, street and car park washing, selected industrial uses and urban agriculture (Akram *et al.*, 2014; Coombes & Mitchell, 2006); and
- Construction of a separate reticulation system (sometimes called '*dual*' or '*third-pipe*' reticulation) – for distribution of non-potable stormwater to avoid contamination of potable water in the municipal reticulation system.

2.3.2 Stormwater as a water resource

SWH should not be confused with rainwater harvesting (RWH). RWH is the use of runoff from rooftops of buildings whilst SWH is the utilisation of catchment-scale stormwater management

infrastructure to collect, store and treat runoff for reuse mainly for non-potable water purposes (Mitchell *et al.*, 2007; Armitage *et al.*, 2013; Fisher-Jeffes, 2015). RWH is typically practised at a household scale mostly in remote villages and has been carried out for centuries (Hamdan, 2009; Mwenge, 2010). SWH is a more recent concept associated with urban areas and mainly promoted due to water scarcity (Mitchell *et al.*, 2007). SWH is normally implemented at a regional scale and requires suitably modified urban stormwater management infrastructure to support the practice. Some of the countries that have adopted SWH as a water resource include *inter alia* China (Hamdan, 2009), UK, USA, Australia (Philp *et al.* 2008), and Singapore (Lim *et al.* 2011).

In South Africa, the stormwater management infrastructure has mainly been focused on flood control with a very limited application for other purposes (Armitage *et al.*, 2013). However, with the current exceptional drought in Cape Town that commenced in 2015 (now considered the 'new normal' *i.e.* more frequent droughts are expected in the future), the city needs to reduce its reliance on conventional surface water schemes, and seek alternative sources such as stormwater (Fisher-Jeffes *et al.*, 2017). The main limitation of stormwater harvesting and use in South Africa is the generally poor water quality and the lack of suitable storage infrastructure (Fisher-Jeffes, 2015). As a rule, additional treatment would be required as harvested stormwater presents a health risk for both potable and non-potable water purposes (Fisher-Jeffes, 2015).

2.4 Stormwater harvesting from surface water storage

2.4.1 Overview

The successful implementation of a SWH scheme would require adequate storage to balance the seasonal mismatch in yield of stormwater and non-potable demand, *e.g.* irrigation (Mitchell *et al.*, 2008). The storage can be provided by ponds and lakes in the stormwater network. Some local studies (*e.g.* Fisher-Jeffes, 2015; Rohrer, 2017; Rohrer & Armitage, 2017) have shown that SWH is viable and has the potential to meet a reasonable percentage of demand in a study area. Meantime, a study such as Inamdar *et al.* (2013) developed an approach for identifying suitable sites for SWH in an urban area. In the study, the volume of harvested stormwater was compared with the available demand, and the sites were ranked with parameters such as demand, the ratio of runoff to demand and weighted demand distance (Inamdar *et al.,* 2013).

2.4.2 Surface water storage options

2.4.2.1 Introduction

The storage options for SWH systems include closed (*e.g.* underground tanks) or open (*e.g.* stormwater ponds). The determination and selection of a suitable storage option for a SWH system is case-specific and depends on climate, system yield, land availability, topography,

geology, demand and end-uses. It must consider the scale of the SWH system and the intended application of the harvested water (Fisher-Jeffes, 2015). According to DECNSW (2006), the design of the storage option should also consider how the water will be collected, stored, treated and distributed to end users. Mitchell *et al.* (2007) determined that the design of the storage option for the SWH system should consider maximising volumetric reliability while minimising storage size and associated costs. A brief overview of closed storage and a discussion on open storage are provided as follows.

2.4.2.2 Closed storage for stormwater

Closed storage systems such as underground tanks and geo-cellular systems are typically used for rainwater harvesting. The tanks collect and temporarily store rainwater that runs off roofs and paving of parking spaces at a specific location with a single or a few properties in a neighbourhood (Hatt *et al.*, 2006; Begum *et al.*, 2008; Armitage *et al.*, 2013). Permeable pavement is an example of a modified parking space for temporary storage of stormwater runoff to be harvested and reused at a local scale with minimal losses from evaporation and seepage (Armitage *et al.*, 2013). Due to limited capacity in closed storage, they are typically used at small-scale or property level SWH systems (Hatt *et al.*, 2006).

2.4.2.3 Open storage for stormwater

Open storage systems such as stormwater ponds, typically collect and temporarily store rainwater that runs off roofs and paving of parking spaces at a large-scale with several properties in a catchment (Hatt *et al.*, 2006; Begum *et al.*, 2008; Armitage *et al.*, 2013). Stormwater ponds may be categorised as detention ponds, retention ponds or constructed wetlands:

- **Detention ponds** – these are dry basin constructed to temporarily hold stormwater for short periods of time to mitigate flood risk downstream of the ponds (Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). They are probably the most widely used stormwater ponds; comprising some 70% of ponds in Cape Town (Rohrer, 2014). They are popular owing to their sizeable storage capacity for flood control while having no permanent pool of water that if not well maintained, may present a public health nuisance (Armitage *et al.,* 2013). They are also easy to construct and operate as they generally only provide a single function of flood control. Some detention ponds are however designed to serve as dual purpose such as recreational facilities or car parks during dry periods. They can also be adapted to contribute to the aesthetic value of the area (Woods-Ballard *et al.,* 2015). Detention ponds do not generally provide a significant water quality improvement benefit, mainly because the stormwater residence time is insufficient (Armitage *et al.,* 2013). Extended stormwater detention ponds do slightly better – mainly through deposition of solids / silt, however, the level of improvement is still limited (Armitage *et al.,* 2013).

- **Retention ponds** – these typically hold a permanent pool of water, providing some level of stormwater quality improvement in addition to peak flow attenuation from storm events to mitigate flood risk downstream of the ponds (Debo & Reese, 2003; Mays 2001; Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). The water quality improvement function in retention ponds is typically characterised by processes such as sedimentation, filtration, infiltration and biological uptake processes to remove pollutants from stormwater runoff (Stahre, 2006; Armitage *et al.,* 2013). Retention ponds provide limited flood control measure, a fundamental requirement in conventional stormwater management (Armitage *et al.,* 2013). Retention ponds also require regular maintenance to avoid public health risks from pollution build-up, mosquitos breeding and reeds covering the entire pond (Armitage *et al.,* 2013). Well maintained retention ponds can offer additional benefits beyond flood control such as a pleasant ambience and sense of affluence to an area providing a sense of serenity and good living (Haddock, 2004). There is evidence that a well-maintained pond system can provide an economic benefit by increasing the selling price of nearby properties by 10% to 25% (USEPA, 1995; Dinovo, 1995). Another advantage of retention ponds is that the permanent pond may be utilised as a source of water for various non-potable purposes (Armitage *et al.,* 2013). Conversely, a poorly maintained retention pond would be characterised with litter and solid waste, is a potential breeding ground for mosquitos and can result in a health hazard for nearby communities. Since retention ponds typically require a permanent pool of water, they cannot be used in arid regions with high evaporation rates and limited rainfall (Armitage *et al.,* 2013).

- **Constructed wetlands** – these are typically marshy with shallow water, partially or entirely covered in aquatic vegetation and provide more stormwater quality improvement than flood control (Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). Constructed wetlands also provide a vibrant habitat for fish, birds and other wildlife, potentially offering a sanctuary for rare and endangered species (Armitage *et al.*, 2013). Although they offer much lower flood control measures than detention and retention ponds, the opportunity to improve ecosystem health and aesthetic appeal that mimics natural systems make them attractive to property owners (Armitage *et al.*, 2013). The water quality improvement function in the constructed wetland is typically characterised by processes such as sedimentation, fine particle filtration and biological nutrient and removal of some pathogens (Field & Sullivan, 2003; Parkinson & Mark, 2005; Armitage *et al.,* 2013).

Other examples of open storage systems include *inter alia* natural wetlands, reservoirs, lakes, rivers, streams and creeks (Goonrey, 2005). Well-designed open storage systems can provide at least four types of benefits, *viz*.: the management of water quantity; the improvement of water quality; the provision of amenity; and the preservation of biodiversity (Armitage *et al.*, 2013). The management of water quantity can be further broken down into flood control and potential for SWH to be a significant water resource in its right (Armitage *et al.*, 2013).

2.4.3 Models used in the estimation of harvested stormwater volume

Many models have been developed since the 1960s with the earliest (*i.e.* the Stanford Watershed Model), able to simulate the behaviour of aquatic systems (Crawford and Linsley, 1966). The USA government agencies largely led the development of models for the assessment of stormwater quality and quantity in the late 1960s and 1970s (Zoppou, 2001). Since then, many models have been developed, and their complexity has significantly increased in the last few years (Elliott & Trowsdale, 2007). The models can be used for the estimation of the harvested stormwater volume. Some models previously used to determine harvested stormwater volume include *inter alia*: Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (eWater, 2013); City Drain (Achleitner *et al*., 2007); Urban Water Optioneering Tool (UWOT) (Rozos *et al.*, 2010); System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) (Lee *et al.*, 2012); SWMM (Storm Water Management Model) and the various proprietary derivatives of SWMM including XP-SWMM (XP Solutions, 2014), PCSWMM (CHI, 2014) and MIKESWMM (DHI Denmark, 2014); Info-works (Innovyze, 2011); UVQ (Mitchell & Diaper, 2005); and Aqua cycle (Mitchell, 2004).

Various studies such as Breen *et al.* (2006), Bach *et al.* (2014) and Akram *et al.* (2014) have shown that most of these models focus on only one component of urban drainage and not the integrated urban water cycle. Most models are unable to provide adequate information to fully inform the decision-making process (Fagan *et al.,* 2010).

The selection of a suitable model for any modelling exercise depends on the data that is available. Increasing the complexity of a model can only increase its reliability up to the level of the available data. To assist in the selection of a suitable model, Rangari *et al.*, (2000) provided valuable guidance based on available data, the characteristics of a specific urban watershed, scale and required detail. Zoppou (2001) reviewed various models and provided guidance on the selection of essential parameters and how to represent them in the modelling framework. The guidance provided in Zoppou (2001) also included representation of spatial-temporal resolution, estimation of water quantity and quality in an urban environment, economic analysis, optimisation and risk analysis.

In a critical reflection on integrated modelling with urban water systems, Bach *et al.* (2014) developed a typology to guide the selection of a suitable tool with considerations such as model structure, data requirement, computational and integration-related aspects, calibration and optimisation. Fletcher & Deletic (2008) detailed the impact of data requirements and calibration of integrated surface water modelling and management. Peña-Guzmán *et al.* (2017) reviewed various models developed from 1990 – 2015 to assess how they account for all components in the Urban Water Cycle (UWC) and potential use as decision-making tools. The review determined that most models are now focused on new supply sources such as RWH and SWH. Hutchins *et al.* (2017) assessed how models have advanced in the past few decades with a focus on technologies that have enabled better representation of the physical processes and the urban water cycle. The study observed that models in urban hydrology have shifted towards integrating natural urban landscape and engineered water cycles. Other authors such as Chiu *et al.* (2008),

Mwenge *et al.* (2008), Goonrey *et al.* (2009), and Darshdeep & Litoria (2009) have also provided guidance on the selection of suitable sites for implementing SWH and reuse.

2.4.4 Performance assessment of stormwater harvesting system

Several performance measures have been developed for assessment of SWH over the past few decades (Mitchell *et al.* 2007). These have been determined to be valuable in the assessment of the efficiency and effectiveness of a SWH system in various environments and circumstances. In a review of the suitability and application to SWH systems, McMahon *et al.* (2006) determined that the choice of suitable performance measures is based on the objectives of the study. Where the volume of water to be supplied is the primary motivation for urban SWH (Mitchell *et al.*, 2007), the Volumetric Reliability (VR) measure is typically as defined by Equation 2-1. VR can be described as the comparison of water supplied to demand for a given period, *e.g.* hourly, daily, weekly, and monthly, typically represented as a ratio (Mitchell *et al.,* (2008).

$$VR = \frac{\sum_{t=1}^{T} Y_t}{\sum_{t=1}^{T} D_t} \qquad \textbf{Equation 2-1}$$

Where Y_t - Yield in each time-step t (m^3)

D_t - Demand in each time-step t (m^3)

T - Total number of time-steps in the simulation period

Another approach is the use of 'Resilience' of a SWH system as a performance assessment method. This approach assesses the ability of a storage unit to recover from a period of failure after a deficit in supply as the basis for the performance of a storage component (McMahon *et al.*, 2006). In essence, 'Resilience' is the relative likelihood that a storage unit will recover from a period of failure to meet demand. The formula for estimation of Resilience as proposed in (McMahon *et al.*, 2006) is given in Equation 2-2.

$$\varphi = \frac{f_s}{f_d} \qquad \textbf{Equation 2-2}$$

Where φ - Resilience

f_s - Number of individual continuous time-steps in which demand is not fully met

f_d - Total number of time-steps in which demand is not fully met

2.4.5 Real-Time Control for stormwater harvesting systems

The traditional design of stormwater management infrastructure typically focuses on only flood control with runoff peaks and volumes estimated based on rainfall and land use of a specific area (South African National Roads Agency Limited, 2013). With some sections of urban areas experiencing increasingly high surface runoff linked to climate change and land development, the stormwater discharge rates and volumes may frequently exceed designed capacity. To minimise the risk of flooding, increasing the physical capacity of stormwater management infrastructure may be required through *inter alia* installation of additional channels, pipes and storage components. In a developed urban area, this may not be possible or costly for various reasons including *inter alia* limited land for expansion, or the need to demolish buildings and roads (García *et al.*, 2014). To address this challenge, Real-Time Control (RTC) has been developed to provide additional capacity without necessarily increasing physical storage (Borsanyi *et al.*, 2008). Stormwater infrastructure management with RTC involves dynamic control of the system with specific operational rules to consolidate available storage with the primary objective of minimising redundancy (Colas *et al.*, 2004; US EPA, 2006). It includes dynamic management of water levels in the storage components to increase retention time and optimise hydraulic capacity (Vallabhaneni & Speer 2011). The management includes continuous monitoring and adjustment of stormwater flow rates and storage volumes with a set of rules depending on the status and needs in the system (Garcia *et al.*, 2015). The earliest stormwater management systems with RTC were implemented in the USA in the 1960s with the goal of volumetric expansion of a constrained network (Borsanyi *et al.*, 2008). Subsequently, several stormwater systems with RTC have been designed and implemented mainly in the developed world including Europe and North America (Garcia *et al.*, 2015).

The additional capacity with implementation of RTC may provide other benefits such as storage for SWH and improvement of stormwater quality (Garcia *et al.*, 2015). RTC has also been identified as a flexible and cost-effective method to deal with the impact of climate and land use changes on stormwater infrastructure (Vezzaro & Grum, 2014). The implementation of RTC also allows for dynamic management with integration of new information such as rainfall forecasts in various data formats *e.g.* radar (Liguori *et al.,* 2012; Ocampo-Martinez and Puig, 2010; Thorndahl *et al.,* 2013). Three approaches for implementation of RTC in stormwater systems were identified in various studies (*e.g.* Colas *et al.,* 2004; US EPA, 2006; Vallabhaneni & Speer, 2011; García *et al.,* 2015; Rohrer, 2017) and are discussed as follows:

- **Local control –** in the local control management approach, the system is managed using measurements taken at a specific location and adjustments concerning prevailing conditions (Colas *et al.*, 2004; US EPA, 2006). Adjustments are then applied either manually (opening and closing of valves) or automatically with actuators. A Rule-Based Control (RBC) that incorporates 'if-then' rules (*i.e.* if this happens, then do this) can be used as a basis for adjustment and control of the RTC systems (García *et al.*, 2015). Most researchers prefer these controls due to straightforwardness in application.

- **Regional control –** the regional control management approach is similar to the local control with regard to the independent management of storage facilities (Gaborit *et al.*, 2013). The difference is the remote regulation – with specific adjustments for a given device being applied to the entire region (Colas *et al.*, 2004; US EPA, 2004). Hence, a manually operated system with site-specific opening and closing of valves would not be suitable for a regional RTC control (US EPA, 2004). A regional RTC control system typically requires a remotely controlled regional communication system such as Supervisory Control and Data Acquisition (SCADA) program located on a central server system (Colas *et al.*, 2004; US EPA, 2006). The SCADA manages data with alarms and operators to monitor and control the processes with dedicated telephone lines, wireless communication with radio, cellular systems or satellite telecommunication devices as typical data transmission systems (Schutze *et al.*, 2004). Regional control systems can also be automatically operated with optimisation algorithms (García *et al.*, 2015). Some stormwater management models, *e.g.* Infoworks (Innovyze, 2011) and SWMM (James *et al.*, 2010) have incorporated optimisation algorithms such as Proportional Integral Derivative (PID) to simulate RTC systems. The PID is a generic closed-loop scheme set to provide a desired process by continuously applying corrective action (Rossman, 2010). The PID controllers are suitable for system optimisation as they allow for the continuous manipulation of the system in real-time to reach the desired state (James *et al.*, 2010). They can be applied to continuously adjust the openings at outlets to control flow rates based on selected values associated with PID and its several other combinations, *i.e.* PI, PD (James *et al.*, 2010). The selected PID values provide the level of adjustment required of the opening and are a function of the difference between the measured variable and the set point (Schutze *et al.*, 2004). The initial PID values are based on projects with similar objectives, and calibration performed using differential equations, real or simulated experiments (Campisano & Modica, 2002). Another advantage of the regional RTC system is the ability to limit optimisation rules to a specific facility and local conditions (US EPA, 2006). Additional discussion and application of PIDs is provided in Chapter 5.
- **Global control –** the Global control management approach is also a server-based system where the data, controls and adjustments of the actuators for the entire network is centralised (Colas *et al.*, 2004). The adjustments in the global control management approach are typically complex and based on a decision support framework with the application of RBC, optimisation algorithms and predictive forecasting (Colas *et al.,* 2004; US EPA, 2006; Rohrer, 2017). The Global control management approach is considered a complex system as it requires the implementation of multiple control rules with predictive forecasting in the decision support framework. It also needs rigorous network analysis and planning before it is implemented – as well as supervisory control by an operator who has a good understanding of the system dynamics (US EPA, 2004; Rohrer, 2017). If well set-up, the approach typically provides the highest functionality and most optimal operational efficiency of the three RTC approaches (Colas *et al.*, 2004).

In the selection of a suitable RTC approach, consideration is typically given to the level of complexity appropriate for the study area especially based on the available data and practical requirements for operation and maintenance (van Daal *et al.*, 2017). Periodic redundant storage in the stormwater network is critical for the successful implementation of RTC, and the extent of the performance would depend on how much capacity can be made available with the optimisation of the control rules (Colas *et al.*, 2004; US EPA, 2006). The challenges that need addressing in the implementation of RTC are data accuracy and the reliability associated with continuous recording and remote transmission (Schutze *et al.*, 2004).

2.4.6 Impact of climate change

The Intergovernmental Panel on Climate Change (IPCC) has projected that average global temperatures will continue to increase with rainfall reducing over the course of the 21st century in the southwestern part of South Africa (IPCC, 2014). Schulze (2005) also showed that there would likely be shorter winter seasons (the rainfall season in the southwest region of South Africa) and a general decrease in rainfall towards the end of the 21st century. With surface water, usually stored in reservoirs, as the primary source of water in South Africa, low rainfall and high temperatures due to climate change are already resulting in widespread droughts (IPCC, 2014; Hoban *et al.*, 2015). The impact of climate change on the environment and human wellbeing linked to increasing temperature and decreasing rainfall has been documented in several studies, *e.g.* Turpie *et al.* (2002); Schulze *et al.* (2005); Mukheibir (2008); RSA (2011a, 2011b); IPCC (2014); Fisher-Jeffes (2015). Some studies (*e.g.* Hewitson *et al.*, 2005; Schulze, 2005) have shown that cold fronts in the coastal cities, *e.g.* Cape Town could mitigate the increase in temperature, but this advantage would likely not extend to the interior. The urban heat phenomena (*i.e.* greater warming of cities due to dense built-up areas) will be more severe in the interior compared with the coastal areas. The rainfall intensities towards the end of the 21st century are expected to increase by 10% to 60% at small urban hydrology scales (Willems *et al.,* 2012). This increase is likely to result in more frequent flooding. SWH schemes would temporarily store runoff to allow treatment and supply of the anticipated flood flows to help address the challenges from climate change. Well-designed SWH could also contribute to addressing the challenges of urban heat by providing water features in cities.

2.4.7 Stormwater treatment from surface water storage

Treatment of stormwater even for non-potable water purposes is essential to avoid health risks from contact (Klamerth *et al.,* 2011). Conventional water treatment systems for potable water typically include screens, settlement, coagulation, flocculation, sedimentation, filtration, disinfection and distribution (Twort *et al.*, 2000). Depending on the proposed end-uses and the pollution levels in stormwater *e.g.* hydrocarbons, nutrients, pesticides and faecal pollution (Foster *et al.*, 2002), advanced technologies may be required to make the water safe for reuse, including the following;

- Ozonation – a process where ozone gas (a product of oxygen molecules exposed to a high electrical voltage) is mixed with raw water to destroy microorganisms, degrade organic matter and other pollutants as it is a powerful oxidant (Nakada *et al.,* 2007). Similarly, advanced oxidation – a chemical treatment process where pollutants are removed from the raw water also uses oxidation reactions (Belgiorno *et al.,* 2007; Klamerth *et al.,* 2010; Radjenović et al. 2009). These treatment processes are effective in the removal of micro-pollutants, but should to be used cautiously with limited concentrations as the excessive application may lead to other toxic bi-products (Rizzo 1996);
- Membrane filtration – a treatment process where water is forced through thin layers of semi-permeable material to remove pollutants (Kimura *et al.,* 2003; Nghiem *et al.,* 2002). Membranes can be effective in removal of emerging contaminants (Nghiem *et al.,* 2002, 2005; Tambosi *et al.,* 2010), but the capital and operation costs may be prohibitive (Grassi *et al.*, 2013); and
- Adsorption – a treatment process with adhesion of gas molecules with pollutants to create a film that can be filtered out of the water (Navarro *et al.,* 2009). The treatment process is also effective in removal of emerging contaminants, are less costly than filter membranes and less likely to produce toxic by-products (Westerhoff & James, 2003; Yener *et al.*, 2008; Yu *et al.*, 2008). However, the regeneration costs of adsorption processes particularly with energy requirement and off-site transport can be prohibitive (Brown 2004).

2.5 Stormwater harvesting from groundwater storage

2.5.1 Overview

The use of groundwater storage for SWH is possible through Managed Aquifer Recharge and Recovery (MAR&R). In this approach, stormwater is temporarily stored in *e.g.* stormwater ponds adapted to function as infiltration cells with specific features for allowing the recharging of groundwater aquifers for future abstraction and supply (Dillon *et al.,* 2009; Wu *et al.,* 2012). Recharge of the groundwater aquifer can also be accomplished through the injection of surface water into specially designed boreholes. The main aim of the transfer of surface water to groundwater aquifers is to make use of large storage capacity offered and to benefit from the limited loss from evaporation (Philp *et al.,* 2008). The treatment processes associated with MAR&R *i.e.* extended retention in the ponds that allow sedimentation of suspended particles and filtration in the groundwater aquifer also provides some level of stormwater quality improvement. Further, the process of SWH results in the reduction of the runoff component in the hydrological cycle water balance (*i.e.* infiltration component is increased), thus providing additional peak flow attenuation from storm events to mitigate flood risk (Fisher-Jeffes, 2015). Although MAR&R can provide significant water quality improvement and water quantity management (both flood control and water supply), implementation usually depends on land

availability, topography (ideally flat) and geology (a suitable aquifer with porous sandy soils) (Wu *et al*., 2012; Fisher-Jeffes, 2015).

2.5.2 The Atlantis Water Resource Management Scheme

The Atlantis Water Resource Management Scheme (AWRMS) is an example of MAR&R system in Cape Town where treated domestic wastewater mixed with stormwater is infiltrated into the local groundwater aquifer for later abstraction and use (Bugan *et al.*, 2016). The AWRMS was commissioned in 1979 and is a pioneer SWH system in South Africa. A schematic of the AWRMS is as shown in Figure 2-1.

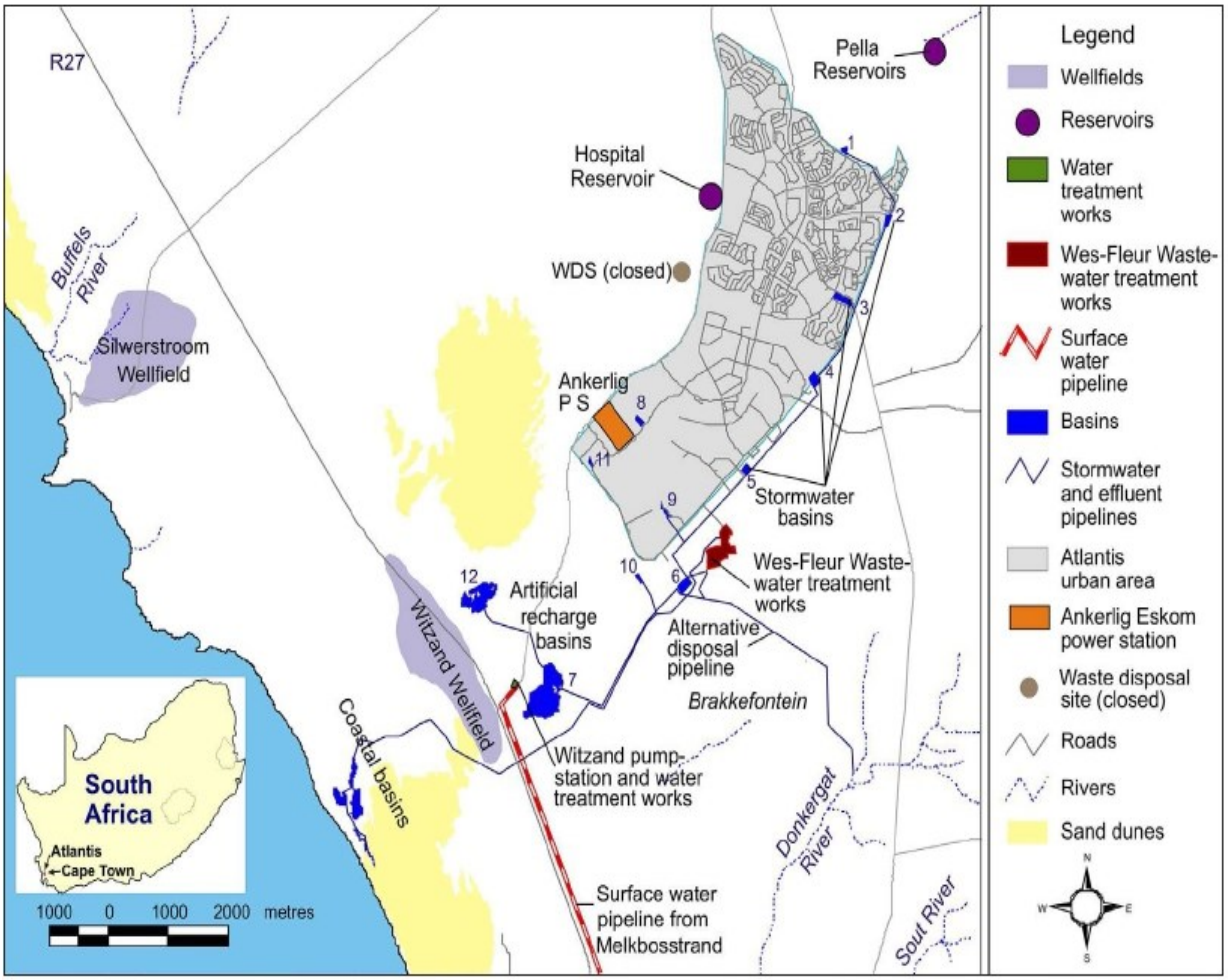


Figure 2-1 Atlantis Water Resource Management Scheme (Bugan *et al.*, 2016)

The system was established to supply water to the town of Atlantis located 50 km north of Cape Town on the west coast that was not then linked to the city reticulation system (DWA, 2010). With only about 350 – 450 mm annual rainfall, few surface water resources, and a rapidly growing population, the long-term water needs of the town could not be met by the natural

groundwater yield of the aquifer (Quayle, 2012). The AWRMS is used to augment the existing groundwater supply system with artificial recharge of stormwater and wastewater to ensure adequate water supply. After almost 40 years in operation, the AWRMS can be considered as a local time-tested SWH scheme that can be used for best practice benchmarking in making use of MAR&R, where stormwater is temporarily stored in groundwater aquifer and later recovered for reuse. In the AWRMS system, approximately 7500 m^3/day of stormwater and wastewater is recharged to boost the groundwater resource by more than 2.7 x 10^6 m^3/year *i.e.* an increase of about 30% (DWA, 2010). In a detailed groundwater model developed in 2017 to assess the performance of AWRMS system, it was determined that there was potential to increase the groundwater resource to 6 x10^6 m^3/year *i.e.* an increase of about 60% (Jovanovic *et al.*, 2017). The lessons from AWRMS with details provided in Quayle (2012) are as follows:

- **Administrative** – for such a scheme to function properly, there must be unconstrained coordination between the owners of the scheme and various departments *e.g.* bulk water supply, wastewater treatment plants, roads and stormwater management;
- **Operation** – need to avoid over-abstraction that may result in a significant drawdown of the water table that would cause seawater intrusion and disruption of the balance of the natural ecology;
- **Maintenance** – regular maintenance is essential to avoid basin clogging from the build-up of fine sediments and organic material. Furthermore, alien invasive plant species and water-thirsty plants should be controlled as they may affect groundwater and affect predictions of the sustainable yield from the aquifer;
- **Salinity** – there was a need to prevent loss of good-quality water to the sea which was achieved by 'flattening' the hydraulic gradient by introducing poor quality water (a combination of wastewater effluent from the industrial plant and the softening plant) to form a 'salt wedge' that assisted in keeping the seawater separate from the good-quality water.

The success of AWRMS provides experience on MAR&R and an excellent practical example of the potential for SWH from groundwater storage (Quayle, 2012). However, the system has not been replicated in other parts of South Africa although some provinces and metros in South Africa including in Gauteng and eThekwini Municipality (Durban) have explored the potential for similar systems to maximise sustainable use of water supply from groundwater storage (Quayle, 2012).

2.5.3 International case studies

MAR&R has been implemented in some countries worldwide including Australia (Dillon *et al.,* 2009; Page *et al.,* 2009; Miotliński *et al.,* 2014); the United States of America (USA) (Murray *et al.,* 2007) and Namibia (Murray *et al.*, 2007; Tredoux *et al.,* 2009). In Australia, MAR&R has

been implemented in Perth, Adelaide and Melbourne with aquifer storage capacities of 250 Mm^3, 80 Mm^3 and 100 Mm^3 respectively (Dillon *et al.,* 2009). Some other examples of MAR&R projects in Australia include Salisbury near Adelaide where stormwater is treated in a wetland and injected into the aquifer; and Burdekin Delta in North Queensland where 45 Mm^3 of water is recharged and abstracted for irrigation of Sugar Cane and other crops; (Dillon *et al.,* 2009; Page *et al.,* 2009; Miotliński *et al.,* 2014). Some examples of MAR&R in the USA include Peace River in Florida and the Kerrville in Texas (Murray *et al.,* 2007). The Peace River and Kerrville schemes comprise injection of treated water into the groundwater aquifer and recovery of about 68 000 m^3/day and 9500 m^3/day respectively. In Namibia, MAR&R is a water resource for the City of Windhoek with artificial recharge from the Von Bach Dam and reclaimed treated wastewater into the Auas aquifer with a yield of 2 - 8 Mm^3/year (Murray *et al.*, 2007; Tredoux *et al.,* 2009).

2.5.4 Stormwater treatment from groundwater storage

2.5.4.1 Nature-Based Solutions

Nature-Based Solutions (NBS) are approaches such as those proposed in the United Nations World Water Assessment Programme (WWAP) Report 2018 (WWAP, 2018). Unlike conventional systems where stormwater is collected and promptly conveyed away from locations of rainfall incidence (end of catchment solutions), NBS utilise local storage *e.g.* groundwater aquifers to keep stormwater in the catchment for reuse (source control solutions) (WWAP, 2018; Fletcher *et al.,* 2014). For example, NBS could manage rainfall by local storage *e.g.* via infiltration to enhance the quantity of groundwater available for human needs (WWAP, 2018). This approach would also minimise the potentially adverse effects of poor quality stormwater on receiving waters (Mitchell *et al.,* 2006; Woods-Ballard *et al.,* 2015; Armitage *et al.,* 2013). Furthermore, the method could also be implemented with a series of natural processes referred to as a 'treatment train' including components such as constructed wetlands, vegetated swales and bio-retention cells in Sustainable Drainage System (SuDS) terminology (Woods-Ballard *et al.,* 2015; Armitage *et al.,* 2013). Although the selection of technology to include in the 'treatment train' is case specific, the objective from the process is similar, *i.e.* improved stormwater quality that is similar to pre-development conditions.

A well-designed 'treatment train' would be expected to make a substantial contribution towards the improvement of stormwater quality through various mechanisms such as sedimentation, filtration, adsorption (the process whereby pollutants bind to the surface of fine sand particles), biodegradation, volatilisation (the conversion of certain compounds to gas or vapour), precipitation, plant-uptake, nitrification and photosynthesis (Armitage *et al.,* 2013). The selection of a 'treatment train' is critical as it is directly linked to high capital and operation costs that can be a deterrent and negatively impact the economic viability of the project. (Philp *et al.,* 2008; Fisher-Jeffes, 2015). Guidelines such as Woods-Ballard *et al.*, (2015) and Armitage *et al.*, (2013) assist in the identification, selection and design of a suitable 'treatment train' including providing information on the potential pollution removal. Even with the various treatment

processes available in the 'treatment train', pollution reduction in NBS systems would typically only be adequate for non-potable purposes or safe discharge to receiving streams (Woods-Ballard *et al.,* 2015; Armitage *et al.,* 2013). Additional treatment would be required to use the harvested stormwater for potable water purposes.

2.5.4.2 Constructed treatment systems

In the case where potable water is to be supplied from groundwater storage, additional treatment would be required to provide water of reliable quality. Depending on the level of contamination of the abstracted groundwater, the treatment may include additional disinfection to remove persistent pathogens or advanced approaches such as discussed in Section 2.4.7. Disinfection processes include chlorination, ultraviolet (UV) radiation, oxidation, and/or membrane filtration (Philp *et al.,* 2008; Mitchell *et al.,* 2007). The selection of the additional treatment is critical as the associated high capital and operating costs are considered a primary cause of the limited uptake of SWH systems (Philp *et al.*, 2008; Fisher-Jeffes, 2015).

2.6 Potential demand for harvested stormwater

2.6.1 Overview

In many studies such as Mitchell *et al.*, (2007), Goonrey *et al.*, (2009), Fisher-Jeffes, (2015); Rohrer, (2017), and Rohrer & Armitage, (2017), harvested stormwater has been restricted to non-potable uses with envisaged low human health risk from contact and ingestion. The proposed uses included *inter alia* irrigation and other non-potable domestic water demands such as toilet flushing and washing machine. The determination of appropriate non-potable demand to supply with stormwater should consider various factors such as water quality at the source, distribution requirement, and end-use (Buchmiller *et al.,* 2000). The likely non-potable water demands and potential for stormwater reuse is discussed as follows.

2.6.2 Urban agriculture

About 70% of the available global freshwater resource is used for agricultural production (Prathapar, 2000). As shown in Figure 2-2, per capita annual water availability of most African countries will be less than 1000 m^3 by 2025. With growing water scarcity and the increasing need to prioritise potable domestic demand, water allocated to agricultural use will inevitably be reduced or alternative water identified as a supplementary resource (Rijsberman, 2006). For example, in West Asia and North African countries where about 75% of freshwater is used for agriculture, much of the water has since been re-allocated to other sectors due to rapid industrialisation, urbanisation and high population (Oweis & Hachum, 2006). Further, the mean annual per capita renewable water in the West Asia and North African countries has reduced from 3500 m^3 in 1960 to below 1500 m^3 and is expected to decrease to less than 700 m^3 by the year 2025 (Oweis & Hachum, 2006).

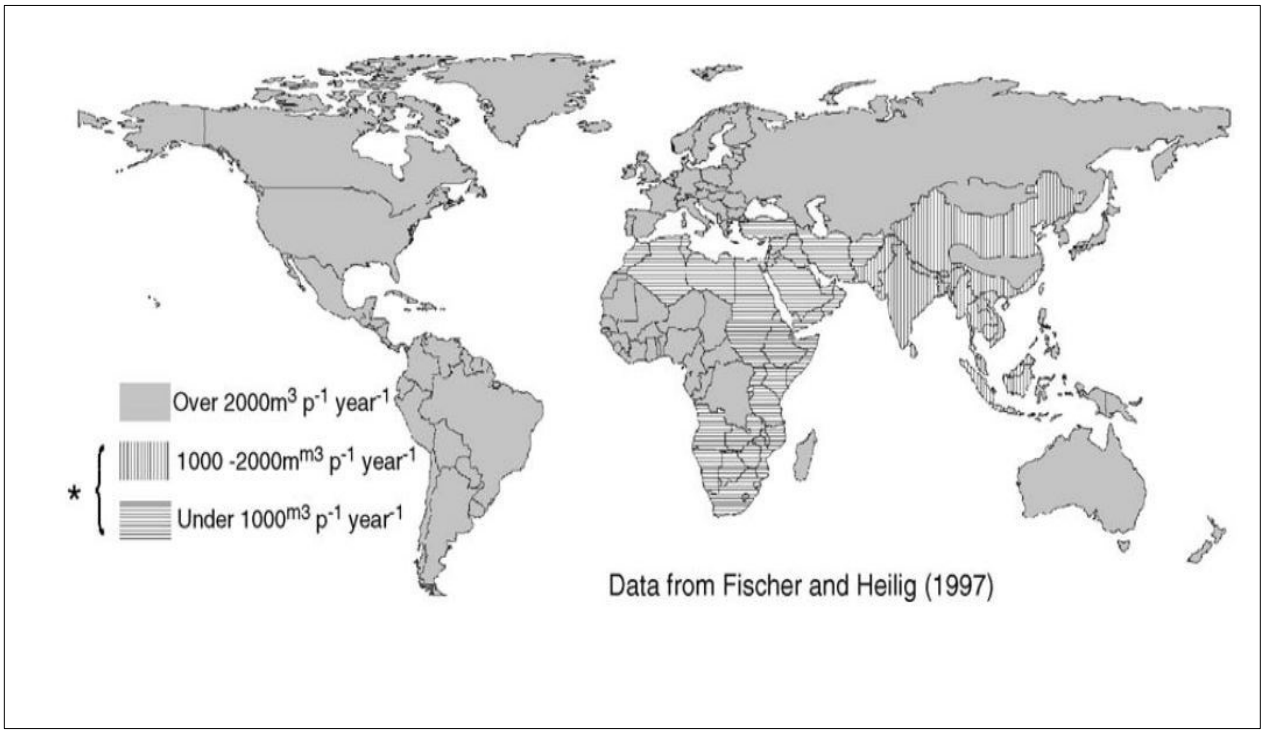


Figure 2-2 Global mean annual per capita renewable water by 2030
(Oweis & Hachum, 2006).

Water scarcity will continue to be a limiting factor for the agricultural sector in the 21st century. The constraining of water to the agriculture sector to provide for other sectors such as domestic, commercial and industry will undoubtedly affect food production, especially in developing countries such as South Africa (Rijsberman, 2006). With limited water resources and increasing demands from various sectors, alternative sources such as stormwater will be required to provide water to the more than one billion people that will be affected by absolute water scarcity – *i.e.* renewable water supplies that are below 500 m^3/capita/year (Rijsberman, 2006). SWH as an alternative water resource could provide for sustainable agricultural production where irrigation increases crop yields. With increasing water scarcity and constraints on future water resources, stormwater reuse will inevitably become a viable option for water supply especially for urban agriculture (Yim *et al.,* 2007).

The use of stormwater for agricultural purposes may be beneficial as it would minimise the competition for limited surface water resources, and the possible presence of nutrients would decrease the required amount of fertilisers (Candela *et al.,* 2007). Various countries have developed guidelines that provide water quality requirements for irrigation purposes. Some of these measures have been summarised in various studies such as Oweis & Hachum (2006), Pedrero *et al.* (2010), and Christou *et al.* (2016). The determination of agricultural water demand is linked to the Crop Water Requirement (CWR), which is largely the water required to meet the crop evapotranspiration (ETo) needs (Allen *et al.*, 2006). Various approaches have been

developed and used for the estimation of ETo. The American Society of Civil Engineers (Jensen *et al.,* 1990) and European Community (Choisnel *et al.,* 1992) evaluated the various procedures under different climatic conditions. The studies confirmed that the Penman-Monteith method was most suitable in both arid and humid climates compared with other empirical methods such as Blaney-Criddle and Hargreaves (Jensen *et al.,* 1990; Choisnel *et al.,* 1992). Based on these findings, the Food and Agricultural Organisation (FAO) developed the modified FAO Penman-Monteith method that has since been widely used in the design and management of irrigation systems (Allen *et al.,* 2006). To enable faster computation of CWR, FAO developed the CROPWAT software based on climatic data and soil properties of over 100 countries compiled by the FAO Agrometeorological Group (Smith, 1992).

2.6.3 Irrigation of residential gardens and public open spaces

The increasing competition for the limited fresh water in dry and water-stressed areas has influenced the consideration of non-traditional water sources, *e.g.* stormwater as an option for non-potable demands such as irrigation of residential gardens and open parks (Mesa-Jurado *et al.,* 2012; Milano *et al.,* 2012). The non-traditional water sources may provide valuable nutrients, and risks associated with reuse for residential and open parks may be lower than in agriculture *i.e.* no entrance of pollution into the food chain (Xu *et al.,* 2010; Rajaganapathy *et al.,* 2011). In Cape Town, South Africa properties with residential gardens typically use 20-40% of the domestic water demand for garden irrigation (Jacobs & Haarhoff 2004). With increasing water supply restrictions in Cape Town and other cities worldwide, *e.g.* in Australia (Begum & Rasul, 2009), the capture and use of stormwater for residential garden irrigation and public open parks may be a reasonable and realistic way of minimising water demand from municipal systems as these uses do not generally require potable water (Seymour, 2005). Currently, most SWH systems are designed and installed for non-potable water such as irrigating public areas, golf courses, agriculture and industrial uses (Hatt *et al.,* 2006; Benetti *et al.,* 2008). In Queensland, Australia, SWH and reuse have already been accepted as an alternative to potable water for demands such as garden irrigation (Mitchell *et al.,* 2006).

2.6.4 Domestic non-potable indoor water demand

A water resource such as stormwater could be suitable for domestic non-potable indoor water needs with envisaged minimal risk to human health (McArdle *et al.*, 2011). The two commonly used measures to determine human health risk to pollution is 'Acceptable Annual Infection Risk Level' (USEPA, 2005), and 'Acceptable Disability-Adjusted Life Years' (DALYs) (WHO, 2008). In a study by Lim *et al.*, (2015), the findings show that toilet flushing with low impact development (LID) treated stormwater were below the USEPA annual risk benchmark of $\leq 10^{-4}$ pppy and within the WHO recommended disease burdens of $\leq 10^{-6}$ DALYs pppy. For potable water use, the level of treatment would depend on the application and extent of pollution present. The National Strategy for Water Reuse in South Africa recommends treatment of water contaminated by high microbial pollution to include membrane filtration, chemical disinfection

(chlorine and bromine compounds) and Ultra Violet (UV) radiation (DWA, 2011). Chlorination would be necessary for indoor water use such as toilet flushing and machine washing as a precautionary measure. Since stormwater for non-potable water use would be of lower quality than potable water, its distribution would require the installation of a dual-reticulation system (Wu *et al.*, 2012). Hunter Water Corporation (2003) determined that water reticulation installation in an already built-up area could cost up to 2.5 times more than similar works in new developments. On the other hand, Wu *et al.* (2012) noted that treatment of the stormwater to potable standards would eliminate the need for dual reticulation as existing pipe networks would be used for the distribution. The cost of treatment to potable water standards would depend on the pollution load but would typically be higher than the traditional sources (Wu *et al.,* 2012). An economic analysis would be required to determine the most appropriate approach *i.e.* full or partial treatment to potable or non-potable water standards and distribution with existing municipal or dual reticulation system respectively. The following methods are available to estimate domestic water demand (Rinaudo, 2015).

- **Time dependant extrapolation method –** This method is suitable for use with time series as it utilises growth rates for projection of future demands based on previous circumstances. The disadvantage of the method is that the estimated values are usually affected by the quality and reliability of the recorded data. For example, water use billing records and demographic data may contain inconsistencies and outliers that may result in projection errors.
- **Unit water demand analysis –** in this method, water use is estimated as the product of the per capita water demand and the number of users. The approach is pragmatic as it considers the actual number of users as the primary parameter in the estimation of water use. The method accounts for site-specific characteristics and is suitable for estimation of approximate values required in preliminary design or feasibility studies.
- **Multivariate statistical models –** in this method, statistical relationships are defined to link per capita water demand (the dependent variable) with variables that influence water use, *e.g.* household income, economic activity (*e.g.* employment status), and housing characteristics (*e.g.* household size, dwelling type). The method estimates water use based on anticipated changes in variables that correspond with historical observations.

Availability of reliable data and suitable models are required to reasonably estimate domestic water demand under various conditions, *e.g.* population growth, ecological needs and climate change (Roy *et al.*, 2012). To minimise capital and operational costs, reasonably accurate demand estimation approaches, *i.e.* short-term (for operation and management), and long-term (for planning and infrastructure design) are required (Bougadis *et al.,* 2005). Long-term estimates typically consider estimated population growth, changes in land use and climate (Bougadis *et al.,* 2005). However, some researchers (*e.g.* Milly *et al.,* 2008; Gober *et al.,* 2010) indicate that population growth, changes in land use and climate may introduce uncertainties that may limit the accuracy, *e.g.* historical trends may vary significantly from the present. Also, various

temporal-spatial drivers determine where, how, when and why water is used (Wu *et al.*, 2012). The drivers may range from human behaviour and attitudes towards the water to general factors, *e.g.* property size, people in a household, affluence and climate (Fisher-Jeffes, 2015). Tools such as Institute for Water Resources – Municipal and Industrial Needs (IWR-MAIN) that combine spatial-temporal data have been developed and used extensively in the USA (Wurbs 1994; Bauman & Boland 1998), and globally (Mohamed & Al-Mualla 2010). With the advent of GIS software, some tools have been developed and used by water utility agencies, *e.g.* UK Environment Agency and California Bay-Delta Authority (Davis, 2003). Nonetheless, integrating diverse spatially varying population demographics, land use and climate into a single model remains difficult (Galán *et al.,* 2009). Various parameters with the capacity to affect domestic water use have been investigated for South African cities. The parameters included population demographics, *i.e.* household size, income, climate – particularly prolonged high temperatures, land use and stand area (Stephenson and Turner, 1996). Some studies undertaken in Pretoria (Van Vuuren & Van Beek, 1997) and Cape Town (Jacobs & Haarhoff, 2004) investigated the likely effect of population demographics in the split between indoor and outdoor water use. Guidance to domestic water demand estimation in South Africa at the time of writing the report in 2018 was provided by the 'Red Book' (CSIR, 2005) with design guidelines for municipal water-demand very similar to the original version referred to as the 'Blue Book' published in 1983 (DCD, 1983). Since the guidelines are almost 40 years old, the design considerations need revision as some key factors such as household size, household income, the affluence of the area, employment status, season of the year, the day of the week *etc.* as highlighted in Roberts (2005) and Heinrich (2006) were not considered (Van Zyl *et al.,* 2008).

2.6.5 Approaches for stormwater harvesting and distribution

Two main approaches are available for the supply of both potable and non-potable water, *i.e.* centralised – a single abstraction location with a water distribution network covering the entire study area; and decentralised – several abstraction locations with distribution networks covering sections of the study area (Hatt *et al.,* 2004; Mitchell *et al.,* 2007; Philp *et al.* 2008; Fisher-Jeffes, 2015; Rohrer, 2017). The centralised system with a source, intake, treatment and extended distribution network has been the primary approach of water supply in urban areas for quite some time. The decentralised approach is typically considered as supplementary and limited to non-traditional sources such as greywater recycling and SWH harvesting (Philp *et al.* 2008). Centralised systems are typically associated with large-scale potable water supply, while decentralised systems are usually limited to non-potable water systems for *e.g.* irrigation of agriculture, golf courses and public open spaces (Hatt *et al.,* 2004; Mitchell *et al.,* 2007). The supply of non-potable water generally requires dual reticulation with each property provided with two connections, *i.e.* potable water from the municipal mains and non-potable water for demands that accept water of lower quality. Such systems are common in Australia where non-potable water is used for irrigation typically limited to catchments smaller than 200 ha. (Mitchell *et al.,* 2007; Philp *et al.* 2008).

2.6.6 The challenges associated with stormwater reuse

The main challenge to widespread uptake of SWH as potable water is the high cost of treatment compared to freshwater sources such as lakes and rivers (Philp *et al.*, 2008). The main challenge with regards to non-potable water – particularly irrigation – is the mismatch of supply and demand especially in regions that experience seasonal rainfall (Hatt *et al.*, 2004). The issue of reliability could be addressed with the provision of storage in the stormwater management infrastructure. However, the availability of land to provide adequate storage is typically limited in urban areas. Furthermore, acceptance of stormwater reuse can be affected by people's perception that the treatment processes will not provide safe water from the highly polluted stormwater (Hatt *et al.,* 2004). No research was identified addressing the issue of people's perception of stormwater reuse in South Africa. However, wastewater could be used as a proxy since the water quality in some drainage channels of Cape Town *e.g.* Lotus River and Black River is not that much better. There does not appear to be any religion or religious values that hinder the reuse of wastewater (Wilson & Pfaff 2008). On the contrary, religious views that tend to emphasise responsibility to the environment and sustainability could support stormwater reuse as they could readily be implemented in an equitable and just manner with costs equitably distributed; and environmental issues are taken into account (Fisher-Jeffes, 2015). In another study, Ilemobade *et al.* (2009) showed that 94% of respondents supported reuse of wastewater during a drought, however, only 36% were willing to use the water themselves. Since water quality is the main factor influencing perception, behaviour change around indiscriminate disposal of contaminants into stormwater systems would go a long way in minimising pollution, thus reducing treatment costs and increasing confidence in the resource. In general, design and implementation of alternative water resources such as SWH need to be undertaken cautiously to minimise the risk of failure since people's perception of the system capacity and performance are already very low. A single or few high-profile failures that put public health or the environment at risk would severely undermine public confidence in the system acceptance and future use of the approach (Hatt *et al.,* 2004).

2.7 Economic analysis

2.7.1 Overview of costs

In the traditional economic analysis for water systems, the costs are attributed to the construction, operation and maintenance of the system whilst the benefits are typically limited to the level of service provided. The estimated benefit: cost values of various competing systems are compared with each other to determine the most viable option (Roebuck, 2007). Non-conventional water supply systems such as SWH are most likely to be viable only after consideration of additional benefits, *e.g.* amenity and biodiversity or where conventional resources are severely constrained (Fletcher *et al.*, 2004; Hatt *et al.*, 2004; Philp *et al.,* 2008). Various studies (*e.g.* DECNSW, 2006; Roebuck, 2007; Philp *et al.*, 2008; Dobes & Bennett, 2009; Fisher-Jeffes, 2015; Rohrer, 2017) have undertaken economic analysis approaches that were suitable for SWH systems *i.e.* where

they considered costs and a more extensive range of benefits *e.g.* flood control, amenity and biodiversity. An overview of some economic analysis approaches follows:

2.7.2 Cost analysis components

The standard unit costs for components of the water supply system are usually readily available (*e.g.* DoCOGTA, 2010; Swartz *et al.*, 2013) and typically consider all the costs associated with the life of a project (Veefkind, 2002). A brief description of the various costs are as follows:

- **Capital costs** – the capital costs comprise all the costs associated with the installation of the system components including *inter alia* land acquisition, planning and feasibility studies, architectural and engineering design, construction (materials, equipment and labour), equipment and furnishings not included in construction, inspection and testing (ADB, 2017). It is essential that as far as possible, all significant costs are identified and included in the valuation to minimise errors (DoCOGTA, 2010). The components that cannot be reasonably estimated until construction commences or require detailed studies, *e.g.* rock excavations need to be adequately provided for in the valuation as provisional sums (ADB, 2017).
- **Operation and maintenance (O&M) costs** – the operational and maintenance costs comprise all the costs that are associated with the management of the system components to adequately deliver the intended outputs (DoCOGTA, 2010). The system cost linked to the O&M costs would typically include *inter alia* rented land (where applicable), operating staff, energy, labour and material for maintenance and repairs, planned periodic renovations, insurance and taxes, financing costs, utilities, and other owner-related expenses (DoCOGTA, 2010). In some preliminary estimations, the O&M costs can be represented as a percentage of the capital costs. Alternatively, estimates from similar existing projects can be used as an initial approximation of the O&M costs (ADB, 2017).

Some approaches for undertaking an economic analysis of water supply systems are provided as follows:

2.7.3 Cost-Effectiveness Analysis (CEA)

Cost-Effectiveness Analysis (CEA) is an approach where the capital, operation and maintenance costs of alternative projects with similar outputs are compared (Roebuck, 2007). CEA is the most common approach used for the analysis of government projects where any differences in project outputs are compared subjectively with the variation in costs (Dodgson *et al.*, 2009). In CEA, all the costs and benefits are linked to a simple single attribute, *e.g.* kilolitres of water, upon which all comparisons are considered for the various project options (Dolan & Edlin, 2002; Fisher-Jeffes, 2015). CEA is typically valuable where costs and benefits are generally similar. Such an approach would provide a limited appraisal for SWH systems as it may not include some benefits that cannot be aggregated into a single attribute, *e.g.* water quality improvement and biodiversity preservation.

2.7.4 Benefit Cost-Analysis (BCA)

In the Benefit-Cost Analysis (BCA) approach, the costs and benefits of various water supply systems are estimated in monetary terms and compared to determine the most feasible project (Dodgson *et al.*, 2009; Dolan & Edlin, 2002). Where the benefits cannot be easily quantified, the circumstances of a community are assessed with and without the water supply system (Dodgson *et al.*, 2009). The limitation of BCA is where some inputs and outputs cannot be explicitly valued in monetary terms. Additionally, the use of BCA is further constrained by lack of quantitative data on most of the benefits associated with SWH (Hatt *et al.*, 2004; Philp *et al.*, 2008; Goonrey *et al.*, 2009; Akram *et al.*, 2014).

2.7.5 Multiple Criteria Analysis (MCA)

Multiple Criteria Analysis (MCA) is a technique typically applied where it is impractical to allocate attributes of similar nature *i.e.* does not only consider a single attribute, *e.g.* allocation of monetary values to the various inputs and outputs (ADB, 2017; Dodgson *et al.*, 2009; Fisher-Jeffes, 2015). In the MCA method, the various inputs and outputs of a project are evaluated against predetermined criteria where components are compared without giving all of them monetary values (Boshoff *et al.*, 2009). Typically, the costs are estimated in monetary terms, but, social and environmental benefits that cannot be directly quantified are assessed in qualitative terms (Dodgson *et al.*, 2009). However, the use of MCA may require experience with the method especially for the valuation of unquantifiable elements (Dodgson *et al.*, 2009).

2.7.6 Life-Cycle Costing Analysis (LCCA)

Life-Cycle Costing Analysis (LCCA) is an approach that considers all relevant costs and revenues related to construction, operation and maintenance of a water supply system for the entire lifetime of the assets (Clift & Bourke, 1999). LCCA is typically applied in association with other approaches and is aimed at determining the costs and benefits over the entire life cycle of a project (Lampe *et al.,* 2005; Fisher-Jeffes, 2015). Various researchers (*e.g.* Lampe *et al.,* 2005; DECNSW, 2006; Roebuck, 2007; Philp *et al.,* 2008; Fisher-Jeffes, 2015) note that costs typically include project activities such as land acquisition, construction, operation, inspection, corrective measures, and disposal. According to Lampe *et al.* (2005), LCCA can also be used as an objective economic analysis method independent of the other approaches. In the independent application, LCCA is used as a common sense concept that 'time is money,' *i.e.* through placing a time value on the money where future expenditures are brought back to a present base year to allow a direct comparison between alternatives (Van Vuuren & Van Dijk, 2006). There are several methods used for economic analysis and evaluating investments in engineering including *inter alia* Net Present Value (NPV), Equivalent Annual Worth (EAW), Internal Rate of Return (IRR), External Rate of Return (ERR), profitability index, payback period, cost-effective methods, capital recovery with return, capitalized equivalent *etc*. (ADB, 2017).

2.7.7 Overview of benefits

Unlike costs that are relatively easy to estimate, the valuation of the benefits is complex. Some indirect approaches have been applied in some studies, *e.g.* de Wit *et al*. (2009) and Fisher-Jeffes (2015) to determine benefits such as water quality improvement in wetlands and the amenity provided from well-maintained stormwater ponds. For example, the Contingent Valuation Method (CVM) has been used to assess benefits based on the willingness of a community to pay for a change in the quality or quantity of an environmental good or service. Alternatively, the Cost of Replacement (CoR) has been used to compare the cost of developing the system on an alternative location (*e.g.* Lampe *et al.*, 2005; Fisher-Jeffes, 2015; ADB, 2017).

2.7.8 Reduction of water demand from municipal systems

Various studies (*e.g.* Roebuck, 2007; Maheepala *et al.*, 2011; Neumann *et al.*, 2011; Fisher-Jeffes, 2015) have shown that SWH can significantly reduce potable water demand from the existing municipal water supply system. The benefit of such reduction in demand is a postponement of the need to provide additional capacity or construction of a new water supply system, and such a delay in investment can have significant economic value (Fisher-Jeffes, 2015). The cost of water from SWH has also been determined to be relatively lower than other options such as rainwater harvesting, sea water and water supply from long distance pipelines (Marsden & Pickering, 2006; Hatt *et al.*, 2006). However, the reliability of the SWH system which is a function of storage capacity and the local rainfall regime would rely on municipal water to supply demands during long dry periods. This would have economic implications for the water supply system, depending on how charging for stormwater provision and for the conventional system are organised (Fisher-Jeffes, 2015).

2.7.9 Flood mitigation and management

Various studies (*e.g.* Woods-Ballard *et al.,* 2015; Fletcher *et al.,* 2008, 2013; Huang *et al.,* 2009; Fisher-Jeffes, 2015) have indicated that SWH can mitigate floods through flow peak attenuation and reduction of runoff volumes. For example, SWH case studies in Australia have shown peak reductions of around 5 – 10% for the 100-year recurrence interval event (Hatt *et al.*, 2006; Fletcher *et al.*, 2008). A local case study in the Liesbeek Catchment in Cape Town, South Africa showed that SWH would attenuate the peak flows of mainly small and frequent storms (Fisher-Jeffes, 2015). However, the study also indicated that results might not be directly transferable to other locations and specific studies needed to be undertaken to determine catchment and regional specific benefits including further peak flow reduction from MAR&R (Fisher-Jeffes, 2015).

2.7.10 Water quality benefits and biodiversity preservation

Conventionally, stormwater management has mainly focused on the efficient removal of rainwater from locations for flood control to minimise 'inconvenience' and has mostly ignored water quality issues (Armitage *et al.*, 2013). In the process of conveyance, traditional stormwater management systems collect and transfer litter, silt, pathogens, hydrocarbons, heavy metals and other forms of pollution to downstream locations, severely contaminating receiving water bodies, negatively impacting biodiversity and amenity in most urban areas including Cape Town (CCT, 2005; Brown & Magoba, 2009; Haskins, 2012). Various studies (*e.g.* Mitchell *et al.*, 2005; Wong *et al.*, 2012) have shown that the processes associated with SWH have the potential to reduce pollution associated with runoff to levels comparable with predevelopment conditions. This has a positive impact on ecosystem health and contributes to biodiversity preservation.

2.7.11 Local amenity

Stormwater ponds designed according to SuDS philosophy and adapted for purposes such as SWH can provide various amenities such as a pleasant ambience, aesthetics and recreational spaces that can provide a sense of serenity and good living to the community (Haddock, 2004; Woods-Ballard *et al.,* 2015; Armitage *et al.,* 2013). There is also evidence that such landscape designed with SuDS philosophy can provide an economic benefit by increasing the selling price of nearby properties by 10% to 25% (USEPA, 1995; Dinovo, 1995).

2.8 Social issues linked to stormwater harvesting and reuse

2.8.1 Social acceptance

Non-conventional water supply approaches such as SWH are usually associated with poor water quality and perceived to be prone to failure due to limited management experience (Hatt *et al.*, 2006; Mitchell *et al.,* 2007; Philp *et al.* 2008). Thus, developers and city authorities are often reluctant to adopt such approaches to augment water supply on a large scale. However, with constrained freshwater sources and increasing water scarcity, alternative water resources such as stormwater are now actively being considered and used for water supply (Philp *et al.* 2008). Decentralised small-scale SWH systems are becoming more commonplace in urban developments in Australia mainly for non-potable water uses (Philp *et al.* 2008). In South Africa, no-one appears to have investigated community views on SWH. Since the water quality of stormwater and wastewater is very similar in many drainage channels in Cape Town, studies on wastewater reuse could be used as a proxy. Wastewater reuse has successfully been implemented in some areas, *e.g.* Australia (Po *et al.,* 2003, 2005), Namibia (Murray *et al.*, 2007; Tredoux *et al.,* 2009), Israel (Friedler *et al.,* 2006), Jordan (Al-Jayyousi, 2004), Spain (March *et al.,* 2004), and some parts of South Africa (CCT, 2007). Other countries that have also implemented wastewater reuse systems to supplement potable water supplies due to constrained water resource

from population growth include China (Junying *et al*., 2004), Japan (Dixon *et al*., 1999), Germany (Nolde, 1999), United Kingdom (Jimenez and Asano, 2008) and the USA (Okun, 1996). As with wastewater, perception could be the key challenge to broader SWH adoption, however, the public view would likely improve positively towards acceptance with sensitisation of people in water-scarce areas (*e.g.* Po *et al.,* 2003, 2005). In the process of sensitisation, the beneficiaries need to be involved in the initial stages of planning and feasibility study. Various studies (*e.g.* Coombes & Mitchell, 2006; Dobbie *et al.,* 2012; Wu *et al.,* 2012; Ilemobade *et al.*, 2009) have all shown that communities do accept the alternative water sources with limited treatment for non-potable water uses.

2.8.2 Public health and safety

For SWH to gain public confidence and acceptance, the system should be set up such that there is the minimal likelihood of failure and very low health risk from the use of the water (Ilemobade *et al.*, 2009). Various public health risks and safety issues have been highlighted, *e.g.* safety of children from exposure to the non-potable water, and the risk related to the consumption of fruits and vegetables irrigated with non-potable water (Friedler *et al.,* 2006). Other public health and safety issues related to open water surfaces, *e.g.* in a stormwater pond include *inter alia* flooding from failing embankments, the potential risk for drowning and mosquitos breeding in stagnant water (DECNSW, 2006; NRMMC *et al.*, 2008; Fisher-Jeffes, 2015). These can often be managed through reshaping embankment slopes, limiting access or allowing adequate water depth beyond levels for breeding of mosquitoes.

2.9 Summary

Water resources management around the world is rapidly changing due to water scarcity that is being exacerbated by the growing demands from a rapidly growing population, rising standards of living and climate change. The impact of climate change, especially in areas with a predicted decrease in rainfall and increase in temperature, will likely further affect the availability and reliability of water thus continue to influence change in management. Non-traditional sources such as stormwater are being considered as a means of alleviating the impact of water scarcity (Wong, 2011; Fletcher *et al.*, 2004). The main changes identified include *inter alia* a shift away from sole reliance on traditional sources, *i.e.* fresh surface and a growing emphasis on environmental and ecological consideration (Wong, 2011). To sustainably provide water to meet the increasing demands, new methods are required that do not need the construction of new systems or large-scale water transfers from one region to another (Gleick, 2000). Although SWH is not widely practised (Wilson & Pfaff, 2008; Ilemobade *et al.* 2009; Akram *et al.*, 2014), various studies (*e.g.* Hatt *et al.*, 2006; Philp *et al.*, 2008) have shown that future water scarcity will significantly influence and drive the shift in the way people think about water reuse. SWH is an attractive proposition compared with other options such as waste-water reuse, desalination, and the expansion of existing reservoir capacity and the importation of water from resources remote

areas (Marsden & Pickering, 2006). Additional benefits of SWH are the avoidance of high energy requirement of desalination from an already struggling energy sector, and the mitigating impacts of climate change through the reduction of the so-called 'heat-island' effect (Wong, 2011). Some local studies such as 'Viability of rainwater and SWH in the Liesbeek River Catchment of Cape Town' (Fisher-Jeffes, 2015); 'The viability of using the stormwater ponds on the Diep River in the Constantia Valley for stormwater harvesting' (Rohrer, 2017); and 'Managed Aquifer Recharge (MAR) for the management of stormwater on the Cape Flats' (Mauck, 2017) all suggest that SWH is indeed a potentially viable water resource and that there are major opportunities for local groundwater storage in the Cape Town.

3. Method

3.1 Overview

In this study, the prospects for stormwater harvesting (SWH) utilising surface and groundwater storage was investigated in the Zeekoe Catchment of Cape Town, South Africa. The various storage options available in the Zeekoe Catchment were explored to determine the opportunity for enhancing SWH as a water resource for potable or non-potable water demand in the study area or transfer of the water to an existing Water Treatment Plant (WTP). An overview of the method is presented in Figure 3-1.

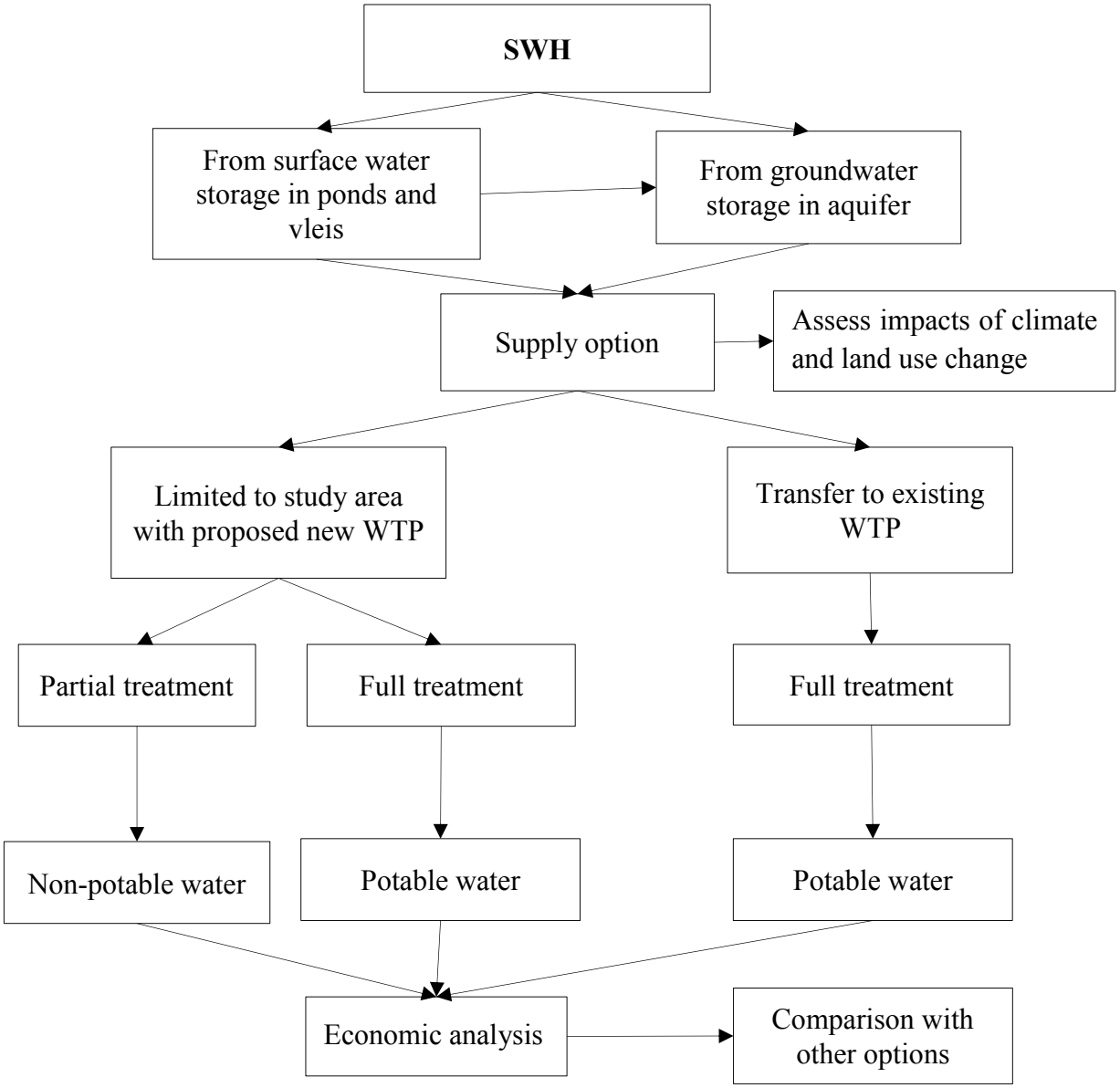


Figure 3-1 Summary of the components considered in the study

This Chapter consists of ten sections including: an overview and statement of the method, discussion of criteria used in the selection of the study area, the suitability and characteristics of the study area, description of the available data, selection and use of the surface and groundwater models and a summary of the method.

3.2 Statement of the method

The study investigated two SWH options, *viz*. directly from surface storage or indirectly via groundwater aquifers. The assessment of the prospects for SWH from the surface water storage included modelling the hydrological process to estimate the quantity of the stormwater resource, identification of the appropriate volumetric capacity and constraints, the effectiveness of Real-Time Control (RTC) to address the challenges of storage; and the impact of climate and land use change. In the case of SWH from groundwater storage, the first step included the use of a model to assess the available opportunity for the surface to groundwater transfer (Managed Aquifer Recharge) and estimation of recharge volumes. The second step was modelling groundwater abstraction (groundwater recovery after Managed Aquifer Recharge) initially in a trial section (1.5 km^2 with a single pond) and finally at a catchment scale (89 km^2 with 61 ponds). The other issues that were investigated included the identification of the appropriate demand to be supplied (potable or non-potable), the extent of volumetric reliability, and all the costs (capital, operation and maintenance) and benefits associated with SWH and supply. In essence, the study aimed to determine the potential for water supply from SWH at a regional scale and identification of areas where the water would be economically used. Initially, the study assessed the potential to utilise stormwater for non-potable water needs such as agriculture, irrigation of residential gardens and open parks, and toilets. Then, assessments were made of the opportunity and cost of the stormwater treated to potable standards and distributed locally in the study area or transfer of partially treated water (to non-potable levels) for blending with raw water at an existing water treatment works. The total costs of production and supply of stormwater as potable and non-potable water were determined and compared with other sources such as treated effluent and seawater desalination which CCT is considering implementing to mitigate the impact of water scarcity.

3.3 Selection of the study area

In the selection of a suitable catchment to be used in the study, the two main considerations included the availability of the storage needed for the economic exploitation of SWH and the availability of data to model the hydrological processes. A preliminary investigation was thus undertaken to determine the availability of storage opportunities (*i.e.* surface and groundwater aquifers) and the associated available data (mainly rainfall and flow data). A study linked to this research identified and categorised the available stormwater ponds in the various catchments of Cape Town. It established that 70% of the ponds were detention ponds, 23% retention ponds, and 7% wetlands – distributed as shown in Figure 3-2 (Rohrer, 2014). The high percentage of

detention ponds was expected as stormwater ponds are conventionally designed for flood control. The study additionally established that 68% of stormwater ponds could attenuate a 20-year flood, 54% were heavily impacted by litter, presenting 'negative amenity' and 74% did not provide for biodiversity preservation (Rohrer, 2014).

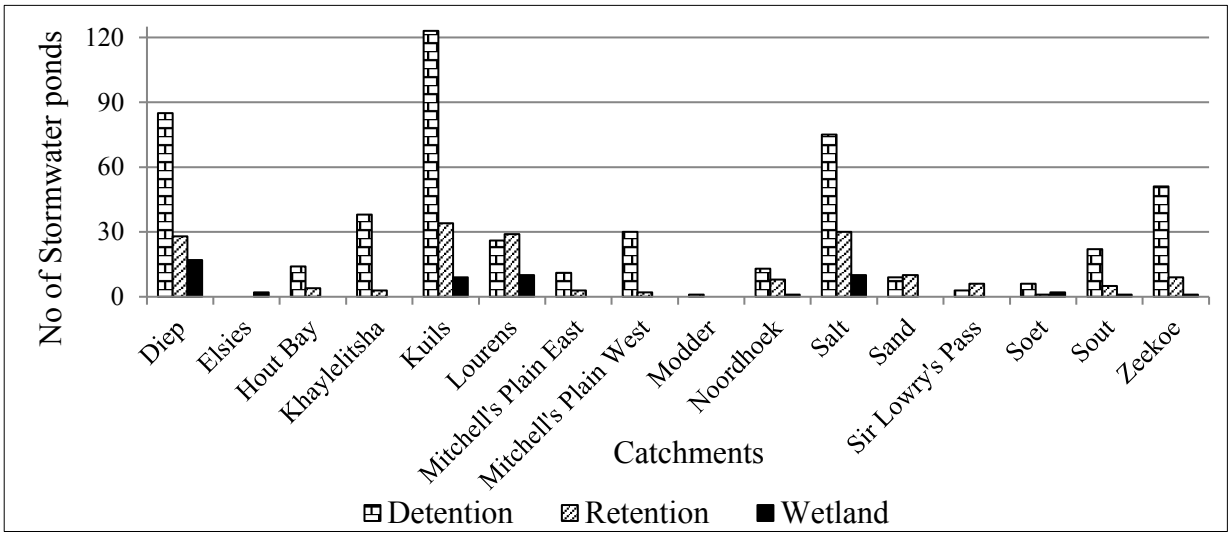


Figure 3-2 Stormwater ponds per catchment (Rohrer, 2014)

Interestingly, 41% of the detention ponds and 51% of the entire stormwater ponds had some multi-functionality in particular for recreational activities or with water features to enhance local community amenity. Some of the stormwater ponds are presented in Figure 3-3 to 3-8.

Figure 3-3 A typical dry detention pond

Figure 3-4 A detention pond with recreational facilities and car parks

Figure 3-5 A typical retention pond

Figure 3-6 A retention pond providing ambience and affluence to an area

Figure 3-7 A vegetated constructed wetland

Figure 3-8 A constructed wetland providing ecology and ambience to an area

The study further noted that stormwater ponds were concentrated in areas where there are large numbers of informal settlements (shanty towns). This highlighted the vulnerability of dry ponds to invasion by poor people looking for vacant urban land. Informal settlements are also associated with poor waste collection services. The pollution generated has a direct negative impact on stormwater quality. The summary of the factors considered in the selection of a suitable catchment are as follows:

i. Open water bodies such as vleis (shallow lakes) and stormwater ponds with potential for adaption to store and supply stormwater;

ii. Availability of good-quality data to model the hydrological process in the catchment;

iii. A catchment with characteristics such as unconfined aquifer with high porosity, hydraulic conductivity and groundwater yield (potential ground water source) that provide opportunities for surface to groundwater transfer;

iv. Proximity to potential stormwater users, *e.g.* agriculture, residential and public parks; and

v. Proximity to an existing Water Treatment Plant (WTP) to minimise the cost of conveyance for treatment of the stormwater to potable there.

Most catchments were unsuitable as there was inadequate flow data to enable the hydrological model set-up and calibration which was essential for such a desktop study. Furthermore, some catchments had limited opportunity for surface to groundwater transfer due to steep slopes while others possessed inadequate surface water storage opportunities.

3.4 Suitability of the Zeekoe Catchment as a study area

The City of Cape Town (CCT) is situated in the south-western tip of Africa whilst the Zeekoe Catchment is located in the south-central part of CCT (Figure 3-9).

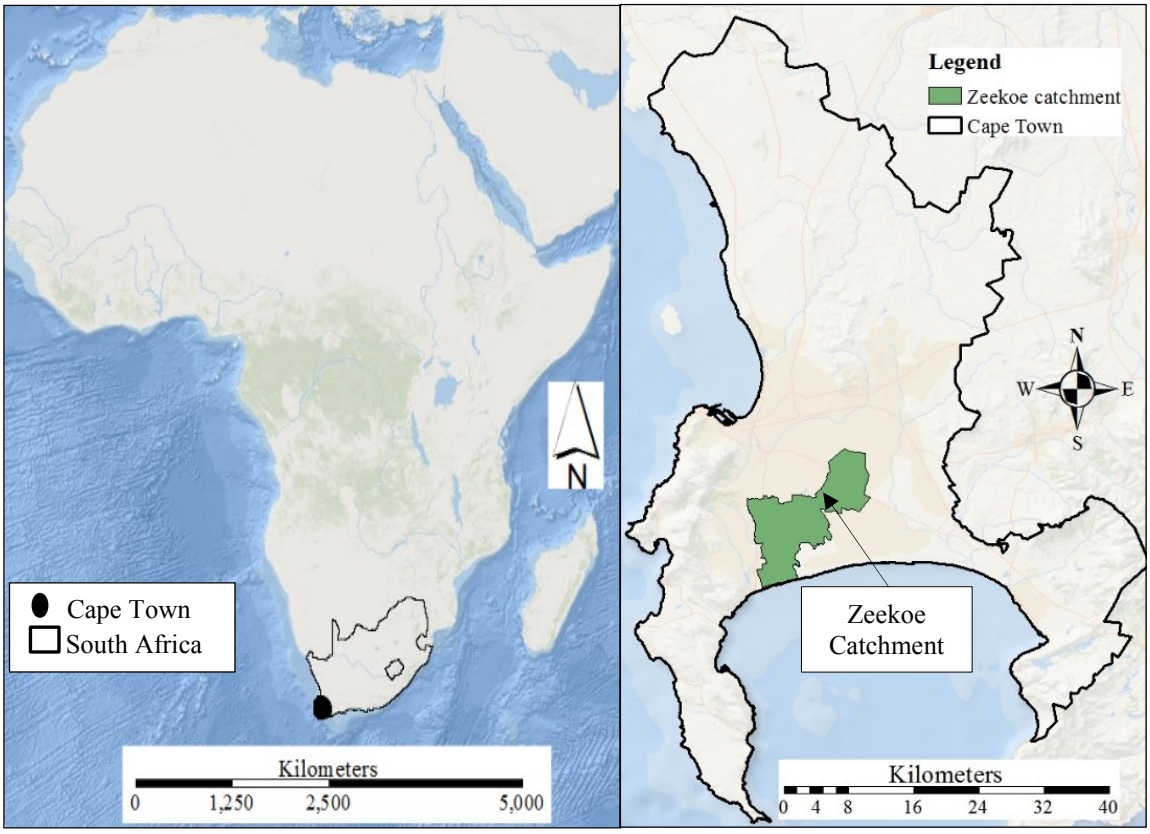


Figure 3-9 Zeekoe Catchment in Cape Town (CCT, 2012)

The Zeekoe Catchment was chosen from the various catchments in precinct of Cape Town as it had many stormwater ponds (some 61 ponds) and large shallow lakes (vleis) with the potential to be adapted to function as surface water storage and suitable location for the infiltration of stormwater to augment an unconfined aquifer. The catchment is located in an area with sandy soils with a relatively high groundwater flow rates as shown in Figure 3-14. The study area is relatively flat terrain (less than 3% average slope) with deep unconfined aquifer with a depth ranging from 20 – 50 m that offer an opportunity for MAR&R to store stormwater for harvesting later as groundwater. The aquifer has previously been identified as a potential groundwater resource in various studies (*e.g.* Tredoux *et al.*, 1980; Seward, 2009; and Adelana *et al.*, 2010). It contains a wide range of land-uses *e.g.* agriculture, public parks and residential gardens where stormwater with limited treatment would be suitable. It is also relatively close to the two largest water treatment plants in Cape Town (Faure and Blackheath; both about 30 km from the proposed location of stormwater abstraction at the Zeekoevlei (#6 in Figure 3-10). There are various water bodies and features in the Catchment as shown in Figure 3-10.

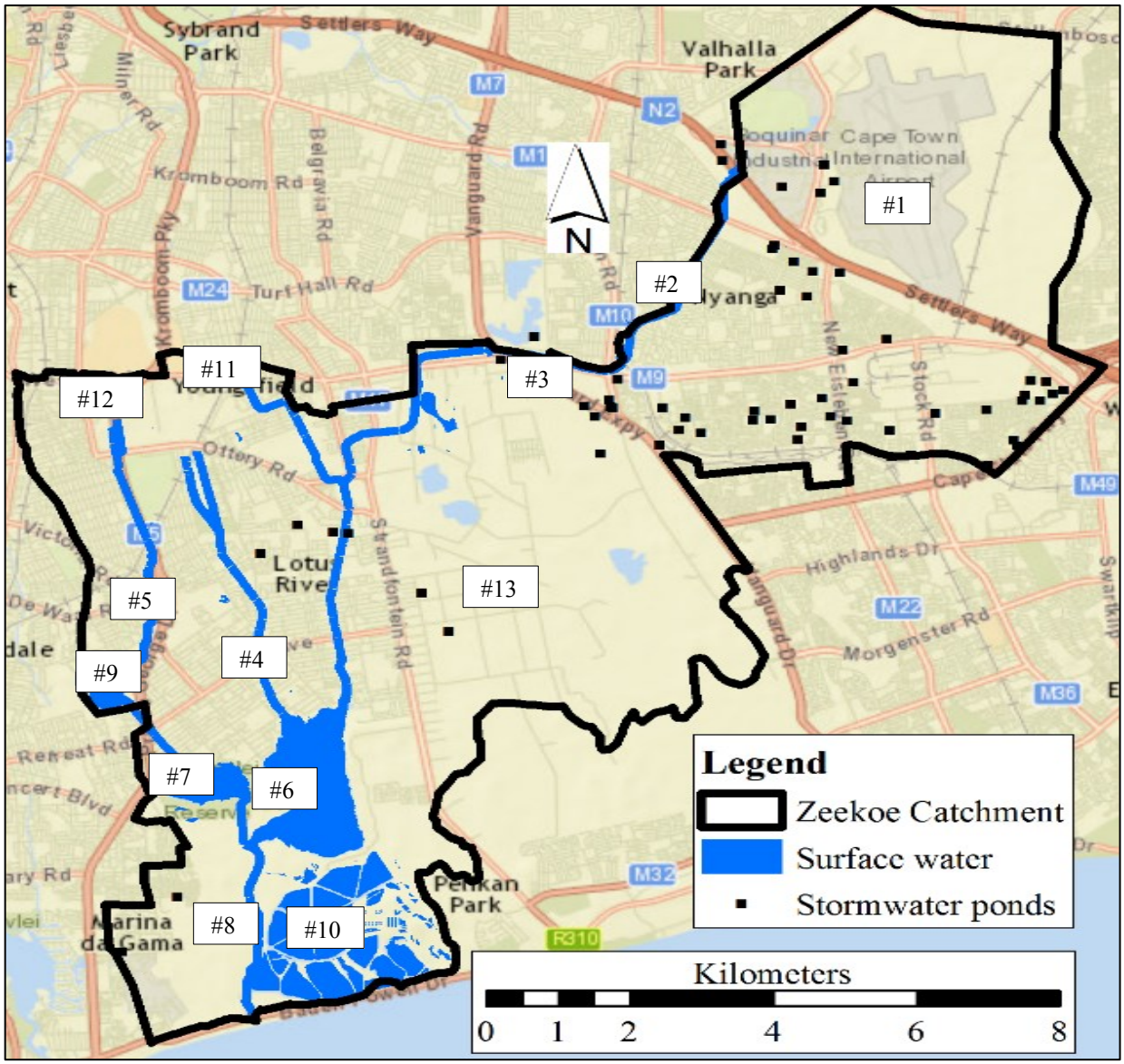


Figure 3-10 Main features in the Zeekoe catchment (CCT, 2012)

The main drainage channel of the Catchment is the Great Lotus (#2) that rises in precinct of the Cape Town International Airport (#1), a significant feature in the study area, and flows through a large proportion of the catchment into Zeekoevlei (#6), the Zeekoe Canal (#8) and finally discharges to the ocean. The other streams in the Zeekoe Catchment are the Little Lotus (#4) and the Southfield Canal (#5) that were constructed to drain the Youngsfield Aerodrome and Military Base (#11) and Kenilworth Racecourse (#12) respectively. The Southfield Canal discharges into Princessvlei (#9) and then Rondevlei (#7). Other key features include the Cape Flats Waste Water Treatment Works (#10), Edith Stephen's Wetland (#3) and the agricultural area #13. Some photos taken along the Great Lotus River are presented in Figure 3-11.

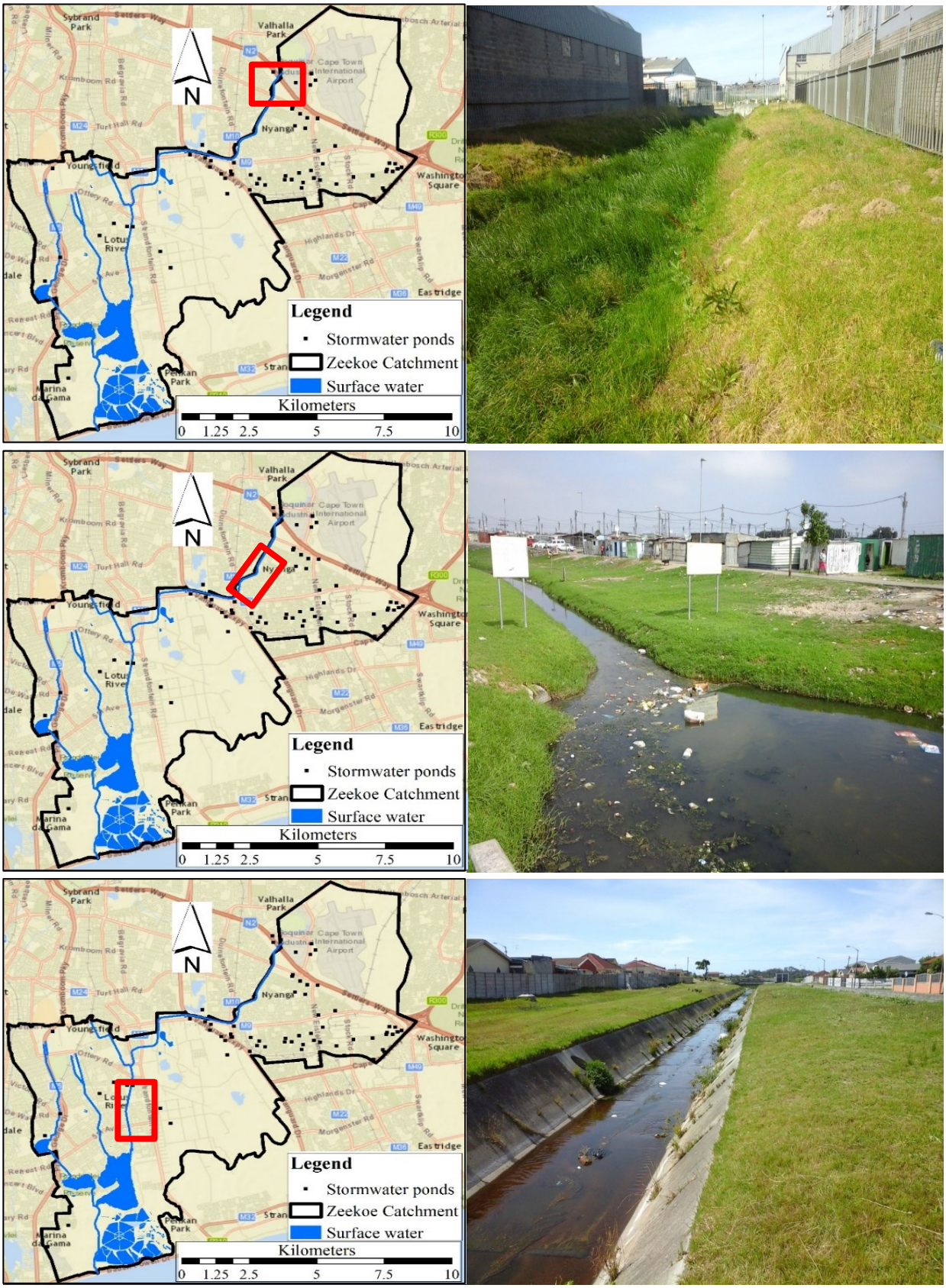


Figure 3-11 Views of typical sections along the Great Lotus River

Figure 3-12 Zeekoevlei

Figure 3-13 Rondevlei

Prior to the construction of the Cape Town International Airport (#1) and urbanisation in the study area, the vleis in the Zeekoe Catchment (*i.e.* labelled #6, #7 and #9 in Figure 3-10) were unconnected with no rivers (Brown & Magoba, 2009). After rainfall events, groundwater seeped into the vleis from the surrounding dunes and marshlands. The Zeekoevlei (**Figure 3-12**) and Rondevlei (Figure 3-13) were also not linked directly to the ocean, although there was a series of marshes stretching from the sea to the south-eastern corner of Zeekoevlei that flooded during high water levels in winter (Brown & Magoba, 2009). In the process of urbanisation, naturally occurring marshland was converted to largely impervious pavements, and this hardening of the earth surface resulted in increased runoff, thus increasing the risk of flooding in the area. To manage floods, the surface depressions and vleis were connected to constructed drains and stormwater canal (Brown & Magoba, 2009). Subsequently, additional flood control infrastructure was created including the Edith Stephens Wetland – a sizeable off-line stormwater pond – and various detention ponds (Grobicki, 2001). The Zeekoe Catchment currently contains some 61 stormwater ponds – mainly concentrated in the flood-prone area in the north-east of the catchment; an area characterised by several informal settlements, poorly drained aeolian sands (Brown & Dallas, 1995) and a generally high water table (Ziervogel & Smit, 2009).

The Zeekoe Catchment is now largely defined by stormwater drains. The Great Lotus River, that was mainly constructed to drain Cape Town International Airport, also drains the adjacent industrial area (Boquinar Industrial Area), as well as densely populated informal settlements, light industrial and low-middle income residential areas before discharging into Zeekoevlei. Along the way, it also flows around the Philippi Horticulture Area (PHA), an urban agricultural area in Cape Town. Since the area is undulating with a gradual overall slope to the sea, the availability of land was the main basis for determining the flow path of the Great Lotus (Brown & Magoba, 2009). The Great Lotus carries the highest pollution load of all the streams in the area as consequence of the areas it drains – most notably the informal settlements that are a source of grey and black water ingress into the stormwater drains. Although most of the Great Lotus is concrete-lined, some upstream sections are earth-lined, allowing for surface-groundwater interaction. The Little Lotus is not as profoundly impacted by pollution as the Great Lotus since it flows through areas of formal residential housing. The Southfield Canal drains the area around the Kenilworth Racecourse, then flows through Princessvlei, Rondevlei and finally into the Zeekoe Canal (#8 in Figure 3-10). The outflow from Zeekoevlei enters the Zeekoe Canal that flows southwards next to the Cape Flats Waste Water Treatment Works and into the sea. All the drains in the Zeekoe Catchment are periodically maintained to remove excess vegetation growth, litter and sediment deposits aimed at reducing sediment and solid waste deposits and improving flow in the channels for flood management.

3.5 Characteristics of the study area

In CCT, the Mean Annual Precipitation (MAP) varies from 350 – 2500 mm distributed as shown in Figure 3-14 with the rainfall in the Zeekoe Catchment ranging from 500 – 1100 mm. The soil type is mainly sandy with typical borehole yields in the range of 0.5 – 5 L/s (Figure 3-14).

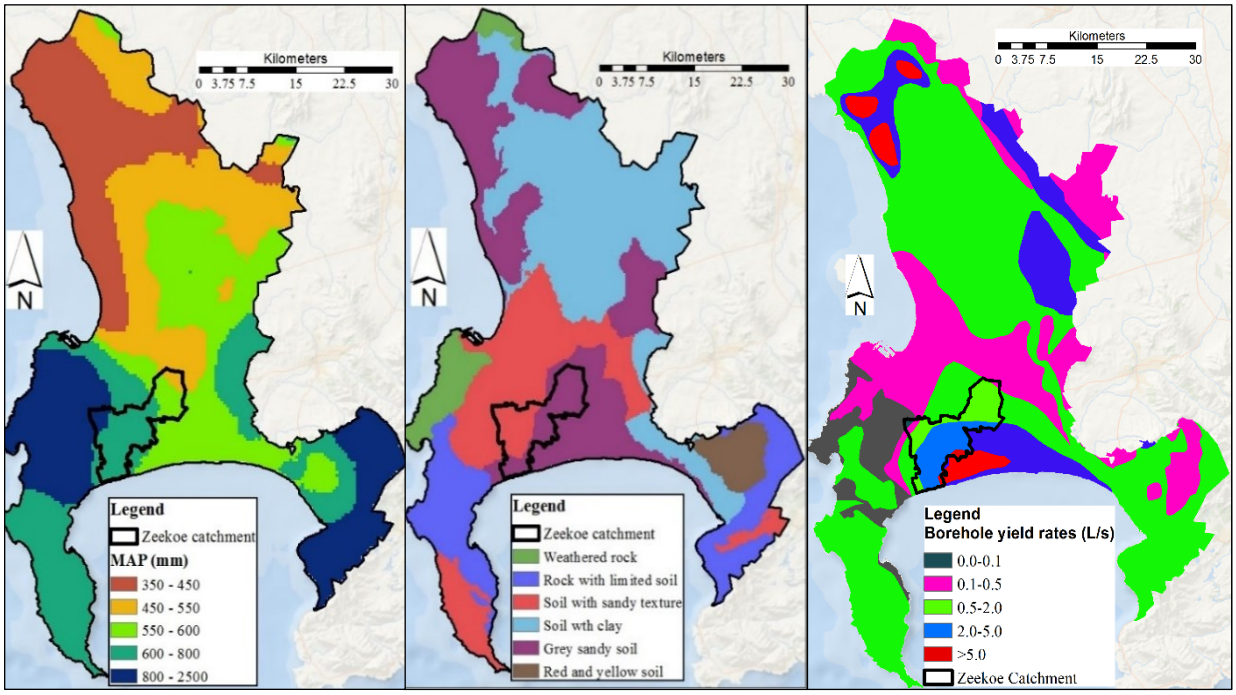


Figure 3-14 Rainfall, soils and groundwater yield in Cape Town (CCT, 2015)

The rainfall regime is such that over 50% of the MAP is in the winter months from June – August and about 80% over the period May – September (Figure 3-15).

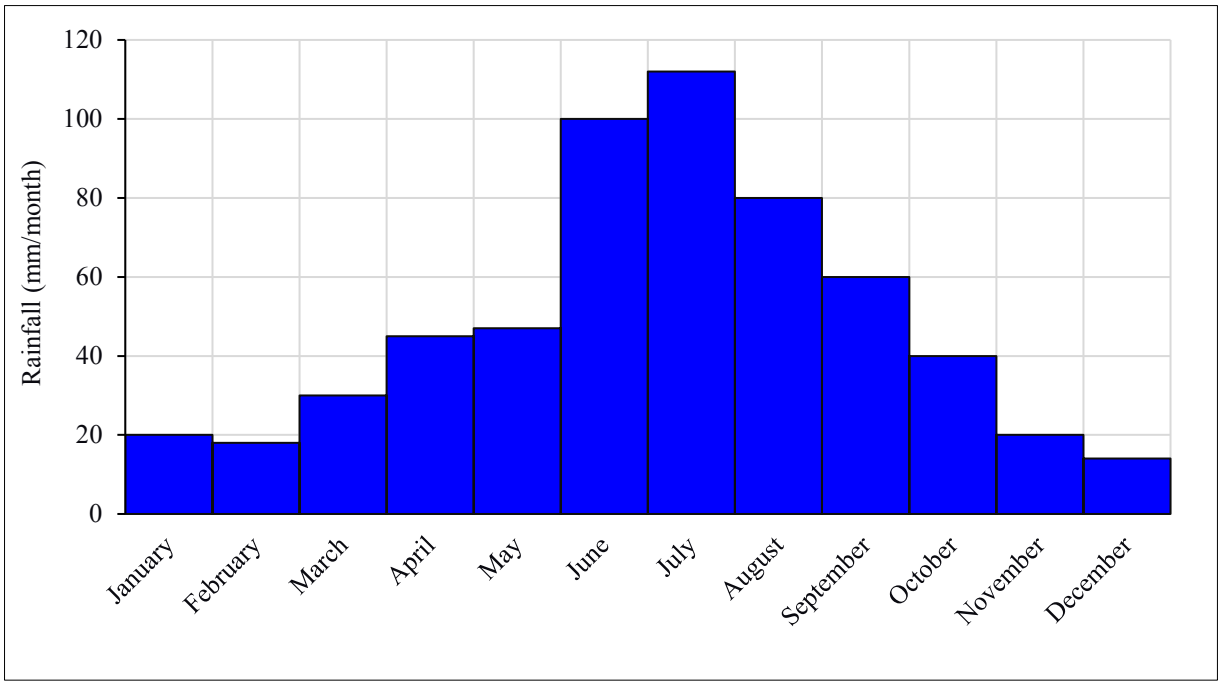


Figure 3-15 Mean monthly rainfall in the study area (CCT, 2015)

3.6 Data in the study area

3.6.1 Available data

The availability of various data sets as summarised in Table 3-1 was essential to reasonably model the hydrology of the catchment in this desktop study.

Table 3-1 Summary of data collected

Item	Data	Location	Resolution	Period	Sources
Hydrology data	Rainfall	CF WWTW	5 minutes	2012 – 2015	CCT
		Hanover			
		CT-Airport			
		Wynberg			
		Southfield			
		Mitchell's plain			
	Rainfall	CT-Airport	5 minutes	1992 – 2015	SAWS
		Mitchell's plain	5 minutes	2005 – 2015	
		Rondevlei	Daily	1952 – 2015	
	Temperature, Humidity, Wind	CT-Airport	Hourly	1992 – 2015	
		Mitchell's plain	Hourly	2005 – 2015	
		Rondevlei	Hourly	1952 – 2015	
	Rainfall, Temperature	CT-Airport	Daily	1960 – 2100	CSAG climate models
		Mitchell's plain	Daily	1960 – 2100	
		Rondevlei	Daily	1960 – 2100	
	River flow	6th Avenue	5 minutes	2012 – 2015	CCT
Water use	Billing records	Zeekoe Catchment	Monthly data	2011 – 2015	CCT
	Land use		Yearly	1998 - 2012	CCT, Google Earth
Stormwater network	GIS shapefiles	Zeekoe Catchment	Pipes and ponds	2015	CCT
Water quality data	*E. coli*, TSS, Temp, TN, TP, EC, DO, pH	Zeekoe Catchment	Monthly grab samples	1992 – 2015	CCT
Water quality data	Arsenic, Cadmium, Chromium, Lead, Mercury, *E.coli*, Temp, EC, TDS, pH	Zeekoe Catchment	Daily grab samples for five days	20th June 2016 - 24th June 2016	Swiss TPH and UCT sampling

3.6.2 Rainfall and flow data monitoring

There are various rainfall measuring stations in and around the study area. There are three stations managed by the South African Weather Services (SAWS) with long-time series, *i.e.* greater than ten years collected at a daily timescale. These include Cape Town Airport (1992 – 2015), Rondevlei (1952 – 2015) and Mitchel's Plain (2006 – 2015) labelled #1, #2 and #3 respectively in Figure 3-16. There are also four stations managed by the CCT that provide rainfall data at a five-minute time interval but with a limited period, *i.e.* 2012 – 2015. The stations include Southfield, Hanover Park, Cape Flats WWTW and Wynberg reservoir labelled #4, #5, #6 and #7 respectively. The two flow monitoring stations labelled #8, and #9 are managed by the CCT and provided data at a five-minute time interval but also with a limited period, *i.e.* 2012 – 2015.

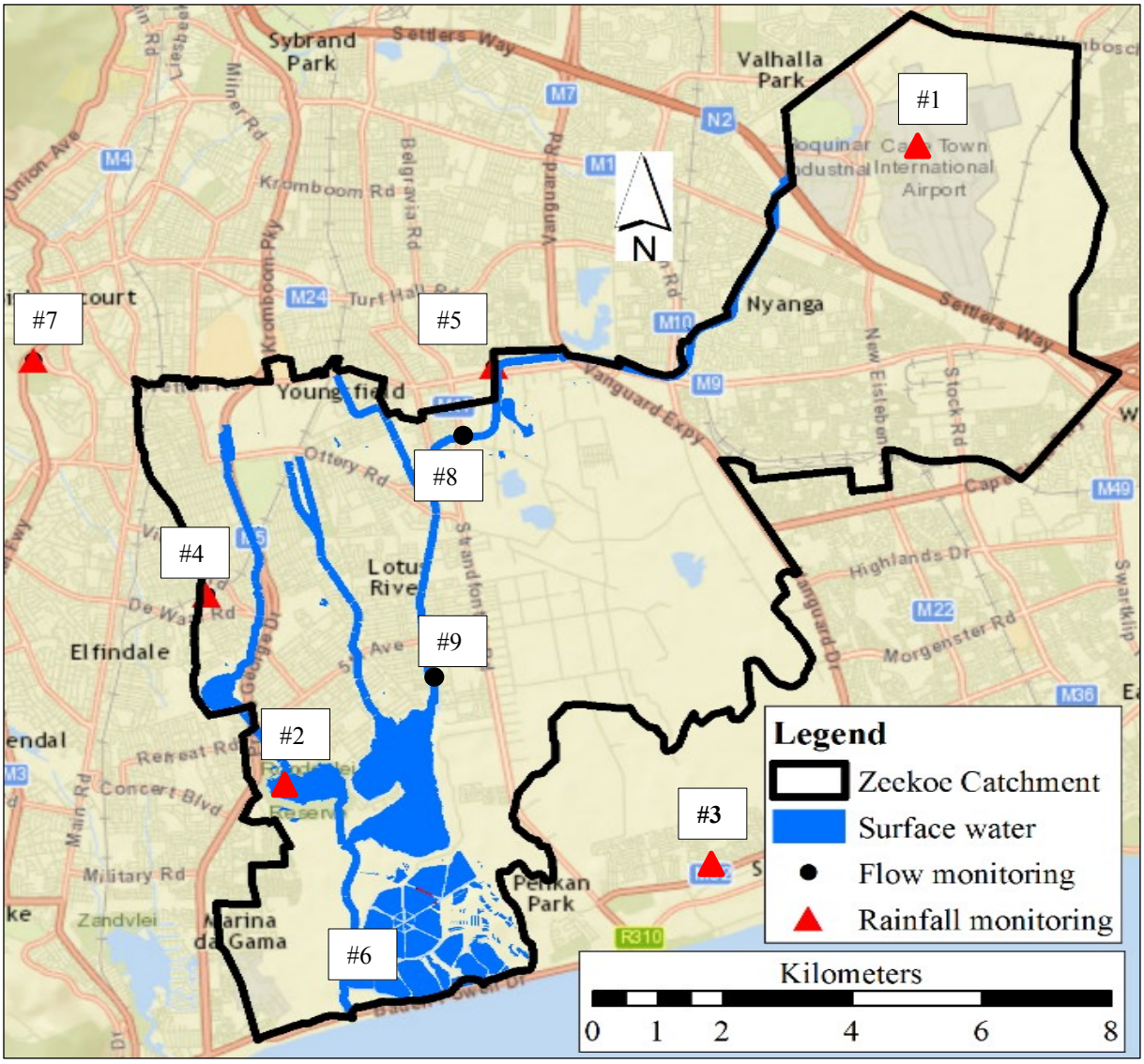


Figure 3-16 Rainfall and flow measuring stations (CCT, 2015)

3.6.3 Evaporation data

Evaporation is a critical process in hydrological modelling as it represents a significant water loss. There were three stations in the Zeekoe Catchment *i.e.* Cape Town International Airport, Schaapkraal and Zeekoevlei at locations as shown in Figure 3-17 with historical evaporation data that was measured with both Class A and Symon's Pans. Unfortunately, the stations are currently not in operation and data is missing for the study period (2006 – 2015). Although the evaporation data was not directly used in the hydrological modelling process, it was used to assess the accuracy of computed evapotranspiration (ET_o) values from empirical methods, *e.g.* Hargreaves (commonly used in hydrological models).

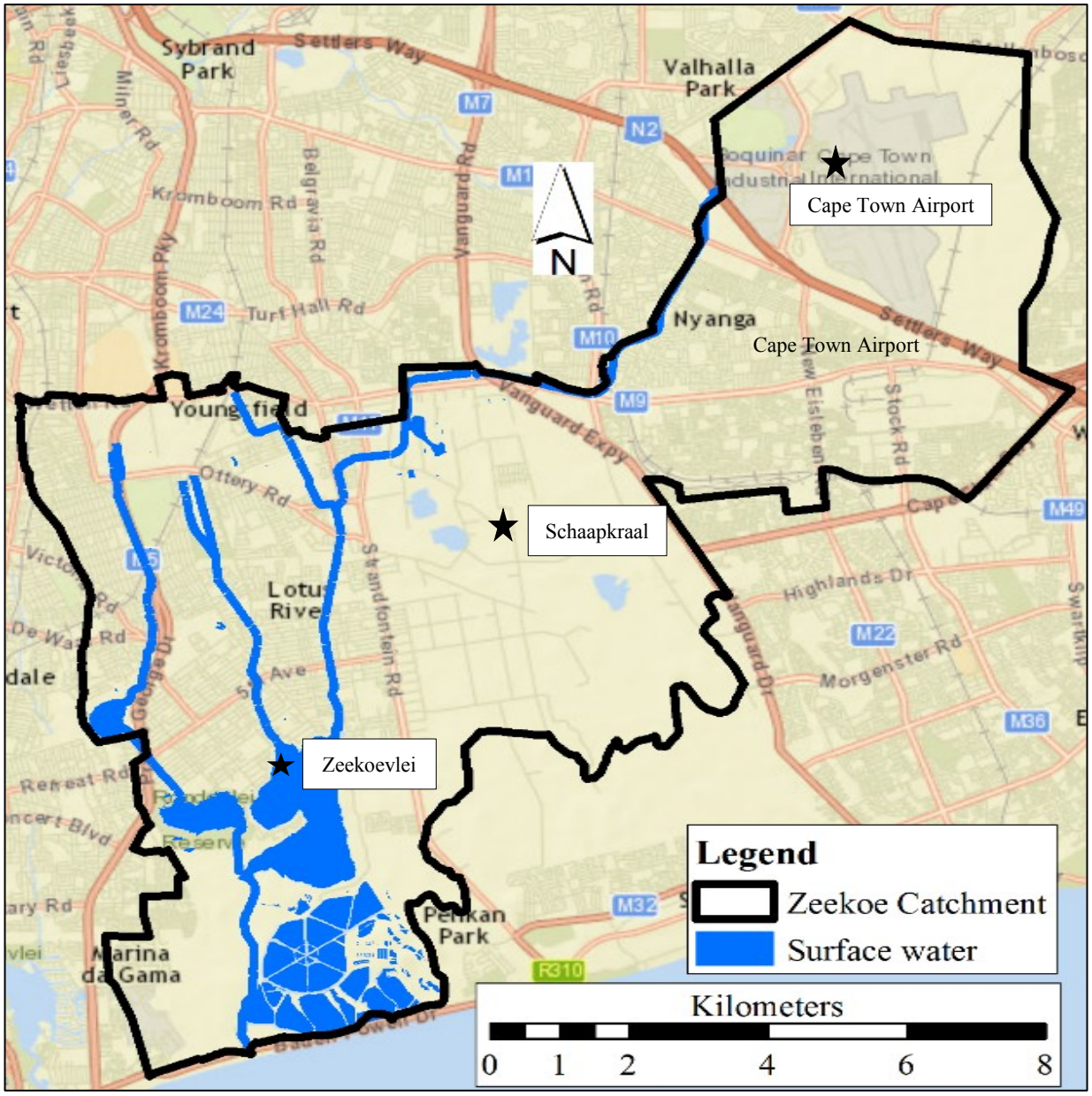


Figure 3-17 Evaporation measuring stations (CCT, 2015; DWS, 2015)

3.6.4 Data from climate change prediction models

The impact of climate change on demand and stormwater yield in the Zeekoe Catchment was also assessed to determine the need and extent required to account for its likely influence. Daily rainfall data for the period 1960 – 2100 from the 26 statistically downscaled Coupled Model Intercomparison Project Phase 5 (CMIP5) was acquired from the UCT – Climate Systems Analysis Group (CSAG) for Rondevlei and Airport Stations. The statistically downscaled data is from General Circulation Model (GCM) of different Representative Concentration Pathways (RCP) *i.e.* RCP 4.5 (intermediate mitigation scenario) and RCP 8.5 (high emission scenario) (Van Vuuren *et al.*, 2011). The seasonal variation of rainfall was also assessed to determine the likely impact on future rainfall. The climate models predict an increase in temperature as high as a 5^{o}C towards the end of the 21st century. Climate change is particularly significant in the projected dry and hot periods where a limited resource is expected to meet high outdoor water needs such as irrigation of residential gardens, agriculture and public open spaces.

3.7 Hydrological model selection

The availability of data for modelling and calibration was essential for the desktop study and was a vital consideration in the selection of the study area. For the model to adequately account for the hydrological processes required in the estimation of stormwater resources, various sets of data were needed including *inter alia* rainfall, evaporation, temperature, river flow, land use and soils. This section discusses the data collected and used in the surface water modelling. In the selection of a hydrological model for the study, the following factors were considered:

i. A tool that can comprehensively model an urban catchment at high spatial and temporal resolution;

ii. Available data and physical characteristics of the study area;

iii. Opportunity for Real-Time Control (RTC) analysis;

iv. Opportunity to the model surface to groundwater transfer;

v. Widely used the software in South Africa and internationally with user support; and

vi. Available at low or no cost (*i.e.* research or education edition).

The tools assessed included: Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (eWater, 2013), MIKESWMM (DHI Denmark, 2014), City Drain (Achleitner *et al.,* 2007), System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) (Lee *et al.*, 2012), Storm Water Management Model (SWMM) and proprietary versions *e.g.* XP-SWMM (XP Solutions, 2014), STORM (Civil Designer) and PCSWMM (CHI, 2014). As shown in Elliott & Trowsdale (2007), these models provide an opportunity to analyse natural and constructed drainage systems for decision support. As discussed in the literature review, the models can also be used for estimation of harvested stormwater volume and evaluate stormwater quality improvement during storage and conveyance (Hutchins *et al.*, 2017). Although reviews (*e.g.*

Breen *et al.,* 2006; Bach *et al.,* 2014; Akram *et al.,* 2014) have shown that most models focus on only one component of the urban drainage, some models such as MUSIC and PCSWMM have shifted towards assessment of integrated natural urban landscape and engineered water cycles. After evaluation of the various models, PCSWMM was selected based on the available data and opportunity to adequately define some specific functions in the modelling framework, *i.e.* extended the detention of water in a pond and opportunity to infiltrate into the underlying aquifer. The capacity to model an urban catchment in detail with Google Earth visualisation was also attractive. PCSWMM is widely used in South Africa especially in the CCT, the developers of PCSWMM run annual training workshops in several cities of South Africa, provide an extensive user support system and offer the software free to students for education and research purposes. It can model various hydrological processes, *i.e.* rainfall, evaporation and infiltration at very high temporal resolution (in minutes and real-time RADAR imagery) and spatial resolution (all available rainfall data) to produce reasonably accurate runoff flow and volume that can be calibrated to mimic observed river flows. Furthermore, PCSWMM can be used for RTC assessment and surface to groundwater transfer simulation. PCSWMM data inputs include temporally and spatially varying rainfall, directly measured and indirectly estimated evaporation and evapotranspiration. Hydrological processes that may be represented in the model include: rainfall abstraction by interception, wetting and depression storage, infiltration (*i.e*. unsaturated soil layers), percolation (*i.e.* infiltrated water into groundwater layers), interflow between groundwater and the drainage system, nonlinear reservoir routing of overland flow, retention and infiltration through stormwater ponds (James *et al.*, 2010). Spatial variability is represented by dividing the catchment into smaller homogeneous sub-catchment areas, each containing distinct land use and soil characteristics.

3.8 Groundwater model

The first part of the groundwater modelling was undertaken with the aid of PCSWMM to determine the potential for stormwater transfer to groundwater storage through MAR with the infiltration being primarily carried out in the existing stormwater ponds. To enhance infiltration and augmentation of the groundwater, the stormwater ponds were modelled as infiltration basins. The second part included modelling of the groundwater abstraction process to determine the withdrawal potential of the infiltrated stormwater. The study also determined the most suitable locations to place the abstraction boreholes relative to the infiltration basins, such that the generated flow fields are limited to the saturated areas. The aim of limiting the flow fields to areas around the saturated areas was to increase the likelihood that the groundwater abstraction process benefits from stormwater infiltration practice. The most popular model applied in similar studies was MODFLOW, a groundwater flow simulation software based on Darcy's law and mass balance equations to derive cell-to-cell flow in an aquifer represented with a matrix (Boskidis *et al.,* 2012). The main limitation of MODFLOW in the application of recharge of an aquifer with stormwater only is the inability to model an unsaturated zone (Brunner *et al.,* 2009; Mauck, 2017). Although some surface water models including *inter alia* PCSWMM, MUSIC

and Infoworks consider infiltration and sub-surface flow, they cannot represent groundwater abstraction. Some tools have been developed to couple surface and groundwater models, *e.g.* the Multiple Model Broker that links SWMM with MODFLOW and IWAS-Toolbox that connects SWMM with OpenGeoSys (a subsurface model) (Kalbacher *et al.,* 2012). Since most of these coupling models are not widely tested and used, recharge is typically measured in a surface water model (*e.g.* PCSWMM) and used as input for the groundwater model (*e.g.* MODFLOW). To adequately represent the groundwater flow, abstraction, and potential water quality improvement, it was decided to model the process from first principles. The process was modelled in MATLAB based on an approach in Mahinthakumar & Sayeed, (2005; 2006) as discussed in Chapter 5.

3.9 Stormwater harvesting and supply options

In this study, a 'catchment scale' SWH was investigated, *i.e.* stormwater ponds providing temporary storage and release to the most downstream location of the catchment for abstraction, treatment and supply as potable or non-potable water based on land use (Figure 3-18).

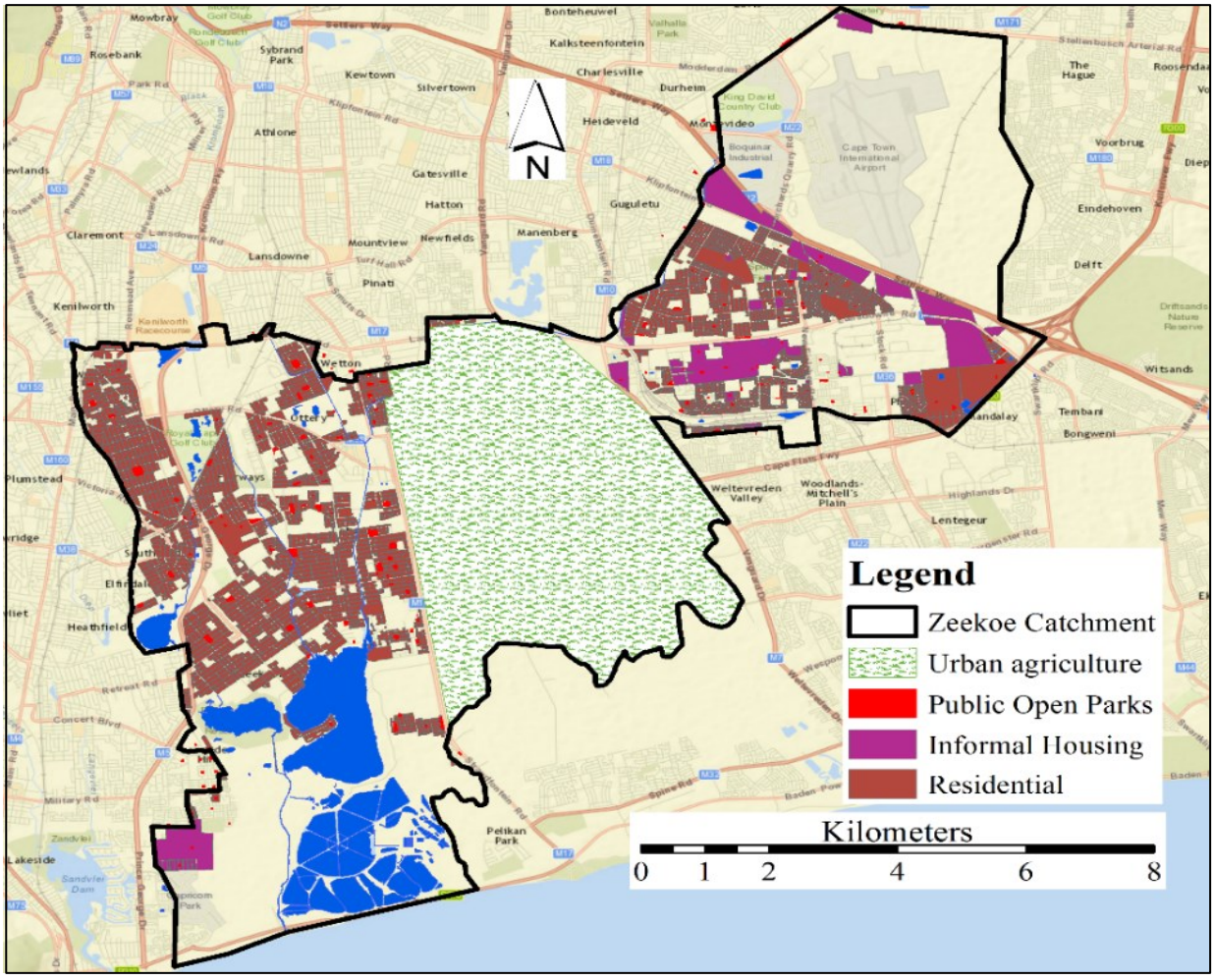


Figure 3-18 Land use in the Zeekoe catchment

In this approach referred to as the 'decentralised system', the harvested stormwater would be restricted to locations in the Zeekoe Catchment. The harvested stormwater would be treated to non-potable water standards, distributed in a dual reticulation system 'third pipe' which is colour coded and secured with locks to minimise health risks and used for selected applications such as toilet flushing, irrigation of residential gardens, open parks, and urban agriculture. Alternatively, the harvested stormwater would be treated to potable water standards, distributed with existing reticulation system and used for all requirements in the study area. The modelling of stormwater harvesting from surface water storage was based on the Yield After Spillage (YAS) (Mitchell *et al.* (2008) as discussed in Section 4.4. The abstraction from the two most downstream vleis (Zeekoevlei and Rondevlei) and distribution in the study area was modelled in EPANET2 integrated in PCSWMM. The other option assessed was abstraction from the two vleis *i.e.* Zeekoevlei and Rondevlei labelled #1 and #2 respectively in Figure 3-19, pre-treated at a new proposed WTP and conveyed to an existing WTP *e.g.* Faure WTP (Figure 3-19). The costing of water abstraction, treatment and distribution processes are discussed in Chapter 7.

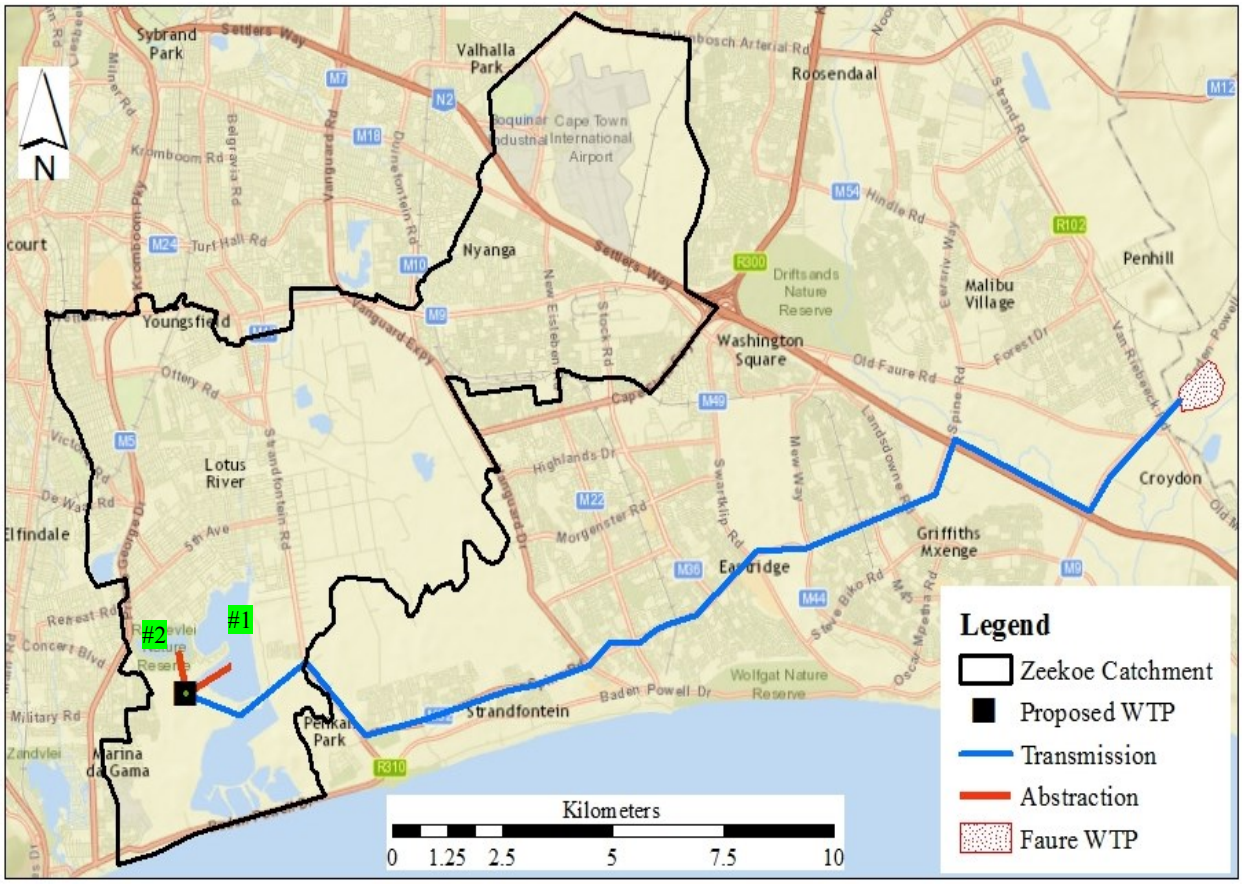


Figure 3-19 Centralised system with abstraction and conveyance

3.10 Summary of the method

A summary of the method adopted:

i. The hydrological process in the Zeekoe Catchment was modelled with the aid of PCSWMM software to quantify the stormwater resource. The opportunity for extended detention of runoff in the various stormwater ponds and vleis was also modelled to determine the reliability of the available storage for SWH;

ii. To address the identified challenges of limited capacity in the existing ponds to capture a significant portion of the runoff, the use of RTC was assessed to determine potential storage enhancement while safeguarding the original purpose, *i.e.* flood-control;

iii. Even with the application of RTC, there was limited additional benefit and considerable amounts of stormwater was lost as spillage from surface water storage. Accordingly, the available aquifer in the study area was considered as the principal storage medium, *i.e.* relying on the storage in the existing ponds only to give time to infiltrate water into the aquifer;

iv. An assessment was undertaken to determine the viability of two water supply options, *i.e.* harvested stormwater treated with a proposed new WTP and distributed locally in the study area or transferred and blended with the raw water stream coming into existing WTPs;

v. For the water supply option where the harvested stormwater was treated with a proposed new WTP, an assessment was undertaken to determine treatment requirement and what could be delivered at each stage of the system as non-potable or potable;

vi. The study also assessed likely impacts of climate and land use change on harvested stormwater in the future; and

vii. An economic analysis was undertaken to determine the viability of the approach, *i.e.* the cost of harvested stormwater compared with existing tariffs and other proposed sources, *e.g.* groundwater, reclaimed water and seawater.

4. Stormwater harvesting from surface storage

In this chapter, the method and associated results relating to the prospects for SWH from surface water storage and determination of the total harvestable volumes are provided and discussed in five sections including the available data and hydrological model of the study area; the stormwater ponds adapted for water supply; and the modelling of the SWH process.

4.1 Data for hydrological modelling

4.1.1 Overview

The availability of data for modelling and calibration was essential for the desktop study and was thus a vital consideration in the selection of the study area. For the model to adequately account for the hydrological processes required in the estimation of stormwater resources, various sets of data were needed including *inter alia* rainfall, evaporation, temperature, river flow, land use and soils. This section discusses the data collected and applied in the surface water modelling.

4.1.2 Rainfall

Rainfall data is a key input in hydrological modelling. As shown in Figure 3-14, the Mean Annual Precipitation (MAP) in the Zeekoe Catchment ranges from 500 – 1100 mm. To reasonably represent the significant range and variability in rainfall, various monitoring stations as shown in Figure 3-7 were used as input in modelling the hydrological processes of the catchment. The data from CCT was at 5 minutes intervals and was used for the hydrological modelling. The available rainfall data was analysed to determine consistency and missing values, and where necessary, was patched and the total volume linearly scaled with reference to the nearest SAWS station.

4.1.3 Evaporation

There are three evaporation stations in the Zeekoe Catchment as shown in Figure 3-8 with historical evaporation data measured using both Class A and Symon's Pans. The stations were not in operation for the modelled period (2006 – 2015). The available evaporation data for a ten year period (1993 – 2002) was used to determine the accuracy of computed evapotranspiration (ET_o) values from empirical methods such as the Hargreaves Method that are commonly used in hydrological models such as PCSWMM. To make the comparison, the available measured evaporation data from Class A pan (E_{pan}) (1993 – 2002) was thus converted to ET_o through an empirically derived pan coefficient (k_p) using Equation 4-1 (Savva & Frenken, 2002, FAO,1998):

$$ET_o = k_p \times E_{pan} \qquad \textbf{Equation 4-1}$$

Where ET_o – Evapotranspiration (mm/day); k_p – Class A pan coefficient; and E_{pan} – Class A pan evaporation data (mm/day)

In the estimation of the daily ET_o, the appropriate values of k_p were obtained from Savva & Frenken (2002). A summary of the k_p values is provided in Table 4-1.

Table 4-1 Values of pan coefficient k_p (Savva & Frenken, 2002)

Wind	Upwind fetch of green crop	Case A: Pan surrounded by short green crop		
		Mean Relative Humidity		
(km day^{-1})	(m)	<40%	40-70%	>70%
<175	1	0.55	0.65	0.75
<175	10	0.65	0.75	0.85
<175	100	0.7	0.8	0.85
<175	1000	0.75	0.85	0.85
175-425	1	0.5	0.6	0.65
175-425	10	0.6	0.7	0.75
175-425	100	0.65	0.75	0.8
175-425	1000	0.7	0.8	0.8
425-700	1	0.45	0.5	0.6
425-700	10	0.55	0.6	0.65
425-700	100	0.6	0.65	0.7
425-700	1000	0.65	0.7	0.75
>700	1	0.4	0.45	0.5
>700	10	0.45	0.55	0.6
>700	100	0.5	0.6	0.65
>700	1000	0.55	0.6	0.65

The required daily wind speed and mean relative humidity data were derived from the historical records available at Cape Town Airport located in the north-west of the study area as shown in Figure 4-1 and 4-2. The pan coefficients k_p corresponding to the daily wind speed (Figure 4-1) and relative humidity (Figure 4-2) were used to estimate the associated evaporation values *i.e.* derived from Class A pan data using Equation 4-1.

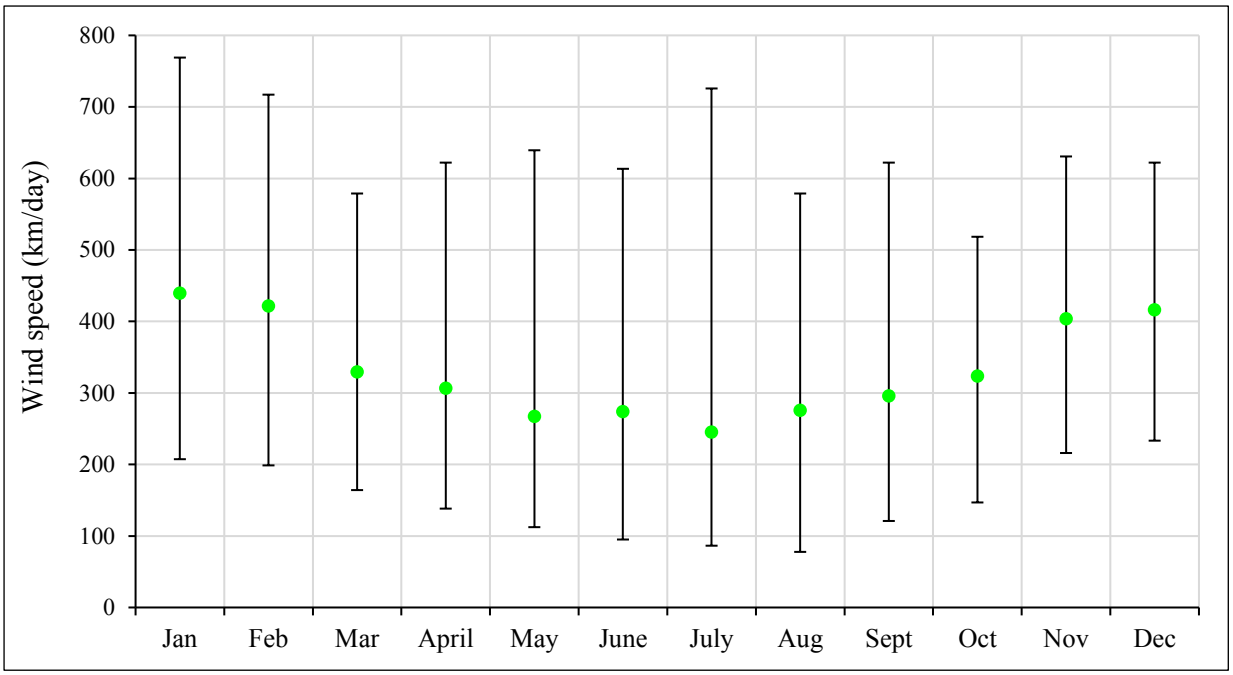


Figure 4-1 Mean daily wind speeds (CCT, 2015)

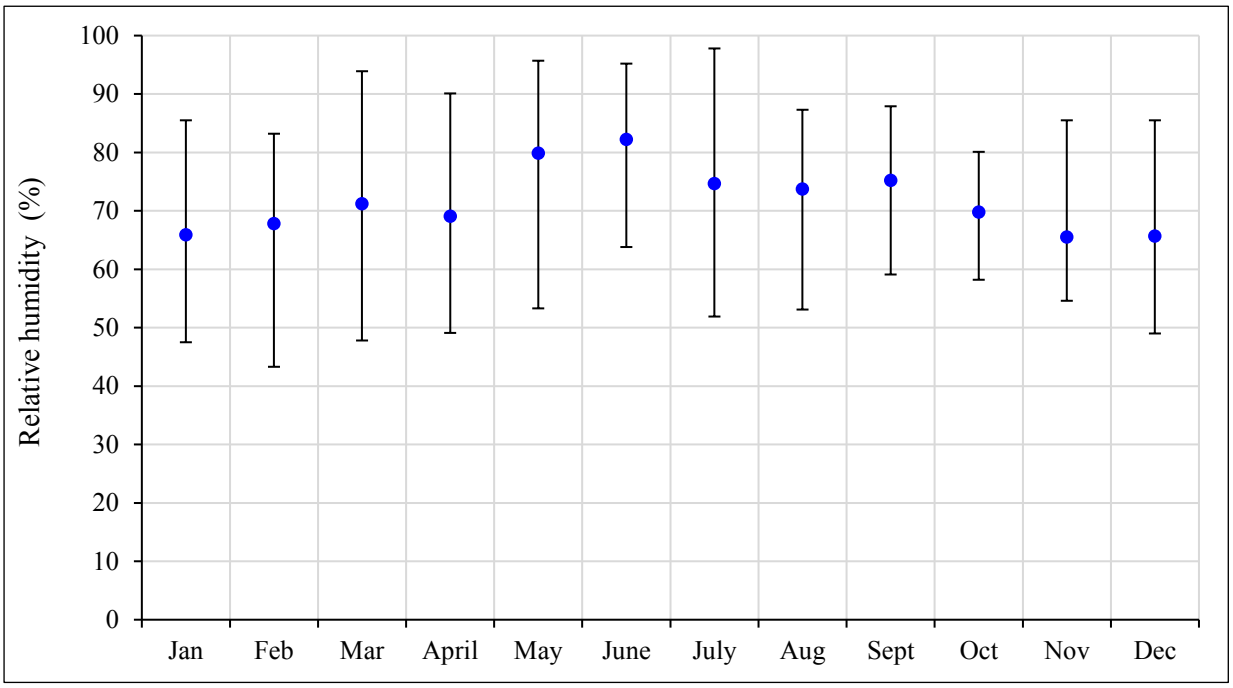


Figure 4-2 Mean daily relative humidity (CCT, 2015)

Jensen *et al*. (1990) compared results from directly measured ETo experiments using a Lysimeter at 11 locations) and various empirical methods including the Hargreaves and Blaney-Criddle methods. The study determined that the Hargreaves method provided values closest to the

measurements from the Lysimeter with Standard Error Estimate (SEE) of 0.9 mm day^{-1}. To confirm the validity of findings for the study area, the results from Class A pan data were compared with empirically derived values estimated using the Hargreaves and Blaney-Criddle methods both based on temperature data (mean temperature data is presented in Figure 4-3).

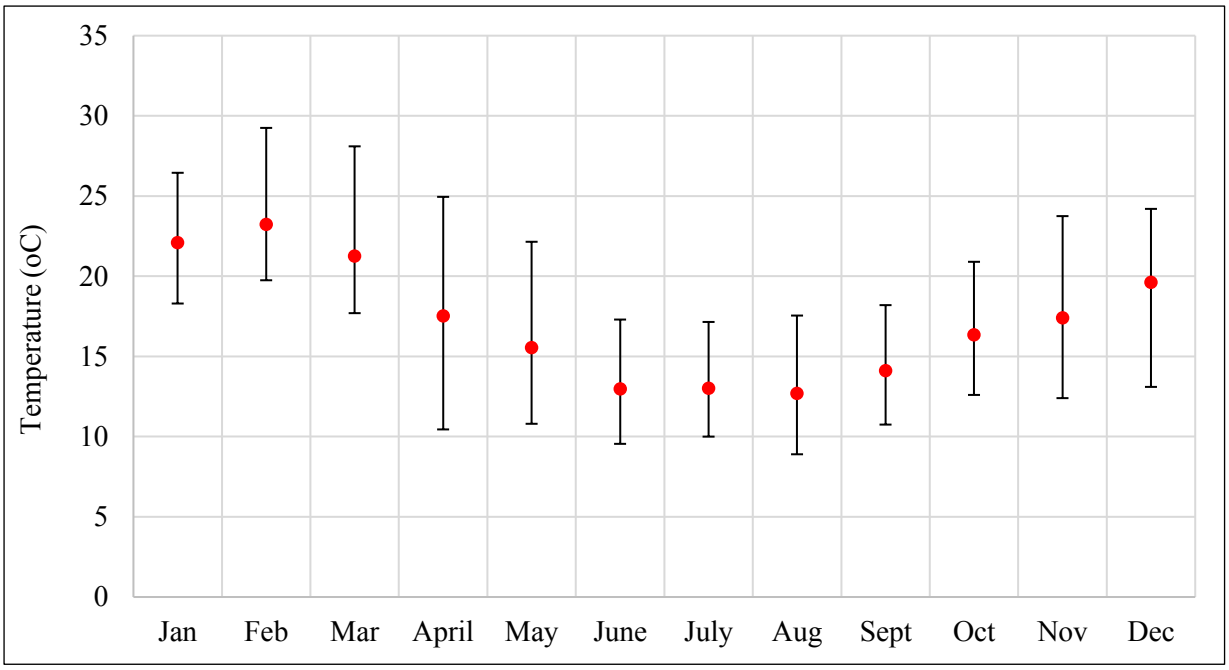


Figure 4-3 Mean daily temperature (CCT, 2015)

The Hargreaves and Blaney-Criddle methods were computed with Equation 4-2 (Hargreaves & Allen, 2003) and Equation 4-3 (Blaney & Criddle, 1962) respectively.

$$ET_o = 0.0023 R_a (T_{mean} + 17.8) * TR^{0.5} \qquad \textbf{Equation 4-2}$$

Where ET_o – Evapotranspiration (mm day^{-1}); R_a – extra-terrestrial radiation (MJ m^{-2} day^{-1}); T_{mean} – daily mean temperature (^{o}C); TR – daily temperature range (^{o}C) (*i.e.* T_{max} - T_{min} where T_{max} and T_{min} are the mean daily maximum and minimum temperature respectively).

$$ET_o = p(0.457 T_{mean} + 8.128) \qquad \textbf{Equation 4-3}$$

Where ET_o – Evapotranspiration (mm day^{-1}); p – mean daily percentage of annual daytime hours (dimensionless); T_{mean} – daily mean temperature (^{o}C).

The other required parameters in the Blaney-Criddle and Hargreaves methods, *i.e.* mean daily percentage of annual daytime hours (p) and extra-terrestrial radiation (R_a) are given in Table 4-2.

Table 4-2 Mean daily percentage of annual daytime hours and extra-terrestrial radiation (FAO, 1998)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
p (dimensionless)	0.32	0.3	0.28	0.25	0.23	0.22	0.23	0.25	0.27	0.29	0.31	0.32
Ra (MJ m^{-2} day^{-1})	29.0	30.7	31.4	30.3	28.1	26.7	27.1	29.1	30.8	30.9	29.6	28.4

The estimated mean evapotranspiration values from the measured data (Class A pan) and empirical methods (Blaney-Criddle and Hargreaves methods) are presented in Figure 4-4.

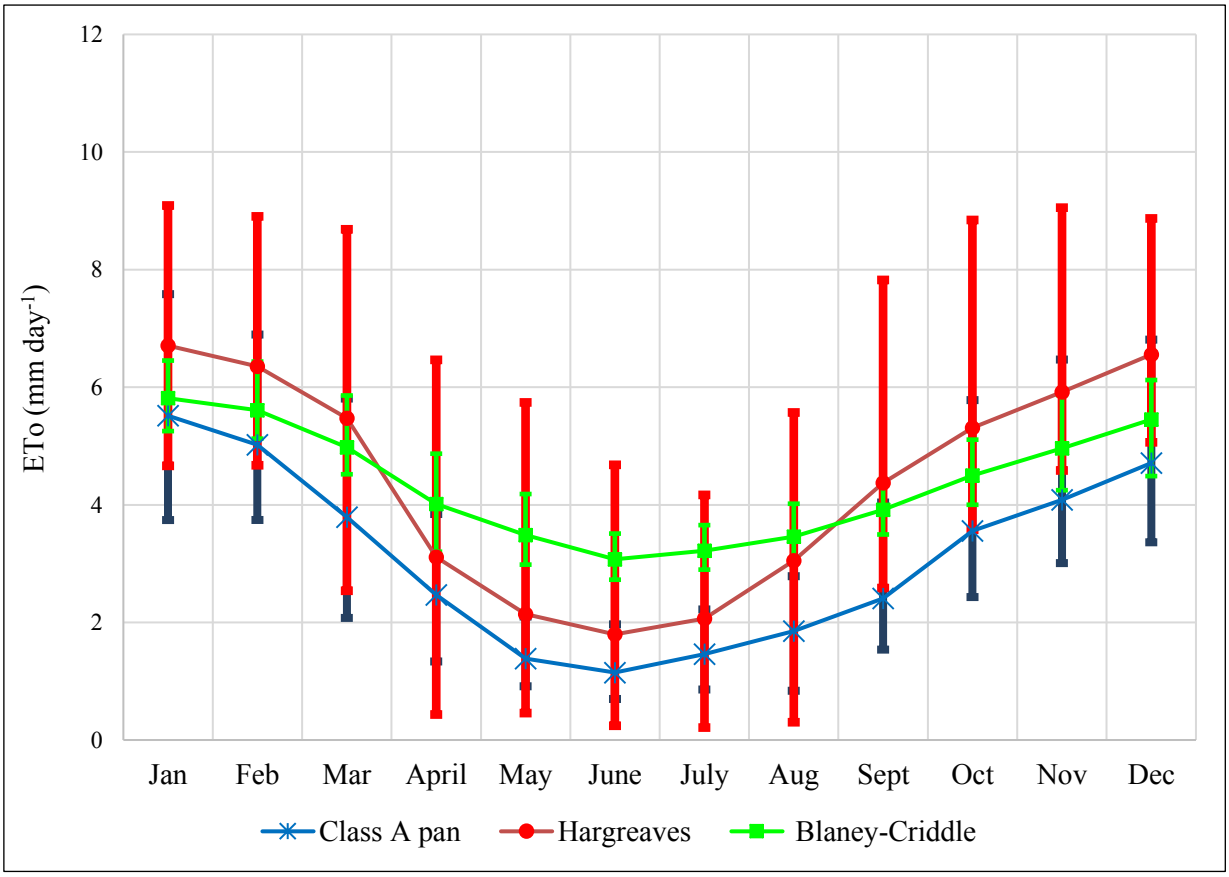


Figure 4-4 Estimated mean daily evapotranspiration values and trends

On the basis of the comparison as shown in Figure 4-4, it was determined that both empirical methods, *i.e.* Hargreaves and Blaney-Criddle produced higher evapotranspiration values than the estimates from the Class A pan. The likely reason for the over-estimation was that the empirical

methods provided evapotranspiration estimates based on temperature as the only measured data. The Hargreaves method was used in the study as it better mimicked the Class A pan and also provided better monthly and annual values than the Blaney-Criddle method.

4.1.4 Data from climate change prediction models

The impact of climate change on the stormwater resource was also assessed to determine the need and extent required to account for its likely influence. Historical and future daily rainfall data (1960 – 2100) from 26 models statistically downscaled from the Coupled Model Intercomparison Project Phase 5 (CMIP5) were acquired from the UCT – Climate Systems Analysis Group (CSAG) for two stations in the study area, Rondevlei and Airport. The 5-year and 13-year moving mean of various climate models at Rondevlei are shown in Figure 4-5.

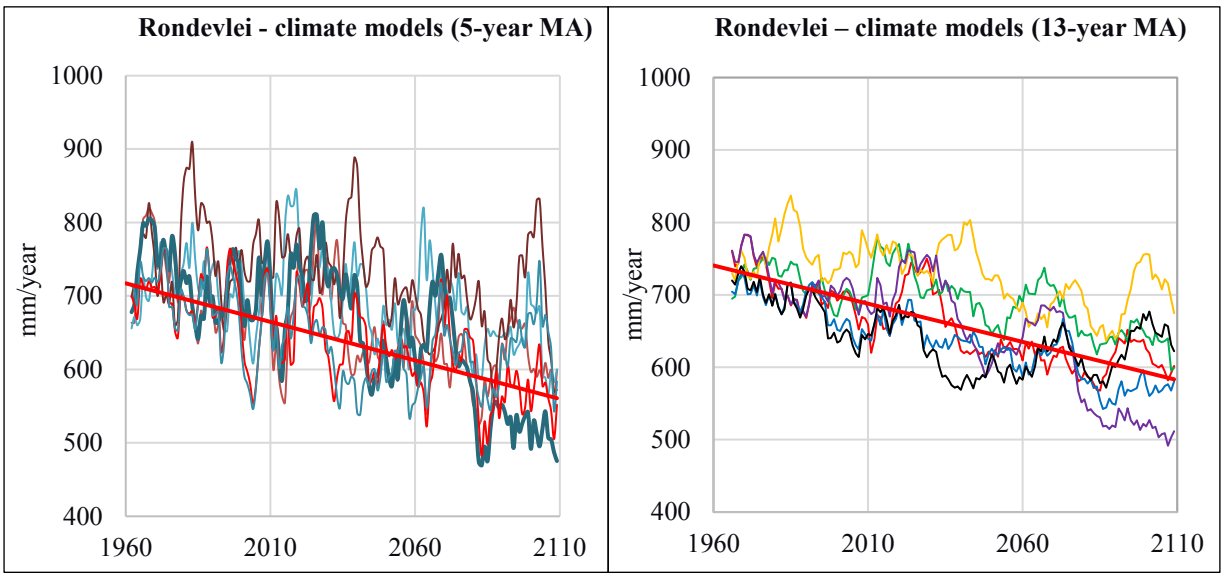


Figure 4-5 Rainfall trend time series from climate models at Rondevlei station

(*after* Hewitson & Crane, 2006)

The climate data was from two different Representative Concentration Pathways (RCP); RCP 4.5 (intermediate mitigation scenario) and RCP 8.5 (high emission scenario). The RCPs are named according to the predicted radiative forcing target levels for 2100 with RCP 4.5 (the medium stabilisation scenario) and RCP 8.5 (the very high baseline emission scenario) (Van Vuuren *et al.*, 2011). The likely seasonal variation in rainfall, temperature and changes over time are as presented in Figure 4-6 and 4-7 (*after* Hewitson & Crane, 2006). Climate change is particularly significant for the projected dry and hot periods where a limited resource is expected to meet high outdoor needs *e.g.* irrigation of residential gardens, agriculture and open spaces.

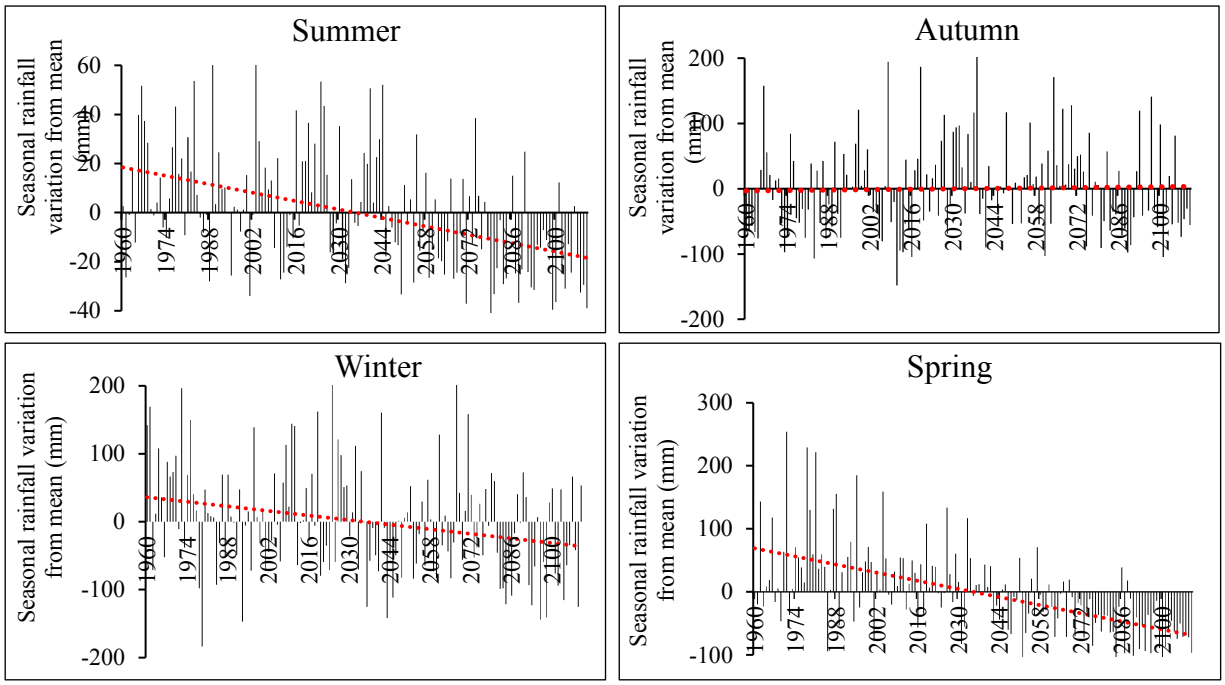


Figure 4-6 Seasonal rainfall variation from the historical mean (2006 – 2015)
(*after* Hewitson & Crane, 2006)

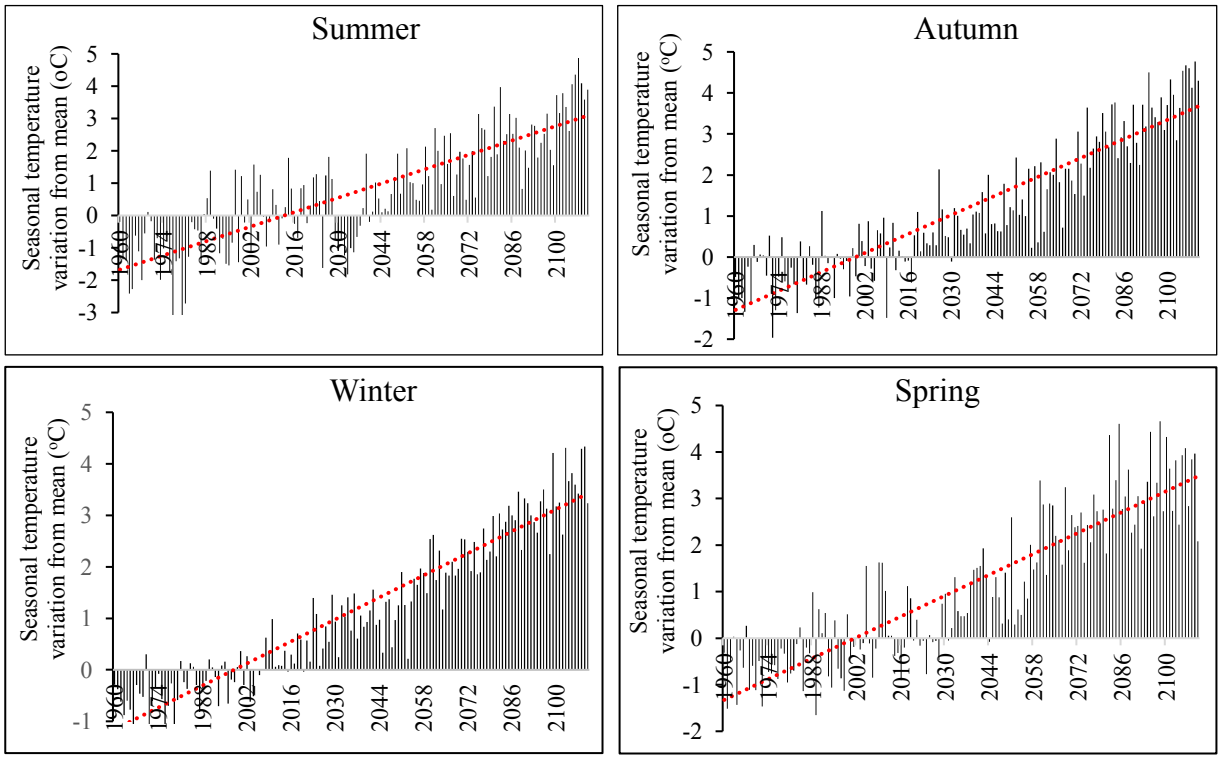


Figure 4-7 Seasonal temperature variation from the historical mean (2006 – 2015)
(*after* Hewitson & Crane, 2006)

4.2 Hydrological model for the Zeekoe Catchment

4.2.1 Overview

For the stormwater model development, the Zeekoe Catchment was subdivided into smaller sub-catchments based on the stormwater pipe network and ponds; density of development; road network; and topography. A total of 118 sub-catchments were generated as shown in Figure 4-8.

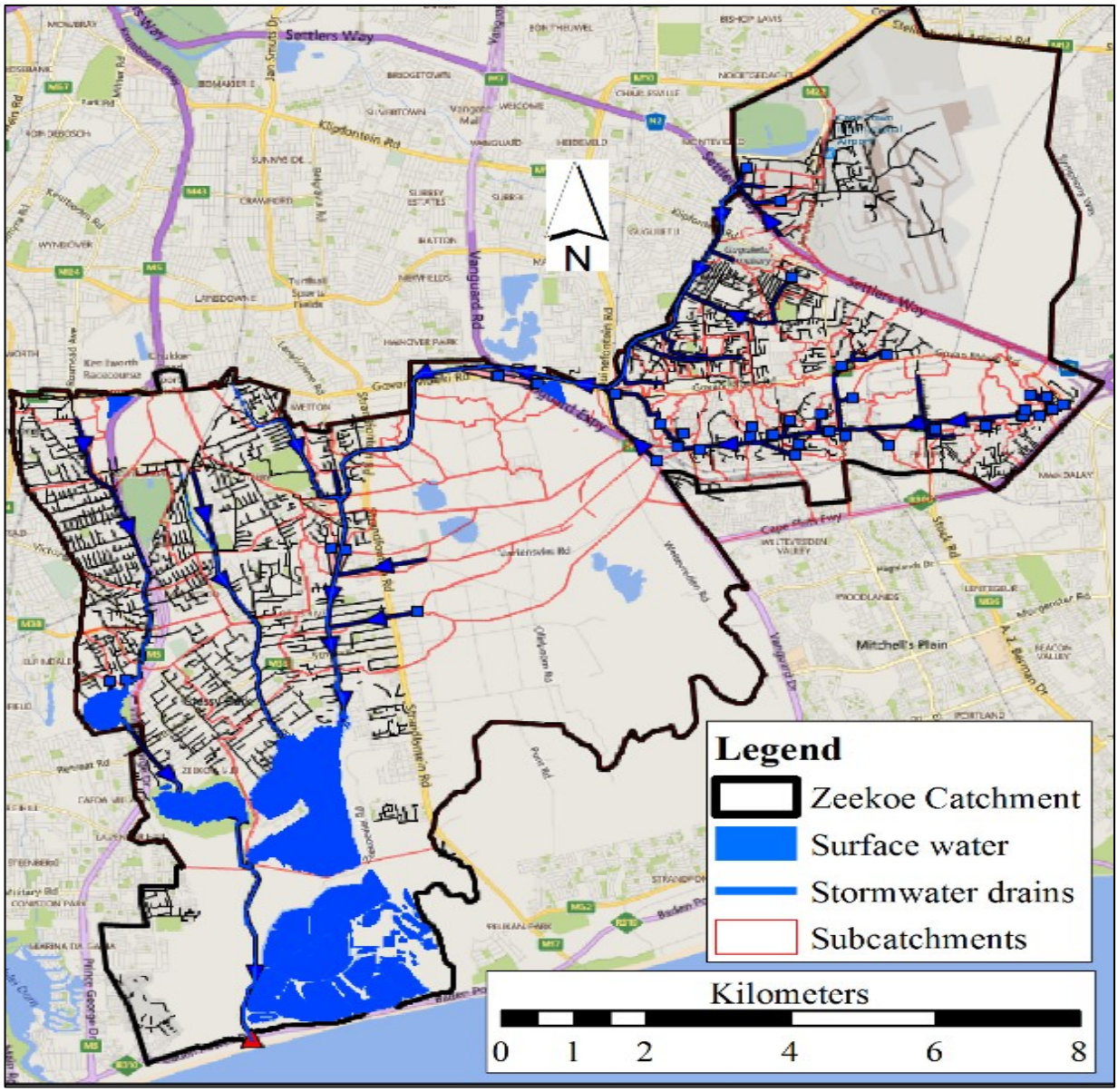


Figure 4-8 Sub catchments and stormwater network (*after* CCT, 2015)

The mean area of the delineated sub-catchments was 0.83 km^2 with some in the highly dense built-up areas as small as 0.01 km^2. In the less dense areas, *e.g.* agricultural areas and nature reserves, the sub-catchments were much larger, typically greater than 1 km^2.

4.2.2 Stormwater network

The stormwater network layout of the Zeekoe catchment was acquired from the CCT in the form of GIS shapefiles that could be uploaded into ArcGIS and PCSWMM. The model was initially set-up to include all the available stormwater pipes and channels, catchpits, manholes, and ponds, but owing to missing data, *e.g.* cover levels, invert levels and pipe diameters, the stormwater network in the model had to be 'fixed' so that at least all water flowed downstream. The data input was carried out in a stepwise manner commencing from the most downstream to the most upstream location in the Catchment as follows:

i. The open channel widths and depths were measured in PCSWMM by drawing transects on the 0.5 m resolution LIDAR DEM. A field visit was undertaken to some of the drainage channels to confirm the estimated values.

ii. Most of the pipe diameters were available and were presumed to be correct with spot checks being confirmed in field visits. If pipe diameters were missing, these were generally inferred from neighbouring pipes draining similar sub-catchments.

iii. For the connecting pipes upstream, all the available diameters were presumed to be correct. Missing pipe diameters were assumed to be equal to those just immediately downstream.

iv. All the pipe lengths were measured with the PCSWMM 'auto-length' functionality using 'Google Maps'.

v. All the available invert levels were presumed to be correct. Missing invert levels were estimated from a linear interpolation of the values just immediately downstream and upstream.

vi. Finally, the modelled network was checked to ensure that everything flowed downstream.

4.2.3 Surface water model parameters

Various parameters such as catchment and sub-catchment geometry, *i.e.* hydraulic length, catchment width and area; catchment topography and slope; land use, geology and soil type; permeable and impermeable areas were required to model the hydrological process. The estimation of the parameters was as follows:

i. Sub-catchment geometry length, width and area – the estimation of the area was based on the catchment delineation; the hydraulic length was set equal to the longest watercourse. The width parameter was determined as the ratio of the catchment area to the hydraulic length. These parameters were essential in the model calibration process and were determined to be very sensitive, *i.e.* even minor changes in parameter values have an impact on the model results.

ii. Catchment topography and slope – the topography and slope were extracted automatically from the LIDAR DEM.

iii. Land use – the land use was based on land use maps acquired from CCT and Google Maps linked via the PCSWMM software. Parameters included the percentage impervious area; percentage routed to pervious; and depression storage (pervious and impervious).

iv. Geology and soil type - the geology and soil types were determined from soil maps acquired from CCT and Adelana *et al.* (2010).

v. Infiltration parameters – infiltration parameters were estimated from soil samples collected from various locations in the catchment in a study linked to this project entitled 'Infiltration Potential of Stormwater Ponds in the Zeekoe Catchment Area' (Mavundla, 2018). The infiltration parameters were estimated from on field measurements using a Double Ring Infiltrometer (DRI) combined with laboratory experiments on samples brought back from the field. The parameters estimated included: maximum infiltration rate in dry soil, minimum infiltration rate in saturated soils, infiltration rate decay constant *i.e.* the rate at which infiltration rate of the soil decreases as it is saturated, soil particle size distribution, in-situ soil density, soil porosity, soil air void ratio, permeability and drying time.

4.2.4 Modelling runoff

The representation of the surface water hydrological process in PCSWMM is based on the conservation of mass and momentum equations that govern the unsteady flow of water through a drainage network of channels and pipes (James *et al.*, 2010). In this study, the dynamic wave routing method in PCSWMM was selected to solve the complete one-dimensional Saint Venant Continuity and Momentum equations as presented in Equation 4-4 and Equation 4-5 respectively (James *et al.*, 2010). It was selected over the other approaches, *i.e.* steady flow routing and the kinematic wave routing approach as the study area is relatively flat and the simulation needed to account for possible backwater effects (James *et al.*, 2010). It also accounts for possible pressure build-up in closed pipes and temporary channel storage (James *et al.*, 2010).

Continuity Equation $$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0$$ **Equation 4-4**

Momentum Equation $$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\,\frac{\partial H}{\partial x} + gA\left(S_f - H_L\right) = 0$$ **Equation 4-5**

Where: Q – flow rate through the conduit (m^3/s); x – length of the conduit (m); H – hydraulic head of water in the conduit (m); A – cross-sectional conduit area (m^2); t – simulation time (s); S_f – friction slope; H_L – local energy loss per unit length of conduit; g – acceleration of gravity (m^3/s).

4.2.5 Modelling infiltration

Infiltration in PCSWMM can be represented by Horton, Green-Ampt or Curve Number methods (James *et al.,* 2010). These methods estimate the component of rainfall that is converted to infiltration in the model. The selection of the appropriate approach to apply in the model was based on the best match with field measured data from a study linked to this project entitled 'Infiltration Potential of Stormwater Ponds in the Zeekoe Catchment Area' (Mavundla, 2018). The field experiments were undertaken with a Double Ring Infiltrometer (DRI) at three sites across the study area (Figure 4-9 and Table 4-3).

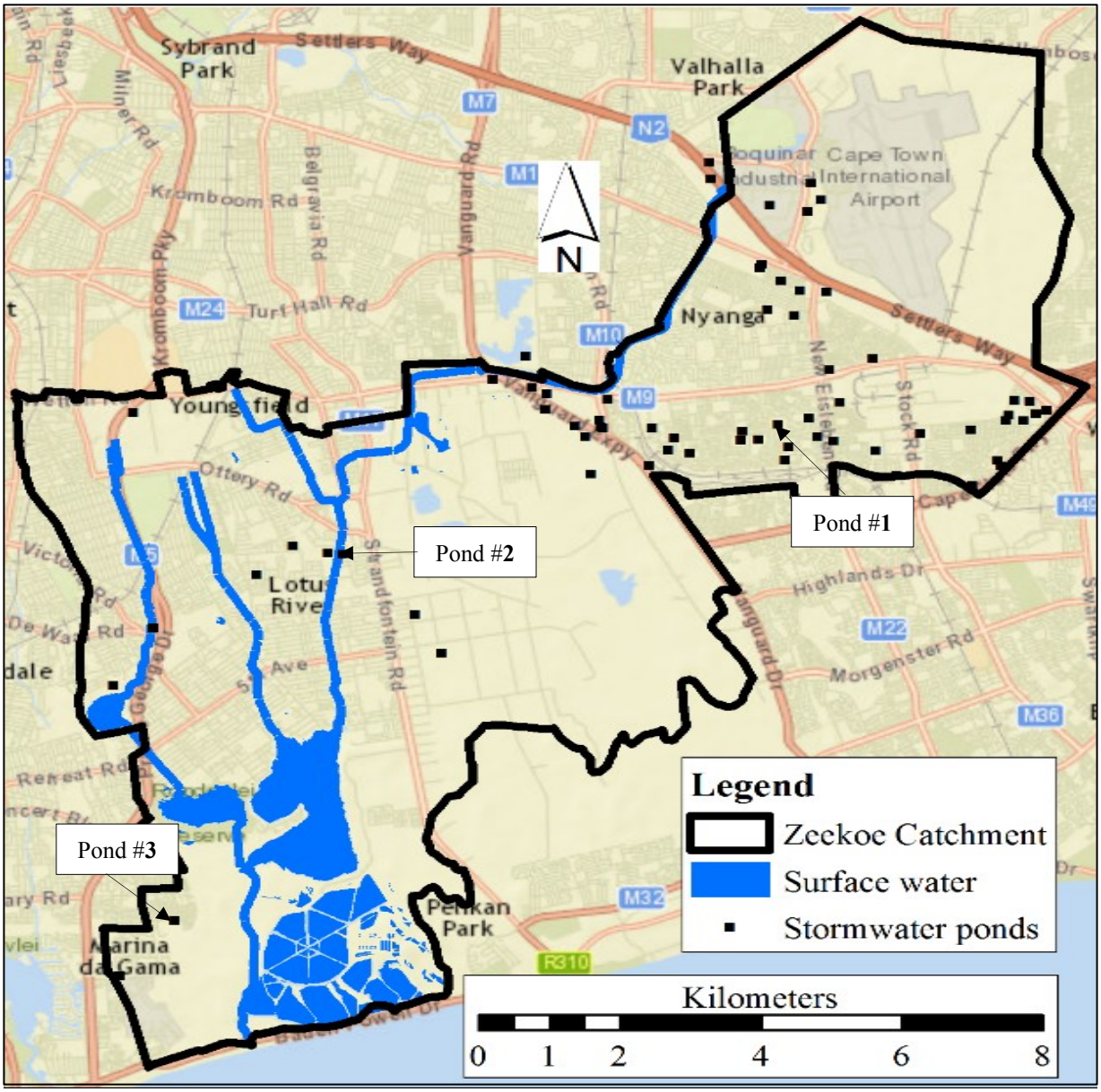


Figure 4-9 The three infiltration test sites (*after* CCT, 2015)

Table 4-3 Detail and locations of the selected stormwater ponds (Mavundla, 2018)

Pond No.	Pond Type	Surface Area (m^2)	Suburb Name	Road Name	Latitude	Longitude	Elevation (m.a.s.l)
1	Detention	32,000	Browns Farms	2309 Msingizane Street	-34.009	18.581	33
2	Retention	10,000	Lotus River	7 Eric Way	-34.025	18.519	15
3	Detention	9,000	Vrygrond	86 Drury Road	-34.087	18.484	8

A summary of the method adopted by Mavundla, (2018) is as follows:

i. A total of 18 infiltration tests (*i.e.* six tests per selected stormwater pond) were carried out including two sets of DRI tests (directly on the surface of the pond and at 200 mm below the surface) at three locations in each of the selected stormwater ponds.

ii. The sub-surface DRI test was required to indicate the change in infiltration rates after scraping off 200 mm of topsoil. The topsoil consisted of compacted fine soil particles deposited by runoff onto the surface of the stormwater ponds, thus altering the properties of the floor over time;

iii. The volumes of infiltrated water were read from graduated burettes maintaining a constant head of 50 mm in both rings.

iv. Readings of the burettes were made at 6-minute intervals until equilibrium was reached, *i.e.* insignificant change in water levels with time.

v. The estimated infiltration rates were plotted on a graph and the Horton's and Green-Ampt equations were fitted to the data to determine the most appropriate method.

The laboratory and field experiments were aimed at determining the general infiltration parameters of the catchment including *inter alia* infiltration rates and porosity. The estimate of infiltration rates at the selected sites was made in accordance with the ASTM D3385-09 and the constant-head method. The diameters of the inner and outer rings of the DRI were 300 mm and 600 mm respectively. The DRI apparatus was firmly inserted into the ground with the rings penetrating the soil to a depth in the range of 80 – 150 mm. To undertake the laboratory experiments, 300 mm shallow surface core-samples were retrieved from each test location and taken to the laboratory for further analysis. The laboratory tests included falling head experiments to determine saturated hydraulic conductivity. Sieve analysis and ASTM D2216 – 10 standard test for moisture content were undertaken to determine various physical properties: *i.e.* bulk density, volumetric water content, porosity, saturation, residual water content, particle density, and particle-size distribution analysis. The data was plotted on a graph on the log-scale of the percentage particles passing versus sieve size (grain size) (Figure 4-10). A summary of the findings are presented in Table 4-4.

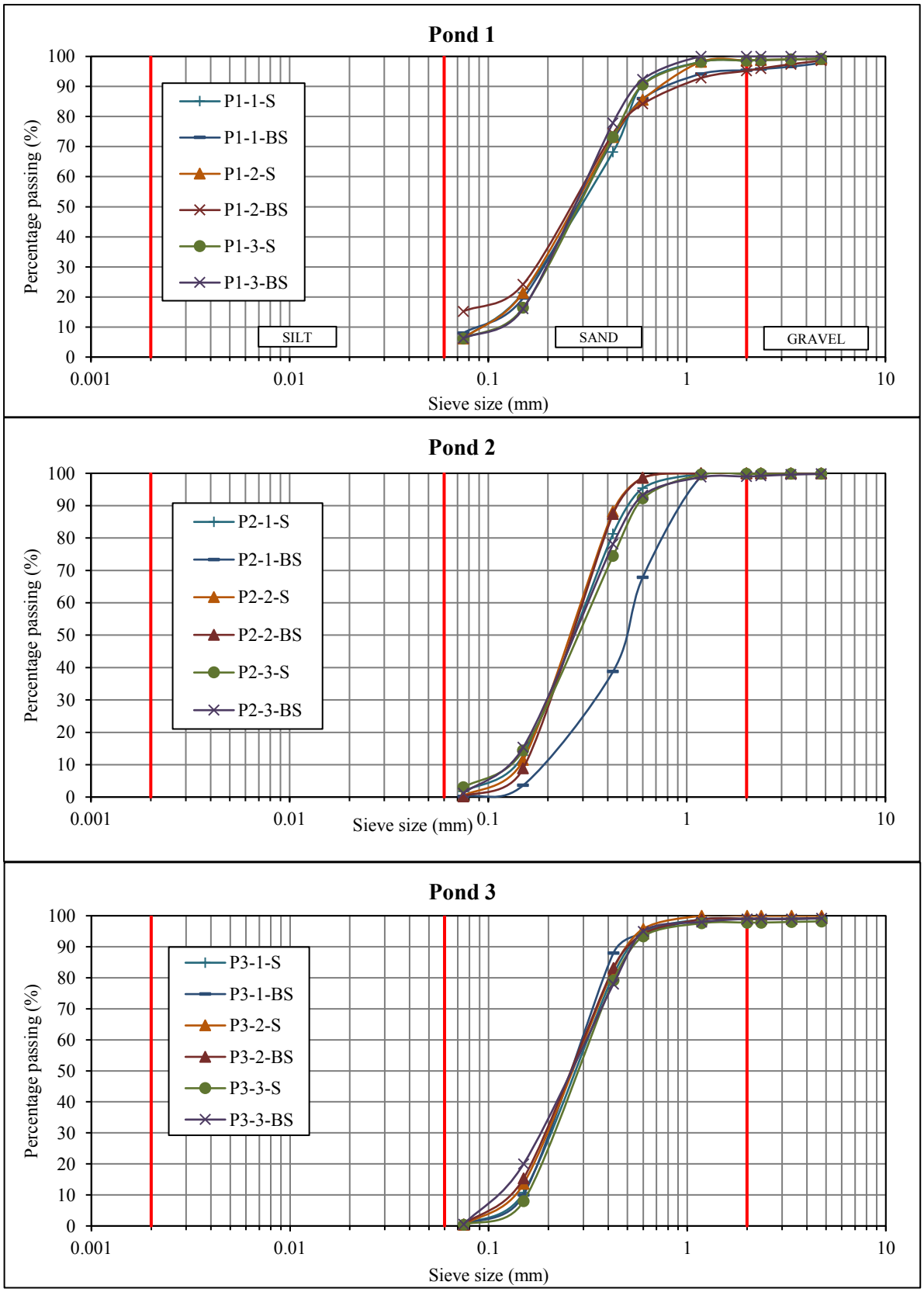


Figure 4-10 Percentage particle passing versus sieve size (*after* Mavundla, 2018)

Table 4-4 Summary of findings from field and laboratory experiments (Mavundla, 2018)

Property		Units	Pond 1		Pond 2		Pond 3	
			*Surface	**Below Surface	*Surface	**Below Surface	*Surface	**Below Surface
Soil Texture	Fines	%	6.2	7.8	1.7	0.5	0.4	0.6
	Sand		92.4	89.1	98.2	99.1	98.5	98.4
	Gravel		1.4	3.1	0.1	0.3	1.1	1.0
Effective Grain Size	d10	mm	0.1	0.1	0.15	0.17	0.16	0.14
	d30		0.19	0.19	0.20	0.25	0.21	0.19
	d60		0.35	0.33	0.32	0.39	0.32	0.30
Coefficients of Uniformity and Curvature (-)	Cu	-	3.53	3.57	2.15	2.32	1.30	1.36
	Cc	-	1.01	1.14	0.85	0.98	0.85	0.86
Soil Group		-	SP-SM	SP-SM	SP	SP	SP	SP
Porosity		%	32	33	44	30	43	38
Void Ratio		-	47	49	78	43	77	61
Specific Gravity		-	2.61	2.60	2.49	2.60	2.56	2.58
Bulk Density		kg/m3	1733	1889	1460	1917	1635	1930
Saturated Density		kg/m3	2091	2074	1834	2278	1892	1981
Conductivity (K20°C Constant head)		cm/hr	4.8	4.8	19.9	11.1	10.5	10.3
Natural Moisture Content		%	6	8	5	5	13	17
* Depth (<200 mm); ** Depth (>200 mm); SP-SM – poorly graded sand with silt; SP – poorly graded sand								

The results show similarities in the soil particle distribution for all the selected ponds across the study area. Furthermore, the other characteristics such as porosity and coefficient of uniformity and curvature, specific gravity and natural moisture content were similar. These similarities justify the reliance on a limited number of test sites to provide general infiltration parameters for the study area. The infiltration rates measured with the DRI experiments at the three ponds were then compared with values estimated with the Green-Ampt and Horton methods to determine the most appropriate approach to be used in the model by plotting them all on the same graph. In Horton's method, the decay of infiltration rate with time is expressed with an exponential relationship as shown in Equation 4-6 (Horton, 1933).

$$f = f_c + (f_o - f_c)e^{-\lambda t} \qquad \textbf{Equation 4-6}$$

Where: f – infiltration rate at any time t (cm/hr); f_o – initial infiltration rate at t – 0 (cm/hr); f_c – final infiltration rate (after equilibrium at steady state) at $t = t_c$ (cm/hr); λ – Horton's decay coefficient which depend on soil characteristics and vegetation cover (hr^{-1}).

The Equation 4-6 was re-arranged to Equation 4-7 and plotted as $ln\,(f - f_c)$ *vs* t (Subramanya, 2001). The initial and final infiltration rates were determined in the field with the DRI.

$$ln(f - f_c) = ln(f_o - f_c) - \lambda t \qquad \textbf{Equation 4-7}$$

From the experiments in Mavundla (2018), the statistical descriptors for the parameters in Equation 4-7 were determined as presented in Table 4-5.

Table 4-5 Horton's method infiltration parameters (*after* Mavundla, 2018)

Statistical measures based on six tests per site	Pond 1			Pond 2			Pond 3		
	λ (hr^{-1})	f_o (cm/hr)	f_c (cm/hr)	λ (hr^{-1})	f_o (cm/hr)	f_c (cm/hr)	λ (hr^{-1})	f_o (cm/hr)	f_c (cm/hr)
Mean	1.9	3.6	1.4	0.6	25.8	20.6	0.8	17.80	10.54
Minimum	1.0	0.7	0.3	0.1	9.3	3.8	0.3	5.82	2.22
Maximum	3.0	5.8	2.9	1.3	31.8	28.2	2.4	41.18	22.80

In the Green-Ampt method, the determination of infiltration rate is based on Darcy's law with the formula as shown in Equation 4-8 (Green & Ampt, 1911).

$$f = k(1 + \frac{\gamma S_c}{F}) \qquad \textbf{Equation 4-8}$$

Where: f – infiltration rate (cm/hr); F – cumulative infiltration (cm); k – hydraulic conductivity (cm/h); S_c – capillary suction at the wetting front (cm); γ – porosity of the soil (%).

The estimation of the parameters required the Equation 4-8 to be re-arranged to Equation 4-9.

$$f = k + \frac{n}{F} \qquad \textbf{Equation 4-9}$$

Where: k and n are parameters of the infiltration model.

The infiltration rates measured with the DRI experiments and values estimated with the Green-Ampt and Horton methods are plotted in Figure 4-11. This shows that the Green-Ampt method represented initial values better than the Horton's approach in Ponds #1 and #3. After equilibrium had been reached, however, the Horton's approach represented the measured asymptotic infiltration rate curve better than the Green-Ampt method in Ponds #1 and the general trend in Pond #2. In Pond #3, the Green-Ampt method represented the infiltration rates better than Horton's approach after equilibrium but the typical asymptotic infiltration rate curve was not obtained in this case.

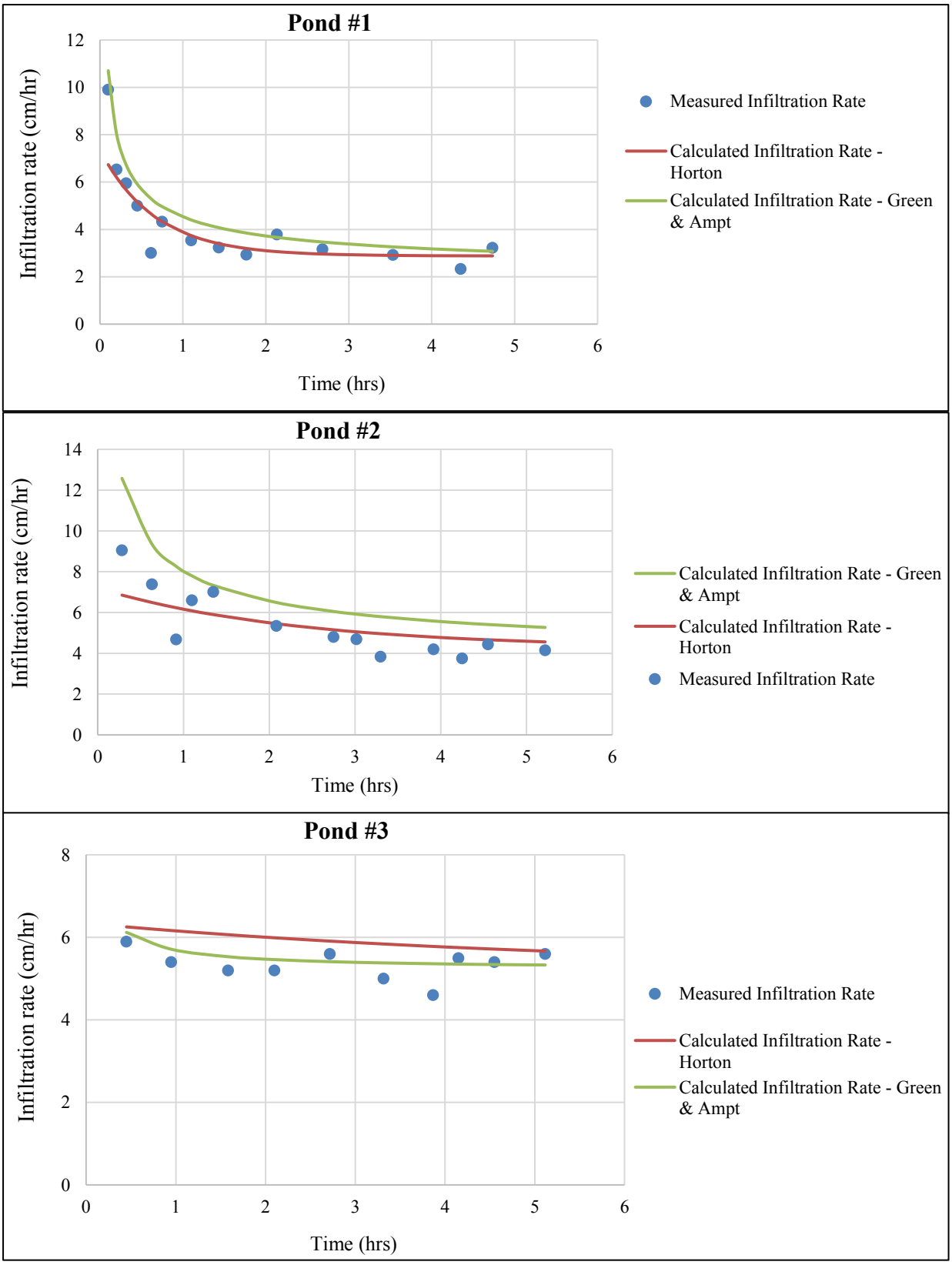


Figure 4-11 Compared measured and calculated infiltration rate (*after* Mavundla, 2018)

The statistical descriptors of the parameters in Equation 4-9 determined from the experiments in Mavundla (2018), are presented in Table 4-6.

Table 4-6 Green-Ampt method infiltration parameters (*after* Mavundla, 2018)

Ponds	Parameters	Mean	Minimum	Maximum
Pond 1	*k* (cm/h)	1.2	0.3	2.3
	γ (%)	31	30	33
	Sc (cm)	7.3	0.1	23.1
Pond 2	*k* (cm/h)	22.1	3.8	32.6
	γ (%)	40	30	45
	Sc (cm)	31.4	22.7	36.8
Pond 3	*k* (cm/h)	9.6	0.1	21.6
	γ (%)	38	32	50
	Sc (cm)	11.1	1.9	30.5

There was difficulty in driving DRI rings into the ground at Pond #3 due to hard compacted soils. The compacted soil presented difficulty for the infiltration test which likely explains the absence of the characteristic asymptotic curve and minimal difference between the initial and final infiltration rates. Both methods represented the final infiltration rates well in all three ponds. Therefore, the Nash–Sutcliffe Efficiency (NSE) and correlation (R^2) coefficients were also calculated to determine the method that provided a better match (Table 4-7).

Table 4-7 Correlation coefficient for measured and calculated infiltration rates

	Pond 1		Pond 2		Pond 3	
	NSE (%)	R^2 (%)	NSE (%)	R^2 (%)	NSE (%)	R^2 (%)
Calculated Infiltration Rate – Horton	0.73	0.79	0.61	0.77	-2.74	0.09
Calculated Infiltration Rate – Green & Ampt	0.75	0.92	0.44	0.81	0.005	0.25

The Green-Ampt and Horton's methods both provided reasonable approximations of infiltration rates except in Pond #3 with compacted soils. The final selection of Horton's method to model the infiltration component was thus based on the benefits of being able to specify an infiltration decay rate in stormwater ponds to account for gradual clogging.

4.2.6 Catchment model calibration

The hydrological model for the Zeekoe Catchment was developed and calibrated as accurately as possible to reasonably estimate the harvestable stormwater volume. The stepwise calibration and verification process was as follows:

i. The rainfall data measured at five-minute time intervals to represent the fast runoff processes that result in short response times in urban catchments was used for the model development and calibration. Various researchers (*e.g.* Neumann *et al*., 2011; Seo *et al*., 2015) have recommended that at least a ten-year long rainfall time-series with several dry, normal and wet years be used in the development and calibration of a catchment model. The high temporal resolution rainfall data was available for the period 2012 – 2015. The flow data needed for the calibration process was also limited to the period 2012 – 2015. The disaggregation of the available long-time series rainfall data measured at a daily interval was not undertaken to extend the five-minute time data as the generated values would not have corresponding flow data for the calibration process.

ii. A sensitivity analysis was undertaken to determine the uncertain parameters that had the greatest impact on the model results to guide the model calibration process. They were determined to be catchment width, impervious area, infiltration, and depression storage;

iii. A manual calibration was initially undertaken where the values of the sensitive parameters were changed by trial and error. The selection of suitable values to apply was guided by visual inspection to assess the improvements achieved in how the output from the model mimicked the observed flows;

iv. Finally, an automatic calibration was undertaken to fine-tune and optimise the results using the Sensitivity-based Radio Tuning Calibration (SRTC) tool available in PCSWMM; and

v. All the calibrated parameters were inspected to confirm that they were within acceptable ranges as recommend in the 'Rules for Responsible Modeling' (James, 2005).

After completion of the model calibration process, an assessment was undertaken to determine the reliability of the results from the model. According to Moriasi *et al.*, (2007), various statistical methods such as Nash-Sutcliffe efficiency (NSE), per cent bias (PBIAS), and Root Mean Square Error (RMSE) can be used. The statistical evaluation techniques that are available in PCSWMM are Integral Square Error (ISE), Nash-Sutcliffe Efficiency (NSE), Coefficient of determination (R^2), Standard Error of Estimation (SEE), Simple Least Squares (LSE), Simple Least-Squares dimensionless (LSE dim), Root Mean Square Error (RMSE) and Root Mean Square Error dimensionless (RMSE dim). Since the model performance evaluation is based on statistics, the selection of the events to be used in the assessment needed to satisfy the 'independence' criteria requirement. According to Willems, (2009), events are considered to be independent if the inter-event period exceeds the recession time. Furthermore, the lowest flow value on the recession leg of the event should be below a threshold considered as base flow (Willems, 2009). A total of ten

events were identified based on these two considerations for the model performance evaluation. As shown in Figure 4-12, the model calibration provided reasonable results, *i.e.* NSE > 0.50 and R^2 > 0.90 as recommended in a study by Moriasi *et al.*, (2007).

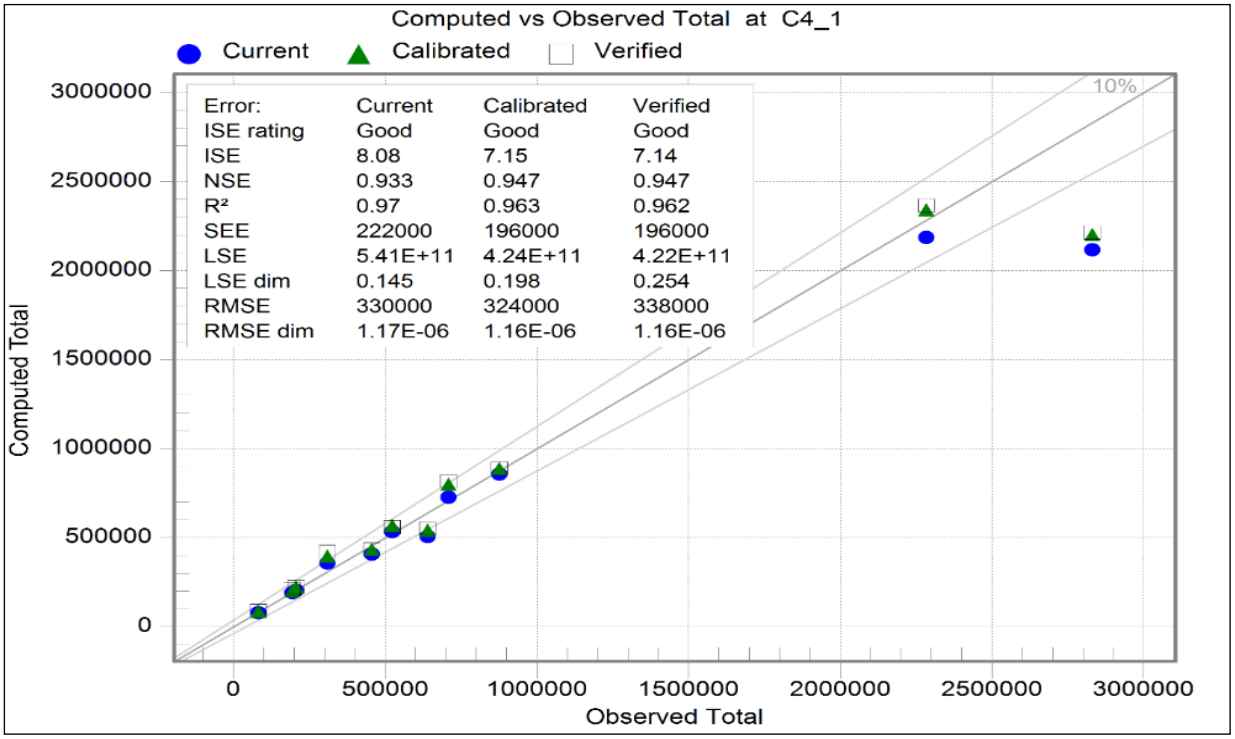


Figure 4-12 Calibration and verification results of flow volume totals (m^3)

4.2.7 Model verification

Verification of the model outputs was undertaken to confirm the reliability of the results with regards to the estimation of the total runoff volume. As shown in Figure 4-12, the model verification also provided reasonable results, *i.e.* NSE > 0.50 and R^2 > 0.90 as recommended in studies (Dawson *et al.*, 2007; Moriasi *et al.*, 2007; Willems, 2009). The scatterplot in Figure 4-12 with standard deviation represented by the solid lines allows visual assessment of the correlation relationship between the computed model results and observed values. According to Willems (2009), model calibration should aim to minimise the standard deviation by reducing the horizontal and vertical distance of the points from the 45° bisector line shown in Figure 4-12. Presence of high scatter (*i.e.* large deviation from 45° bisector) is an indication of high uncertainty and bias in the model which would be a source of errors in the estimation and prediction of flow volumes (Willems, 2009). In the calibration and verification processes, the extent of scattering and deviation from the 45° bisector was minimised to 10%. The summary of the results from the calibration and verification processes including model continuity errors and routing continuity errors are as shown in Table 4-8.

Table 4-8 Calibration and verification results of total runoff vs observed time series

	Observed vs. Calibrated	Observed vs. Verified
Integral Square Error rating	Good	Good
Integral Square Error	7.15	7.14
Nash Sutcliff Efficiency	0.947	0.947
R^2	0.963	0.963
Runoff quantity continuity error		
Flow routing continuity errors	1.207	0.027
Highest continuity errors at nodes	Node J529 (6.98%); Node J643 (3.73%); Node J527 (1.57%)	

4.3 Stormwater ponds adapted for water supply

4.3.1 Characteristic of the storage components

As discussed in Chapter 3, the Zeekoe Catchment has some 61 stormwater ponds and three shallow lakes (vleis) with the potential to be adapted to function as surface water storage and supply for SWH. The descriptive statistics including a variation of sizes and geometric shapes of the ponds is presented in Table 4-9.

Table 4-9 Geometric features of the stormwater ponds

	Volume (m^3)	Surface Area (m^2)	Depth (m)
Maximum	140,185	88,094	2.12
75th Percentile	27,968	18,946	1.62
Mean	20,835	13,319	1.57
25th Percentile	4,001	2,957	1.50
Minimum	1,957	1,727	1.23

The stormwater ponds were modelled with Real-Time-Control (RTC) techniques discussed in Section 4.3.2 to determine if the effective storage could be increased as used for SWH and supply.

4.3.2 Selection of appropriate RTC modelling approach

The simulation of RTC for SWH in the Zeekoe Catchment considered rainfall data, control rules and actuator settings. Rainfall forecast data was acquired from the Global Forecast System (GFS) model managed by the National Centre for Environmental Prediction (NCEP) and used for the RTC application. The GFS model provides global forecasts up to two weeks prediction in a spatial form. The GFS model also provides time series rainfall forecasts at a three-hourly temporal resolution. The data is available in the study area, at latitude 34° 00′ S and longitude

18° 31′ E, from 6th May 2011 (system commencement date) to two weeks after the date of download. The available GFS forecast data were extracted for the period 2011 – 2017 and compared with measured data in the study area. It was determined that there were some differences in the timing of the peak (shift in peak times) and the magnitude of the events as shown in Figure 4-13. Although the difference in magnitude of peak and associated volume was minimal for most events (*i.e.* less than 10% difference in 80% of events), some peaks in GFS data were 30% higher than recorded data.

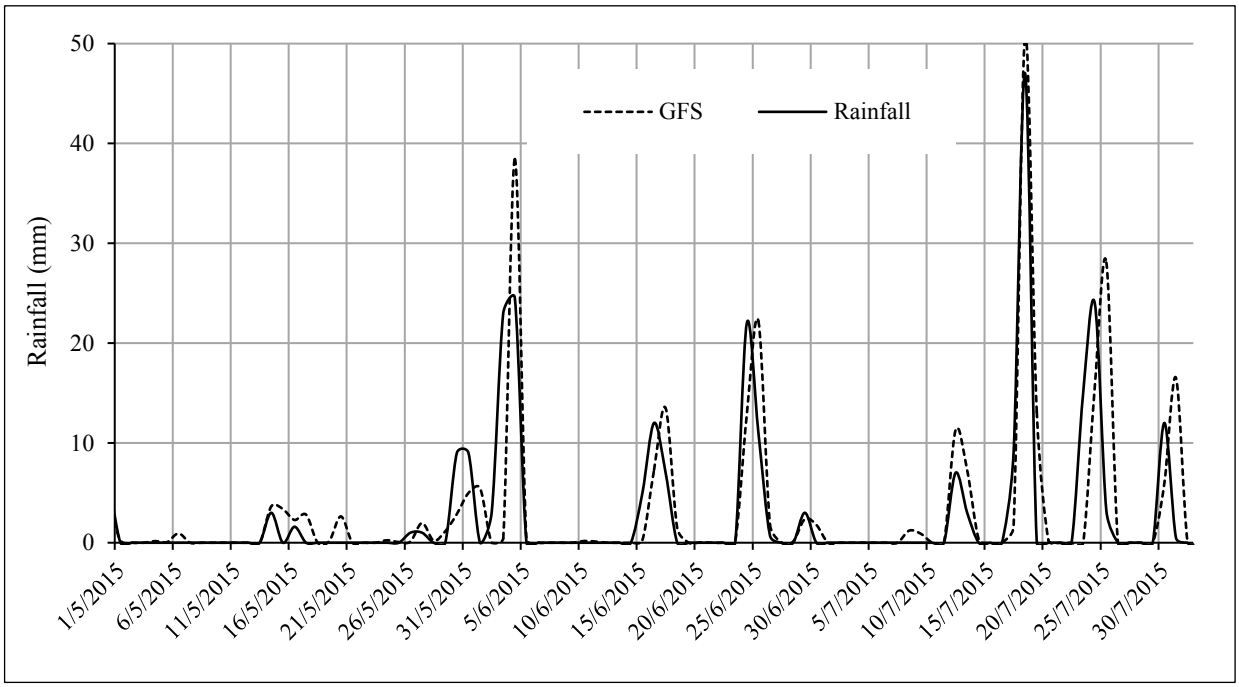


Figure 4-13 Comparison GFS and measured rainfall data (NOAA, 2017; CCT, 2015)

The disadvantage of an overestimation in the forecast would be in the release of water from storage with no subsequent occurrence of a flood. Since one of the main aims of RTC is to ensure the release water from storage whenever there was forecasted rainfall that might cause flooding, the GFS data was not adjusted as emptying ponds based on an overestimation would account for lower volumes from actual storms. PCSWMM provides various options to dynamic model management of water levels and outflow from storage units with control rules. The options considered included the following.

- **Control rules linked to specific water levels and inflow rate values** – is a local Rule-Based Control management approach that incorporates 'if-then' rules (*i.e.* if this happens, then do this) with adjustments made concerning prevailing conditions as discussed in Section 2.3.5. An example of the 'control rule' syntax applied on one of the ponds is given with results in a plot as shown in Figure 4-14.

Rule SU1A
If Node SU1 Depth = 0
And Node J23 Inflow < 5(based on capacity and predetermined rate of filling)
THEN Orifice OR1 Setting = 0 (outlet completely closed)
Priority 1 (Rule takes first priority)

Rule SU1B
If Node SU1 Depth <=1 (based on depth and capacity of storage unit)
And Node J23 Inflow < 10 (based on capacity and predetermined rate of filling)
THEN Orifice OR1 Setting = 0.5 (outlet partially open)
Priority 2 (Rule takes second priority)

Rule SU1C
If Node SU1 Depth >1.5 (based on depth and capacity of storage unit)
And Node J23 Inflow >15 (based on capacity and predetermined rate of filling)
THEN Orifice OR1 Setting = 1 (outlet completely open)
Priority 3 (Rule takes third priority)

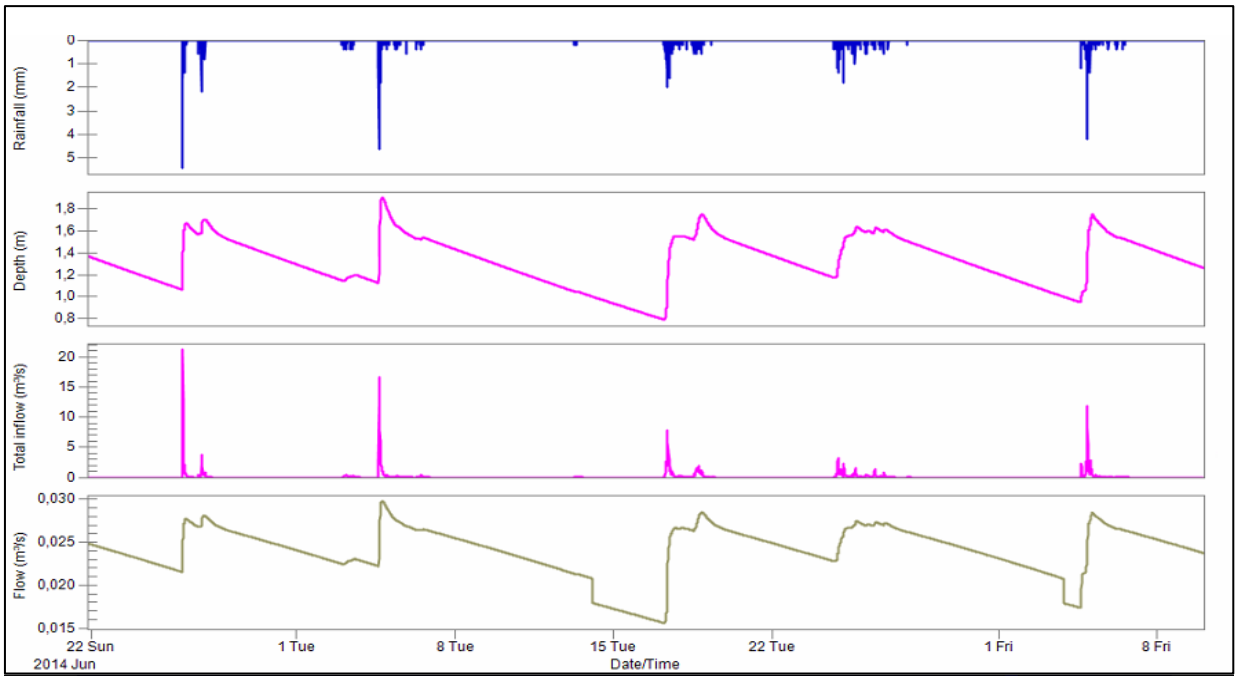


Figure 4-14 Sample results with control rules linked to specific values

The use of the option is common due to simplicity and straightforwardness of site-specific control with any errors limited to the site and independent of the whole system (USEPA, 2004; García *et al.*, 2015). The downside of the option is that many commands and syntax are required for each storage unit limiting the flexibility of the operation. Secondly, the approach does not provide

the benefits of a regional control of outflows from various storage units linked to a downstream reservoir as required in the study.

- **Control rules linked to specific open/close times** – it is a possible to regulate outflows from storage with a 'time to open/close' option. An example of the 'control rule' syntax associated with the operation is as follows:

  Rule SU1A
  If Simulation date > = 5/31/2015
  And Simulation date < = 6/5/2015
  THEN Conduit C1 STATUS = OPEN
  ELSE Conduit C1 STATUS = CLOSED

  In this case, the outlet from the storage unit is fully open for the entire duration of a forecasted rainfall which would generate runoff that exceeds available capacity. The success of the option depends on the reliability and accuracy of the timing and magnitude of the forecasted rainfall data. As shown in Figure 4-13, the difference in forecasted and measured data would result in an inaccurate determination of the 'open and close' periods.

- **Proportional-Integral-Derivative (PID) controller** – PIDs may be used to model controlled outflows from storage with a generic closed-loop which continuously adjusts the system with corrective actions to provide desired conditions (James *et al.,* 2010). In the closed-loop, the three PID parameters provide an opportunity to empirically tune the system to converge towards desired pre-defined conditions. The output from the PID controllers is defined as shown in Equation 4-10 (James *et al.,* 2010).

$$w.l(t) = K_p e(t) + \frac{K_p}{T_i}\int e(\tau)d\tau + K_p T_d \frac{de(t)}{dt} \qquad \textbf{Equation 4-10}$$

$$P = K_p e(t); \quad I = \frac{K_p}{T_i}\int e(\tau)d\tau; \quad D = K_p T_d \frac{de(t)}{dt} \qquad \textbf{Equation 4-11}$$

Where $w.l$ (t) – water level; K_p – proportional coefficient; $e(t)$ – error (difference between desired and actual water level); T_i – integral time, $e(\tau)$ – integral time error; T_d – derivative time; t – simulation time step; P – proportional controller; I – integral controller; D – derivative controller.

In the process of system adjustment, various PID values were assessed to determine the most suitable options for certain conditions such as magnitude of storm and capacity of

pond. The selected PID controller parameter values were iteratively modified with a control strategy as follows:

i. If there were no forecasted rainfall, the stormwater would be held in the pond until there was capacity in the vleis downstream where the abstraction for water supply was planned.

ii. If there were forecasted rainfall that exceeded available capacity, RTC control rules were set to allow for the pre-emptive drawdown of water levels to provide for capacity in the stormwater ponds to avoid flooding;

An example of the 'control rule' syntax associated with the operation was as follows:

Rule SU1A
IF Node SU1 Depth = 0; Pond empty
AND Node J23 Inflow < 5; flow less than 5 m^3/s
THEN Orifice OR1 Setting = PID 1 -1 -1; direct action control
ELSE Orifice OR1 Setting = PID -1 1 1; reverse action control

For example, P was initially set at 0.01, and other values (I, D) given 0 values. The P value was then changed stepwise to values such as 0.1, 1, 10, -0.1, -1 until there was no added advantage. With the P value locked, I and D values were also adjusted. Finally, the model was run with optimised PID values. The water level variation from a selected stormwater pond with a comparison of scenarios with and without application of PID controllers are presented in Figure 4-15. In the comparison of water levels variation for the controlled case (with PID application) and without RTC in Figure 4-15, the following key features that needed to be achieved with the application of RTC can be observed.

i. The water depth remained constant and declined relatively slowly for the controlled case (with PID application) where there was no risk of flooding *i.e.* extended detention as required for SWH.

ii. The water depth dropped rapidly to accommodate anticipated flow from forecasted rainfall *i.e.* RTC control rules were set to allow for the pre-emptive drawdown of water levels to provide for capacity in the stormwater ponds to avoid flooding;

It was determined that RTC with PID controller option was sensitive to inflow rate and variation of the water level. The parameters guided the continuous management of the open and close operation of the outlet from the storage unit. The option was independent of the simulation time and was thus suitable for the study when the dynamic management of the outlet was needed.

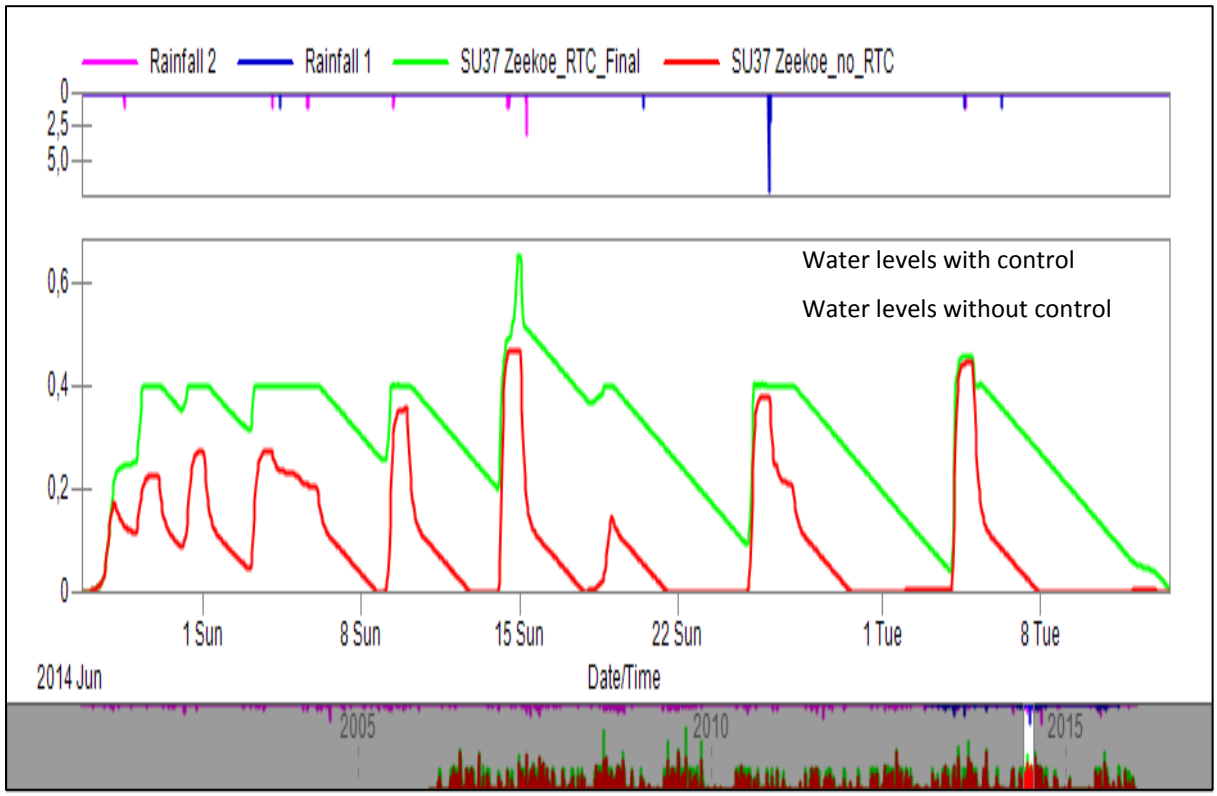


Figure 4-15 Water level variation in the selected stormwater pond (1 m maximum depth)

4.4 Modelling the stormwater harvesting process

4.4.1 Method for modelling the stormwater harvesting process

There are two fundamental approaches used to model SWH with storage units, *i.e.* Yield After Spillage (YAS) and Yield Before Spillage (YBS) (Mitchell *et al.*, 2008). The mathematical models that describe the operating rules are presented in Equations 4-10 and 4-11:

YAS: $Y_t = min(D_t, max(V_{t-1} - V_d, 0);\ V_t = min(V_{t-1} + I_t + P_t,\ V_{cap}) - Yt$ **Equation 4-12**

YBS: $Y_t = min(D_t, max(V_{t-1} - V_d, 0);\ V_t = min(V_{t-1} + I_t + P_t,\ Y_t\ V_{cap})$ **Equation 4-13**

Where Y_t – yield *i.e.* volume taken from the storage for water use at current time t; D_t – demand at current time t; V_t – storage volume at the end of the current time-step; V_{t-1} – storage volume at the end of the previous time-step; V_d – dead storage volume; I_t – inflow into the storage at current time t; P_t – current incident precipitation volume; and V_{cap} – maximum storage capacity.

The selection of an approach and time step to model SWH was based on previous case studies (*e.g.* Mitchell *et al*., 2008; Campisano & Modica, 2014). The YAS approach is the most widely used method and also provides more conservative results compared with YBS (Palla *et al.,* 2011; Campisano & Modica, 2014). In the selection of an appropriate time step to model SWH, studies (*e.g.* Campisano & Modica 2014) recommended short time-steps (*i.e.* minutes) for small storage elements such as rainwater tanks to account for the rapid changes in the water levels. On the other hand, in another study that used YAS method with large storage units (Mitchell *et al*., 2008), it was determined that there was an insignificant difference in results from SWH with daily and six-minute time steps. To assess the impact of time scale in this study area, model results of SWH at five minutes and daily time steps were compared. The mean annual volume of harvestable stormwater on a daily time scale was 6% less than the modelled values based on the five minutes interval. With the minimal difference in results, SWH was thus modelled at a daily data time scale to reduce the computational load. It was undertaken using the historical long-time rainfall data sets, and the impact of climate change on the volume of stormwater assessed using data from climate change prediction models. This data was available on a daily time scale for both long-time historical rainfall and climate change prediction models.

4.4.2 The modelled volume of harvestable stormwater

Some studies (*e.g.* Neumann *et al*., 2011; Seo *et al*., 2015) have recommended modelling SWH with at least ten-year rainfall time-series. Others (*e.g.* Herrmann & Schmida, 2000; Konig, 2001; Yuan *et al*., 2003; Liaw & Tsai, 2004; Mitchell, 2007; Roebuck, 2007; Neumann *et al*., 2011; Seo *et al*., 2015) proposed periods of 20, 30 and over 50 years. To assess the effect of time series length, SWH was modelled with 10-year (2006 – 2015) and 20-year period (1996 – 2015). The mean monthly modelled flow volumes over the periods are presented in Figure 4-16.

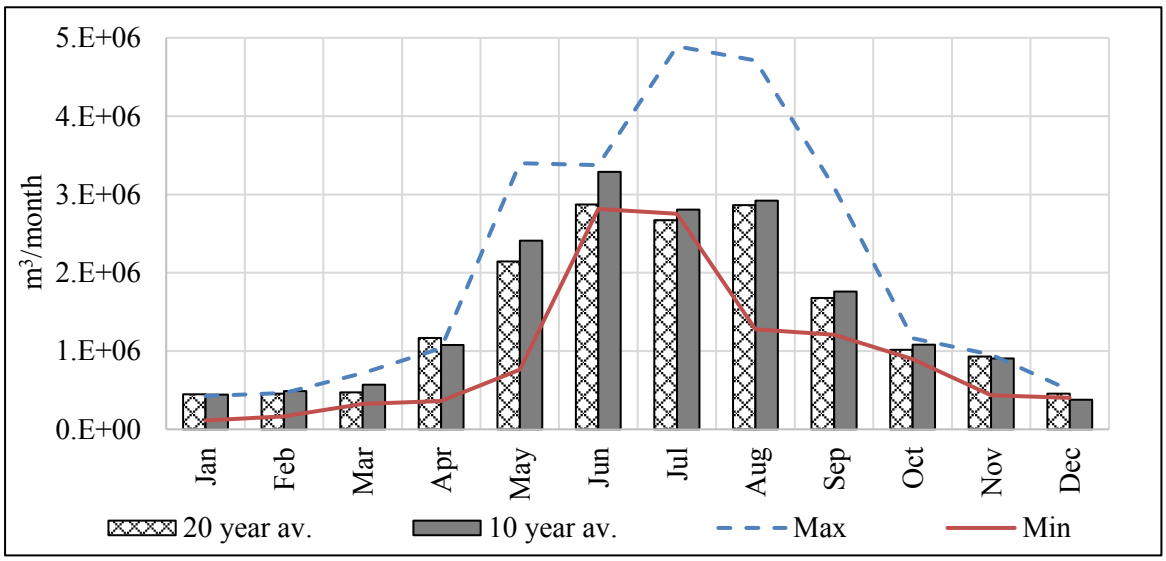


Figure 4-16 Modelled mean annual flow volume and limits in the study area

The mean annual modelled flow volumes over the ten-year period (2006 – 2015) and 20-year (1996 – 2015) period were 18 Mm^3 and 17 Mm^3 respectively. The difference in the mean annual modelled flow volumes in the two periods was likely due to the presence of relatively drier years included in the 20-year period compared with the 10-year period. Flow values from the wettest year (2013) and driest year (2015) were also extracted and plotted against the mean values to indicate limits, *i.e.* maximum and minimum as shown in Figure 4-16. The mean annual modelled flow volumes for the wettest and driest years were 25 Mm^3 and 12 Mm^3 respectively.

4.4.3 Impact of land use change on the stormwater resource

The impact of land use change was estimated with the assistance of the Cape Town Spatial Development Framework (CTSDF), which provided planned developments per suburb up to 2040 as shown in **Figure 4-17** (CCT, 2012).

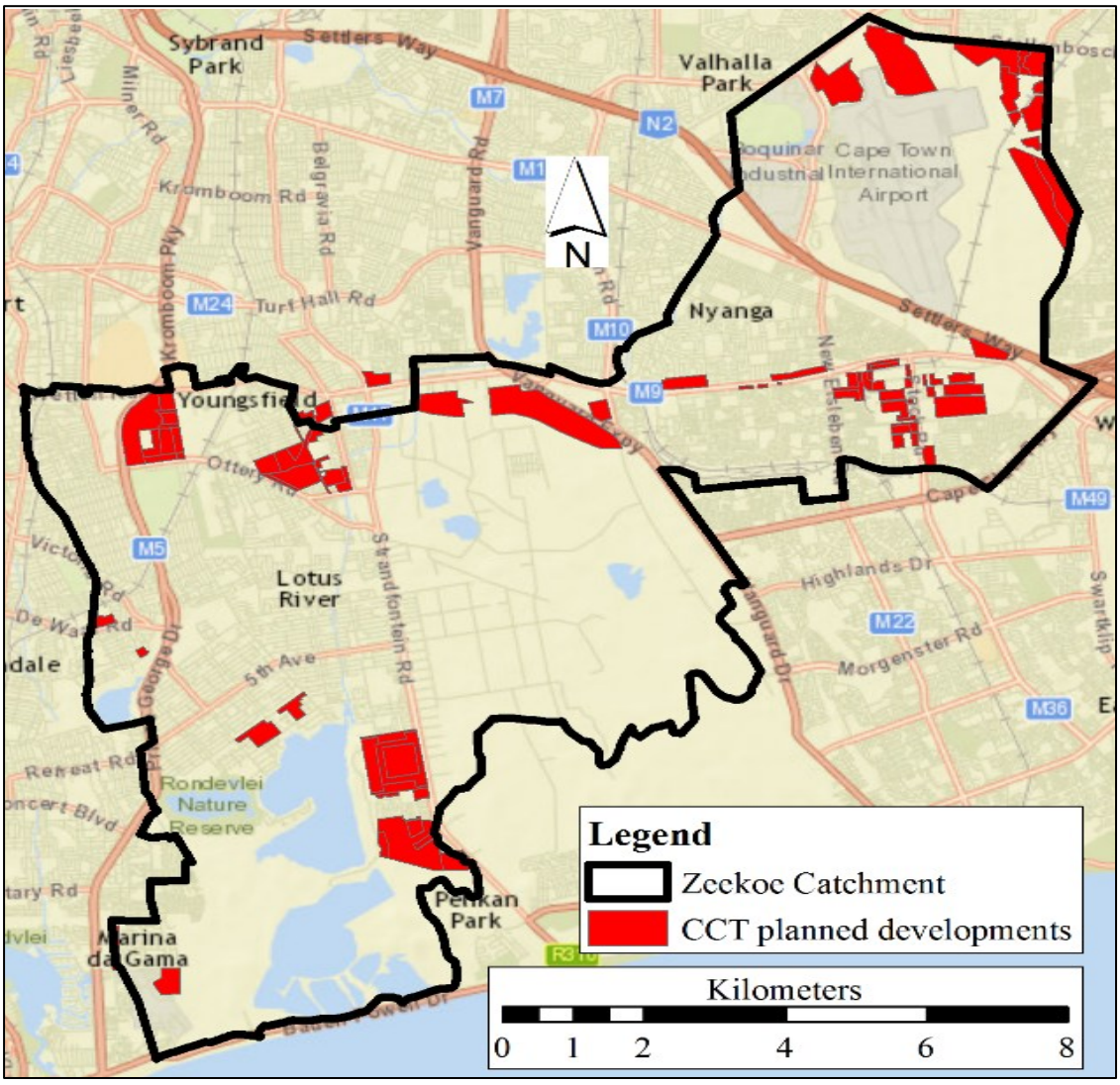


Figure 4-17 CCT planned developments in the study area (CCT, 2012)

With the planned development and future land use change, some natural 'greenfield' areas will be converted to impervious surfaces – although this can be mitigated through suitable Sustainable Drainage Systems (SuDS). The level of imperviousness as a result of the land use change without application of SuDS was estimated from the CTSDF (CCT, 2015). The land-use changes only provided for an additional 5% imperviousness in the study area up to 2040. An assessment was thus carried out to determine the potential increase in harvestable stormwater from surface runoff with 50% imperviousness corresponding to the planned development up to 2040. Furthermore, an assessment was undertaken to determine the potential increase in mean annual harvestable stormwater beyond 2040 with development scenarios using theoretical imperviousness of 75% as a worst-case situation. The results are shown in Figure 4-18. It was noted that with an increase in imperviousness, there was a corresponding increase in the potential harvestable water resource as surface runoff – in turn requiring additional storage to enable its capture for re-use.

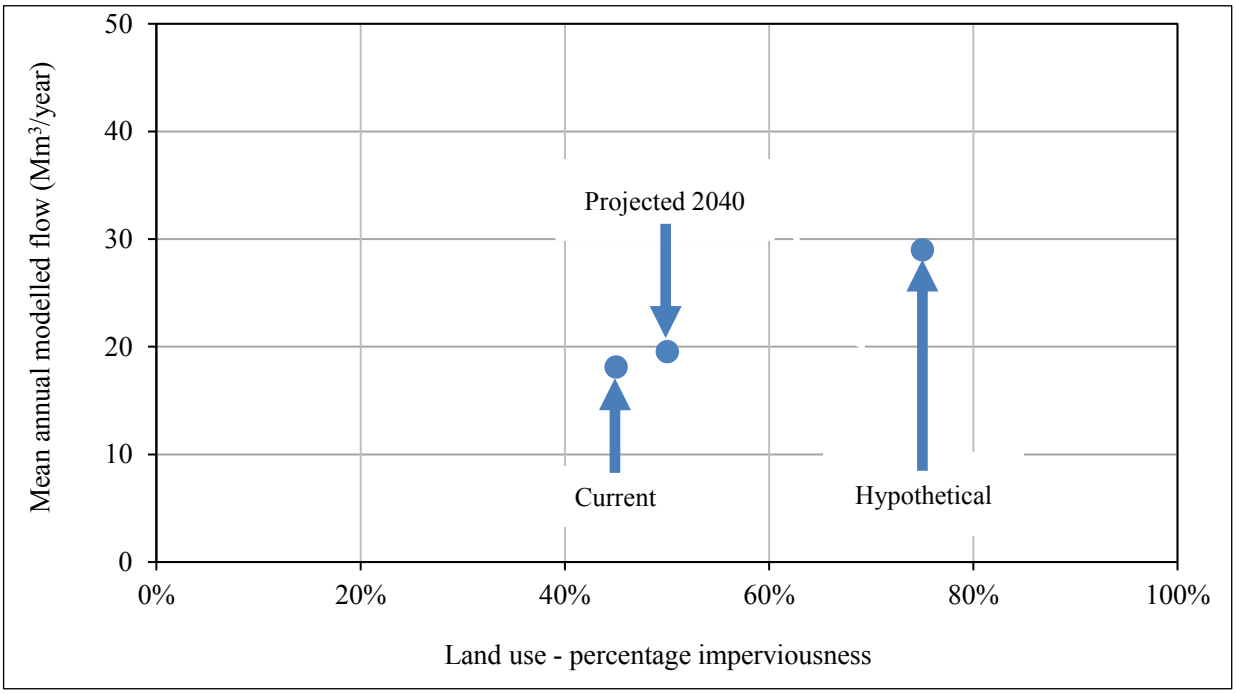


Figure 4-18 Plot of runoff increase as a function of land use

4.4.4 Climate change impacts

4.4.4.1 Overview

The assessment of climate change impact was based on rainfall and temperature data from the 26 models from UCT – CSAG as discussed in Section 3.6.4 and Section 4.1.4. The data from 25 climate change prediction models show that the climate is getting drier and the impact would be a likely reduction in harvestable stormwater. Only one model, HadGEM2-CC-rcp85, showed that the climate would be slightly wetter than the historical conditions. The data from the climate change models also show significant variability, a characteristic that was identified in the long

time-series historical and future rainfall data. Figure 4-19 presents an example of data from one of the models showing the variability in the rainfall and the succession of wet and dry years.

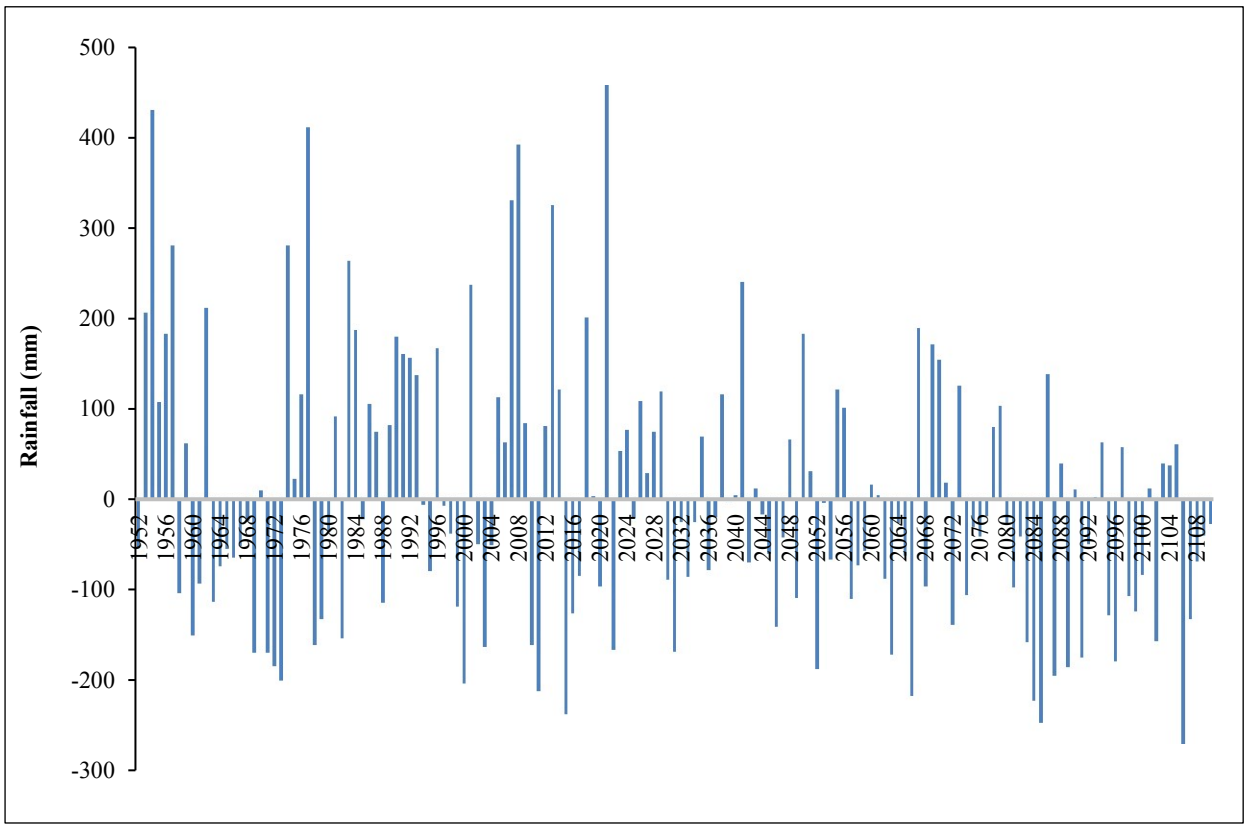


Figure 4-19 Annual rainfall variation from mean of modelled period (2006 – 2015) (*after* Hewitson & Crane, 2006)

Although the rainfall variability appears to be on a downward trend with progressively lower rainfall in the future, there will still be some wet years. It is important to note that the predicted data from the climate change models is highly unlikely to be exactly replicated, however, the ensemble provides an indication of the range of possibilities. It seems that the likelihood of dry years will increase, and wet years decrease, towards the end of the century. SWH was modelled with the data from each of the 26 climate models to determine the likely impact of climate change on future stormwater volumes. The mean annual harvestable surface water resource for the future period 2090 – 2100 was estimated and compared with the historical period of 2006 – 2015 as the base case and the volumetric change is as shown in Figure 4-20. The climate predictions show a mean decrease of 30% in potential harvestable surface water resource with some models showing over 50% reduction (Figure 4-20).

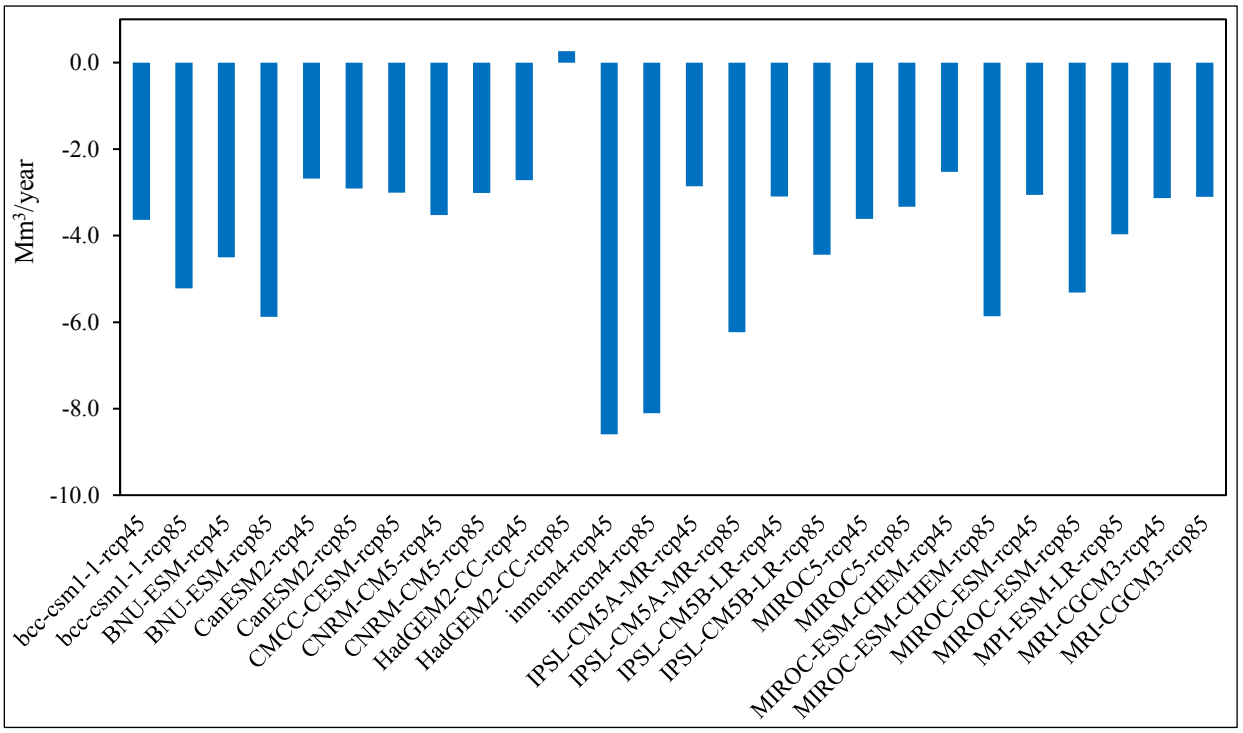


Figure 4-20 Change in potential future surface water resource from the study catchment

The primary cause of the decrease is mainly due to the reduction in total rainfall and increase in evapotranspiration as shown in Figure 4-21.

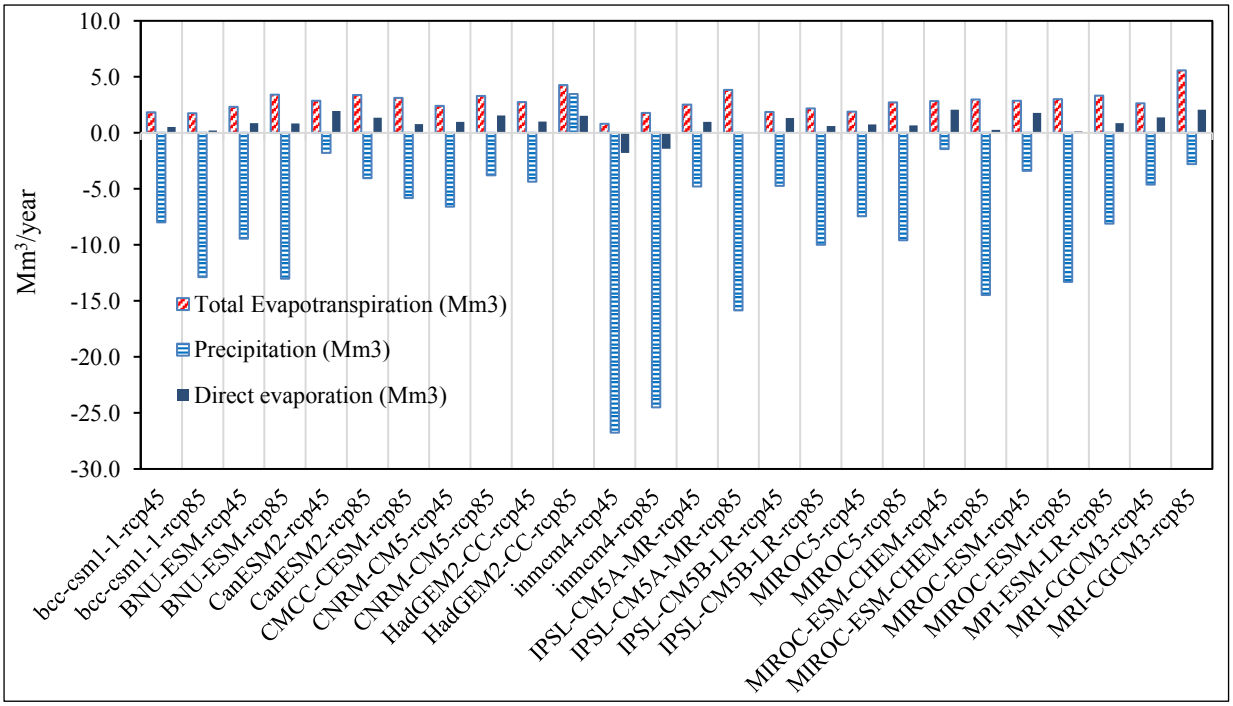


Figure 4-21 Change in rainfall and evapotranspiration in the study catchment

4.5 Summary

The data, hydrological modelling and application of RTC for SWH in the study area has been described in this chapter. It was determined that the potential harvestable stormwater from the Zeekoe Catchment is about 18 Mm^3 (9% of Cape Town water demand in 2018). The mean annual modelled volumes for the wettest and driest years are 25 Mm^3 and 12 Mm^3 respectively. With the planned development and future land use change, an additional 5% imperviousness in the study area was projected for 2040. An assessment was undertaken to determine the potential increase in mean annual harvestable stormwater including beyond 2040 with theoretical imperviousness of 75% as a worst-case situation. The results show significant increase in the potential harvestable water resource as surface runoff – in turn requiring additional storage to enable its capture for re-use. The 26 climate change prediction models also show a likely decrease in harvestable stormwater volume of 3 – 9 Mm^3 (15 – 50% of mean annual modelled volumes).

5. Stormwater harvesting using aquifer storage

In this chapter, the method applied and results relating to the opportunity for SWH using aquifer storage are provided in three sections. Section 5.1 discusses the modelling of the surface to groundwater transfer *i.e.* Managed Aquifer Recharge (MAR) and estimation of recharge volumes based on PCSWMM model (hydrological model discussed in Chapter 4). Section 5.2 discusses the modelling of groundwater abstraction (groundwater recovery after Managed Aquifer Recharge) with a trial section (1.44 km^2 with a single pond) and at catchment scale (89 km^2 with 61 ponds). Section 5.3 and 5.4 present the setup and results of the trial section and catchment scale models.

5.1 Stormwater transfer to groundwater storage

5.1.1 Overview

An assessment was carried out with the aid of PCSWMM to determine the potential stormwater transfer to groundwater storage through MAR with the infiltration being primarily carried out in the existing stormwater ponds. Figure 5-1 shows the model representation of surface and groundwater interaction processes in PCSWMM (James *et al.*, 2010). The direction of groundwater flow depends on the height of the saturated zone above the soil layer and surface water top level in the node above aquifer bottom (m).

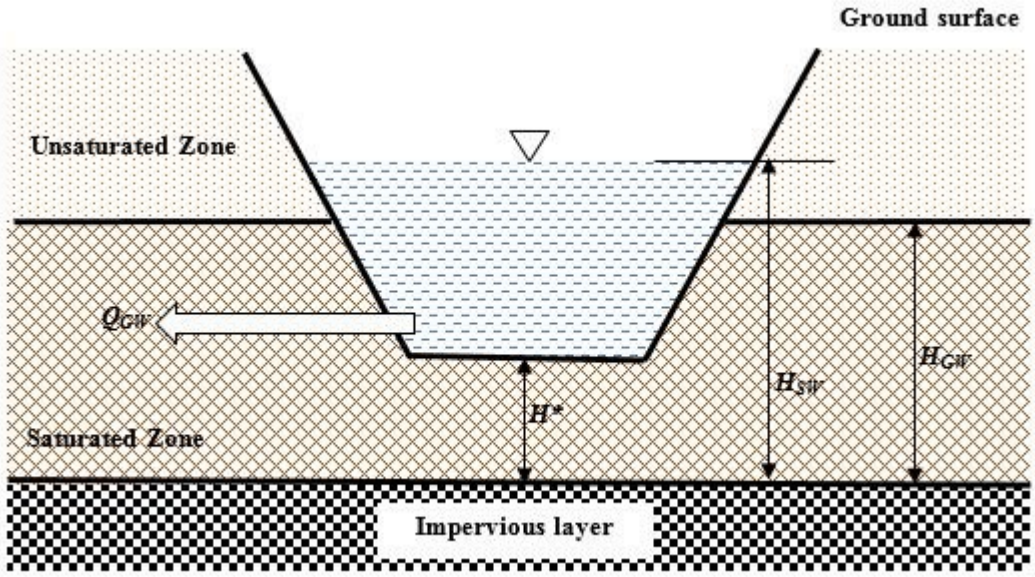


Figure 5-1 the two-zone groundwater layers in SWMM (James *et al.*, 2010)

Where: Q_{GW} – groundwater flow (cms/ha); H_{GW} – height of saturated zone above aquifer bottom (m); H_{SW} – height of surface water top level in the node above aquifer bottom (m); H^* – threshold groundwater height (m).

5.1.2 Modelling the infiltration process

The equations and associated parameters accounting for the infiltration process (surface to groundwater transfer), and groundwater flows in the aquifer are discussed in this section. The study area is particularly well located in a section of Cape Town with a high natural recharge and significant groundwater storage potential (Adelana *et al.*, 2010). The catchment scale infiltration component (a portion of rainfall that is transferred to the groundwater aquifer and not directly contributing to runoff) was modelled as part of the surface water model discussed in Chapter 4. The modeling of stormwater ponds adapted to promote the infiltration process using the available LID / SuDS tools in PCSWMM is described in this chapter. The basic approach adopted in this study was to make use of the existing stormwater ponds that were largely designed for flood control and modify them to provide an additional infiltration function. Since PCSWMM did not have ordinary infiltration cells suitable for MAR, the study used bio-retention cells as the most suitable option available that would also blend well with the stormwater pond environment. Thus the modeling of surface to ground transfer with modification of the existing stormwater ponds to function as infiltration cells was implemented using elements borrowed from bio-retention cell. The use of bio-retention cells has been a subject of various studies such as Clary *et al.*, (2008); Hathaway *et al.*, (2008); Hunt *et al.*, (2008); Trowsdale & Simcock, (2011); Kim *et al.*, (2012); Peng *et al.*, (2016); and Youngblood *et al.*, (2017). These studies have proposed a soil filter layer that includes organic matter, fly-ash and appropriate vegetation (tolerates a wide range of conditions from very dry to very wet; and ideally indigenous to the area) to enhance stormwater quality improvement and assist with the removal of pathogens. Regular maintenance and replacement from time-to-time of the filter media layer to re-establish the designed infiltration rates and stormwater quality improvement benefits are essential. To take advantage of the available aquifer storage in the study area as identified in various studies such as: Henzen, (1973); Tredoux *et al.*, (1980); and Adelana *et al.*, (2006, 2010), this study investigated a case where the stormwater ponds were converted to bio-retention cells to enhance infiltration. A typical bio-retention cell is composed of three horizontal layers, *i.e.* surface, soil and storage layers with an underdrain at the bottom as shown in Figure 5-2 (Brown *et al.*, 2011).

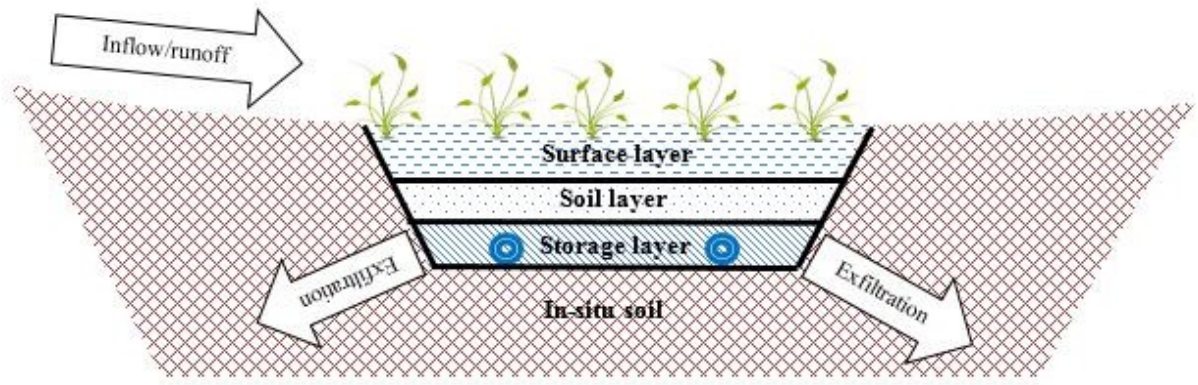


Figure 5-2 A bio-retention cell (*after* Brown *et al.*, 2011)

In the modelling of enhanced infiltration with bio-retention cells in PCSWMM, appropriate values were allocated to the surface, soil, storage layers. These were based on the research undertaken by Mavundla (2018) supplemented by recommendations from various publications (*e.g.* James *et al.*, 2010; Adelana *et al.,* 2010; Brown *et al.*, 2011) as follows:

i. **Surface layer** – the top section of the bio-retention cell that receives both direct rainfall and runoff from the catchment. The modelling of water balance in the section is based on a simple continuity equation (James *et al.*, 2010). The surface layer properties are specific to the bio-retention cells geometric characteristics, *i.e.* surface area and depth (consistent with the stormwater pond shape) and vegetation cover (100% of the pond surface area). The modelling of the water balance in the surface layer was determined from Equation 5-1.

$$\Phi_1 \frac{\partial d_1}{\partial t} = i + q_o - e_1 - f_1 - q_1 \qquad \textbf{Equation 5-1}$$

Where: Φ_1 – fraction of freeboard above the surface not filled with vegetation; d_1 – depth of water in the surface layer (mm); i – precipitation rate falling directly on the surface layer (mm/hr); q_o – inflow to the surface layer from runoff captured from other areas (mm/hr); e_1 – surface layer evapotranspiration rate (mm/hr); f_1 – infiltration rate of surface water into the soil layer (mm/hr); q_1 – surface layer runoff or overflow rate (mm/hr).

ii. **Soil layer –** the middle section of the bio-retention cell generally consists of an engineered soil mixture with organic matter or fly-ash (Clary *et al.*, 2008; Hunt *et al.*, 2008; Peng *et al.*, 2016; Youngblood *et al.*, 2017) and a filter media with a thickness of 450 – 900 mm (James *et al.*, 2010). The soil properties in the study area *i.e.* porosity (0.30 – 0.44), field capacity (15.6 – 17.6%), wilting point (4.4 – 5.2), conductivity (4.8 – 19.9 cm/hr), conductivity slope (9.7 – 9.9 cm/cm) and suction head (5.9 – 114.5 cm) were determined by Mavundla, (2018) are suitable as filter media. The modelling of the water balance in the soil layer was determined from Equation 5-2.

$$D_2 \frac{\partial \theta_2}{\partial t} = f_1 - e_2 - f_2 \qquad \textbf{Equation 5-2}$$

Where: D_2 – thickness of the soil layer (mm); θ_2 – soil layer moisture content (fraction); f_1 – infiltration rate of surface water into the soil layer (mm/hr); e_2 – soil layer evapotranspiration rate (mm/hr); f_2 – percolation rate of water through the soil layer into the storage layer (mm/hr).

iii. **Storage layer** – the bottom section of the bio-retention cell consists of crushed stone or gravel with thickness between 150 – 450 mm, voids ratio (0.47 – 0.78) and filtration rate (25 – 75 cm/hr). The modelling of the storage layer is determined from Equation 5-3.

$$\Phi_3 \frac{\partial d_3}{\partial t} = f_2 - e_3 - f_3 - q_3 \qquad \textbf{Equation 5-3}$$

Where: Φ_3 – voids fraction of storage layer (fraction); d_3 – depth of water in the storage layer (mm); f_2 – percolation rate of water through the soil layer into the storage layer (mm/hr); e_3 – storage layer evapotranspiration rate (mm/hr); f_3 – exfiltration rate of water from the storage layer to native in-situ soil (mm/hr).

In modelling the hydrological performance of bio-retention cells, several assumptions were made:

i. **Surface layer** – the plan area of the stormwater ponds adapted to function as bio-retention cells in PCSWMM was constant for the entire depth, the inflow was uniformly distributed over the entire surface area, and water movement inside the bio-retention cell was one-dimensional in the vertical direction.

ii. **Soil layer –** the moisture content is uniformly distributed throughout the soil layer.

iii. **Storage layer** – the storage layer acts as a reservoir.

The terms in Equation 5-1 to 5-3 are numerically computed in PCSWMM using the properties of the layers in the bio-retention cells (*i.e.* surface, soil and storage); climate data (rainfall and evaporation); soil characteristics (porosity and voids ratio); and the features of bio-retention cells (vegetation cover, depth and surface area). The information on aquifer depth for the study area was based on various studies such as Adelana *et al.,* (2010); Geber, (1981); and Henzen, (1973); and surface elevation was from LIDAR DEM data from CCT. The data were interpolated across the catchment to determine the mean aquifer depth and surface elevation at all of the stormwater ponds in the study area as shown in Figure 5-3.

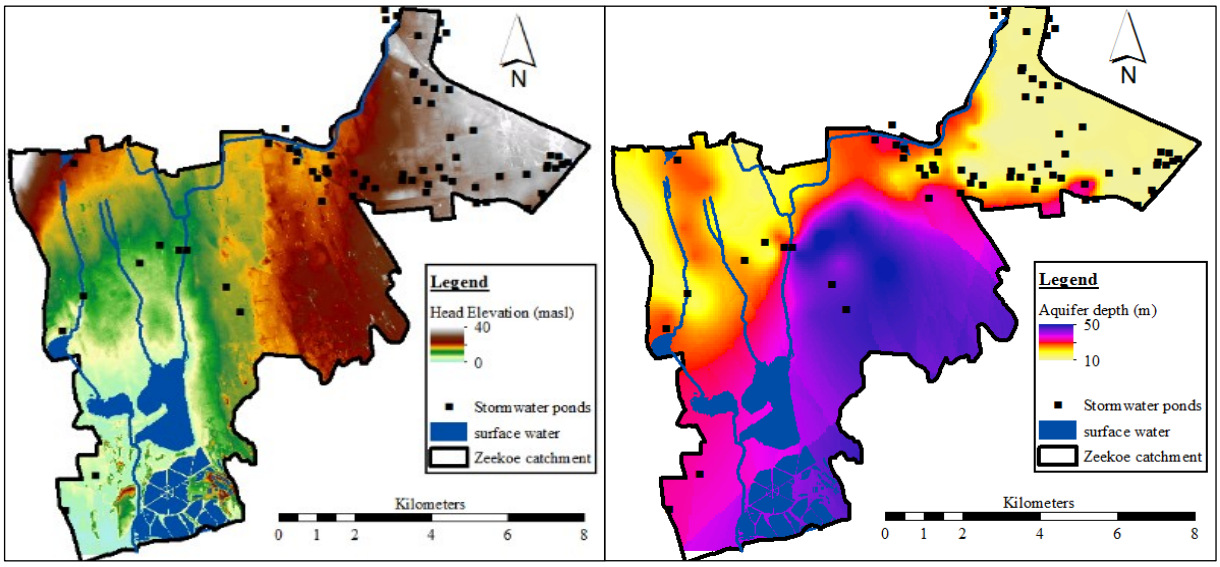


Figure 5-3 Aquifer depth and surface elevation (*after* Adelana *et al.*, 2006; CCT, 2015)

5.1.3 Estimation of the supplemental groundwater resource

The estimation of the groundwater resource, *i.e.* stormwater transferred to aquifer storage with infiltration in bio-retention cells was modelled in PCSWMM as discussed in Chapter 4. The modelled mean annual values for evaporation, evapotranspiration, surface runoff and infiltration for the cases of pre-construction and post-construction over the modelled period (2006 – 2015) are presented in Figure 5-4.

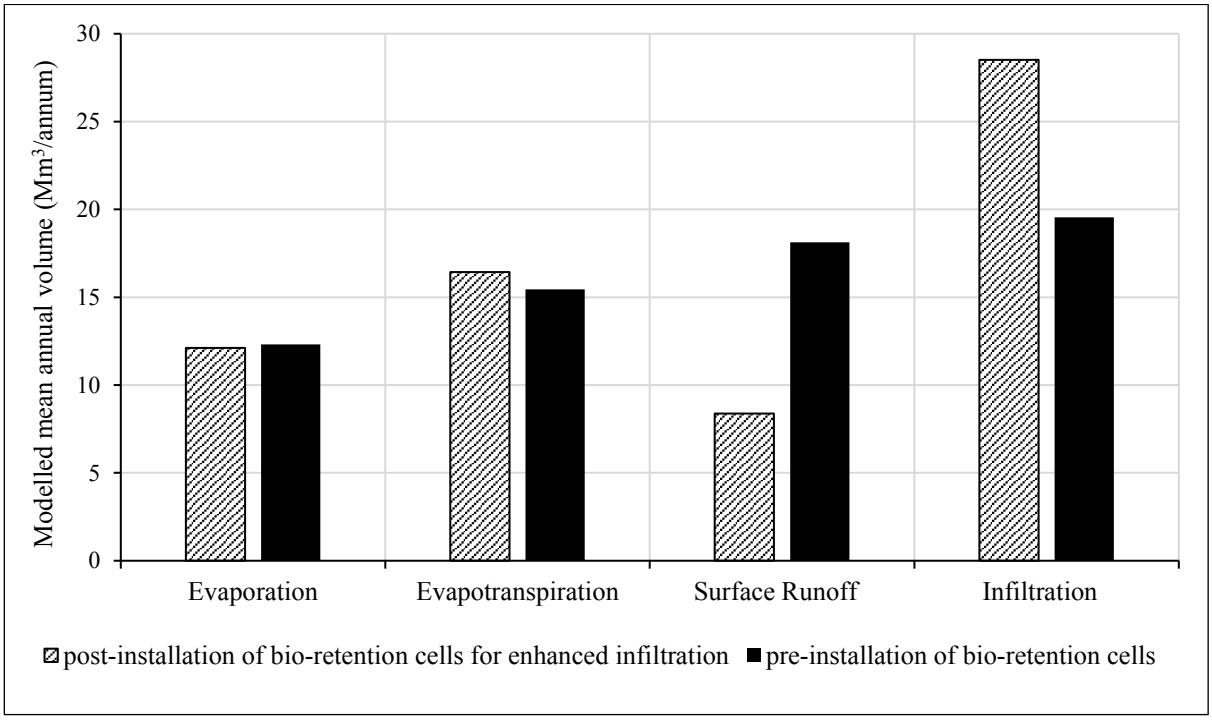


Figure 5-4 Comparison of existing and post-modification to bio-retention (45% imperviousness)

As shown in Figure 5-4, the model indicated significant infiltration even before adoption to bio-retention. This can be attributed to large sections of the study area with rural farmland characteristics where natural recharge takes place. Furthermore, the study area consists of physical characteristics that support natural infiltration including *inter alia* sandy soils (pervious) and reasonably flat terrain (slopes generally less than 3%). The expected future population growth typical in urban areas will likely – without the adoption of a Sustainable Drainage System (SuDS) approach – result in the natural 'greenfield' areas converted to impervious surfaces. To determine the impact of land use change, a case was assessed where land development typical in urban areas increased imperviousness to a theoretical 75% value, and the results are given in Figure 5-5.

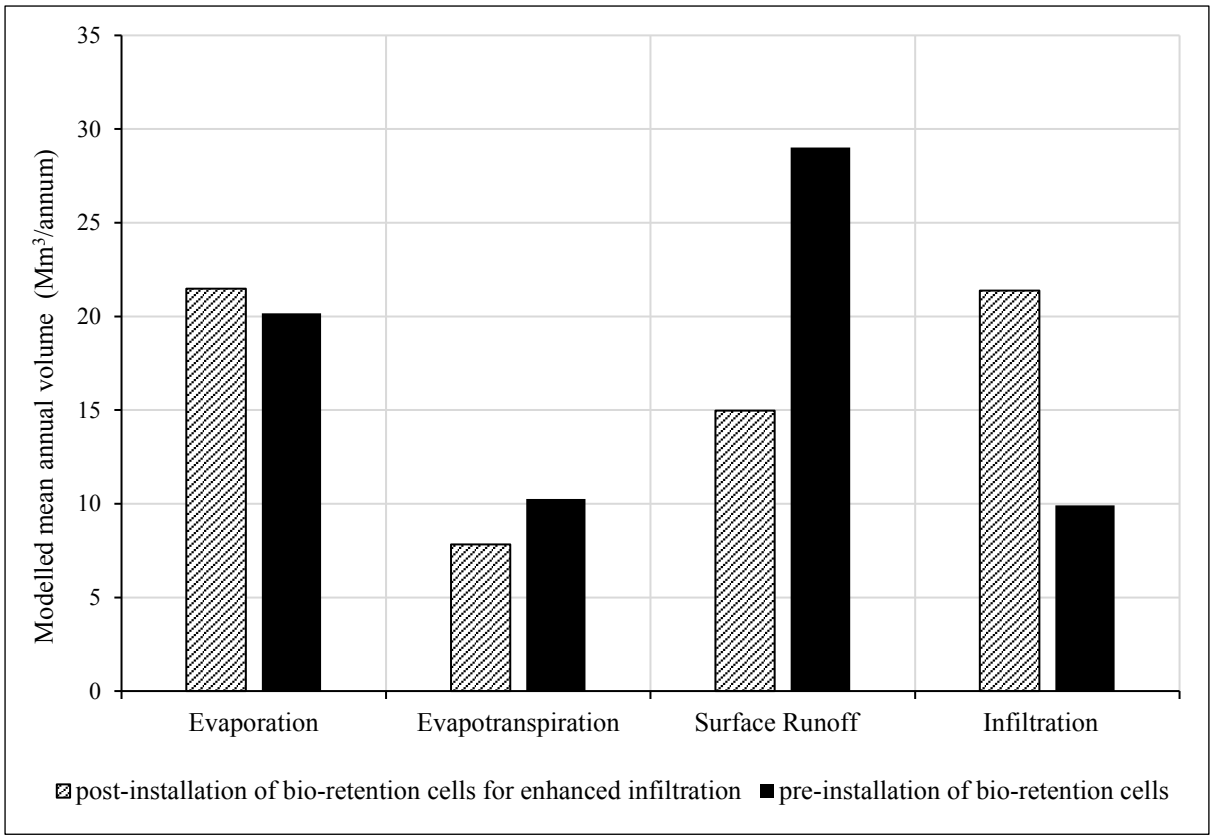


Figure 5-5 Comparison of existing and post-modification to bio-retention (75% imperviousness)

As shown in Figure 5-5, with an increase in imperviousness there was a corresponding increase in the potential harvestable water resource as surface runoff – in turn requiring additional storage to enable its capture. However, increased imperviousness also results in decreased natural infiltration. In this case, infiltration via bio-retention cells would provide for transfer to the large and available groundwater aquifer.

5.2 Modelling groundwater abstraction

5.2.1 Groundwater abstraction model structure

The groundwater abstraction was modelled in MATLAB using an approach presented in Mahinthakumar & Sayeed (2005; 2006) using data from previous research including, *inter alia*, 'A conceptual model for the development and management of the Cape Flats aquifer, South Africa' (Adelana *et al.*, 2010); 'Managed aquifer recharge potential for the Cape Flats Aquifer' (Mauck, 2017); and Cape Flats Aquifer and False Bay – 'opportunities to change' (Hay *et al.*, 2015). The model structure is presented in Figure 5-6.

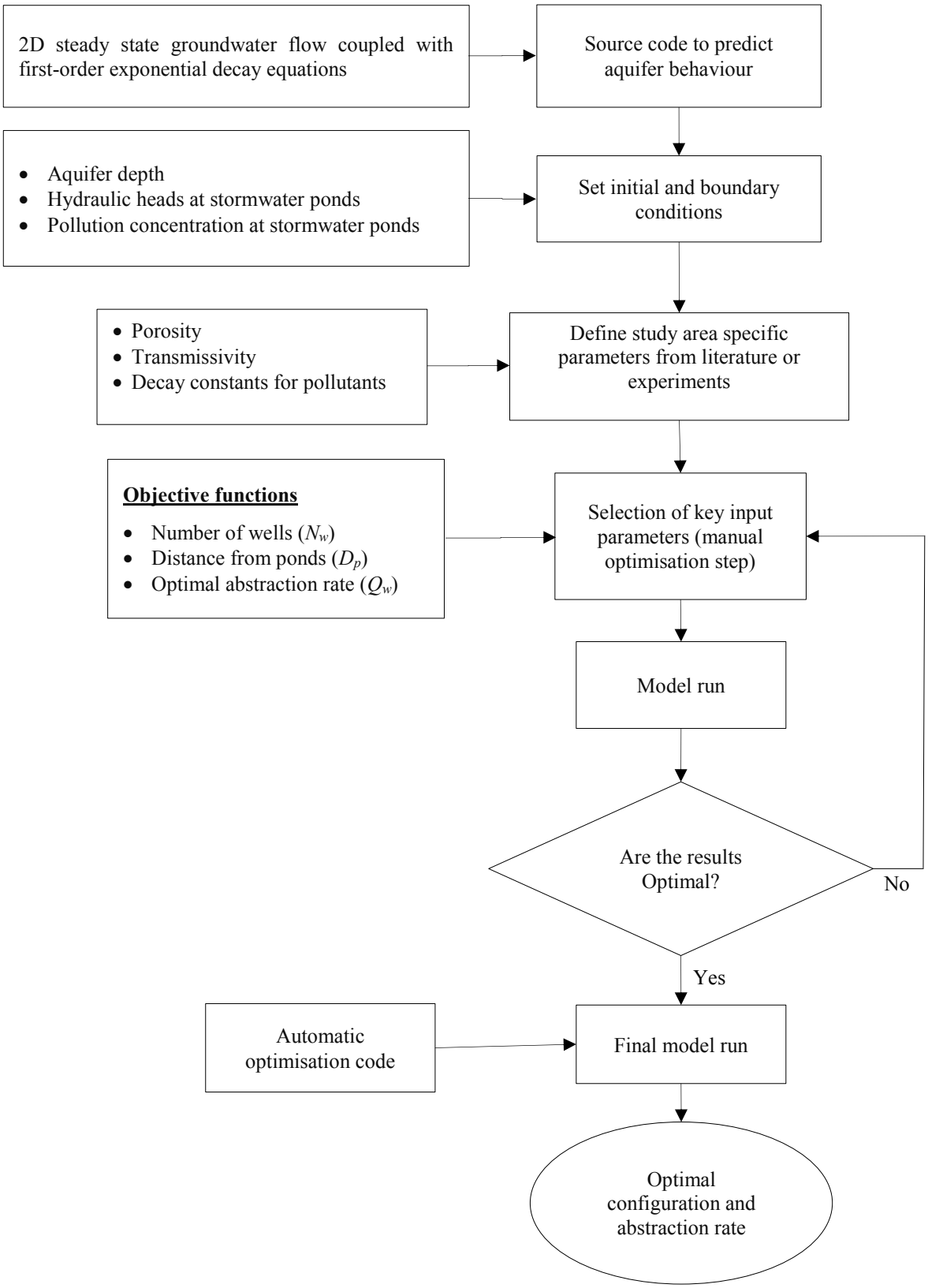


Figure 5-6 Groundwater abstraction model structure

These studies provided information on the geological conditions and aquifer depths in the study area. Other key information included *inter alia*, soil type and seasonal water table variation needed for defining initial and boundary conditions, and geophysical features that could affect groundwater flow, *e.g.* calcrete, clay and peat deposits and layers. Additional data was collected from the study area in a project linked to this research, i.e. Mavundla (2018) as discussed in Chapter 4 (Section 4.2.5). The model development and implementation included discretising the aquifer into a matrix with nodes needed to numerically solve the groundwater flow equations and determining the cell-to-cell water flow.

5.2.2 The equations used for the groundwater flow modelling

The equations used for modelling the groundwater flow implemented in MATLAB are based on a hybrid optimisation approach that combines genetic algorithms (GA) with some local search methods to solve the groundwater flow equations as proposed in Mahinthakumar & Sayeed (2005; 2006) and with direct assistance from the principal author. The goal was to determine the most suitable parameters, *i.e.* an optimum number of wells, pumping rate and distance of wells from the bio-retention cells. According to Mahinthakumar & Sayeed (2005; 2006), the hybrid optimization approach used to model groundwater flow and abstraction of infiltrated stormwater, solves the two-dimensional steady-state partial differential equation with a time step component commonly known as Richard's equation (Richards, 1931) (Equation 5-4).

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{1}{K(x, y)} \frac{\partial h}{\partial t} \qquad \textbf{Equation 5-4}$$

Where v_x and v_y – velocity (flux) in x-direction and y-direction; $K(x, y)$ = hydraulic conductivity in 2-dimensions; dh – hydraulic head; dx and dy – spatial steps in x and y-direction

In the groundwater flow and abstraction model for the study area, Equation 5-4 was represented as a finite difference equation (Equation 5-5) with discrete nodes defined in an indexing system on a matrix layer covering the study area as shown in Figure 5-7.

$$\frac{h_{i+1,j,t} - 2h_{i,j,t} + h_{i-1,j,t}}{(\Delta x)^2} + \frac{h_{i,j+1,t} - 2h_{i,j,t} + h_{i,j-1,t}}{(\Delta y)^2} = \frac{1}{K(x, y)} \frac{h_{i,j,t} - h_{i,j,t-1}}{\Delta t} \qquad \textbf{Equation 5-5}$$

In the modelling of the study area, the discrete nodes were placed in the centre of the square elements of the matrix formed with a series of straight intersecting rows and columns in a well-structured grid (Figure 5-7). To limit the model to the extent of the irregular catchment boundary, four rectangular elements were defined and set to nearly align extent of the study area. The rectangular elements were labelled #1, #2, #3 and #4 (Figure 5-7). Model runs were implemented for each of the rectangular elements to determine the cell-to-cell groundwater.

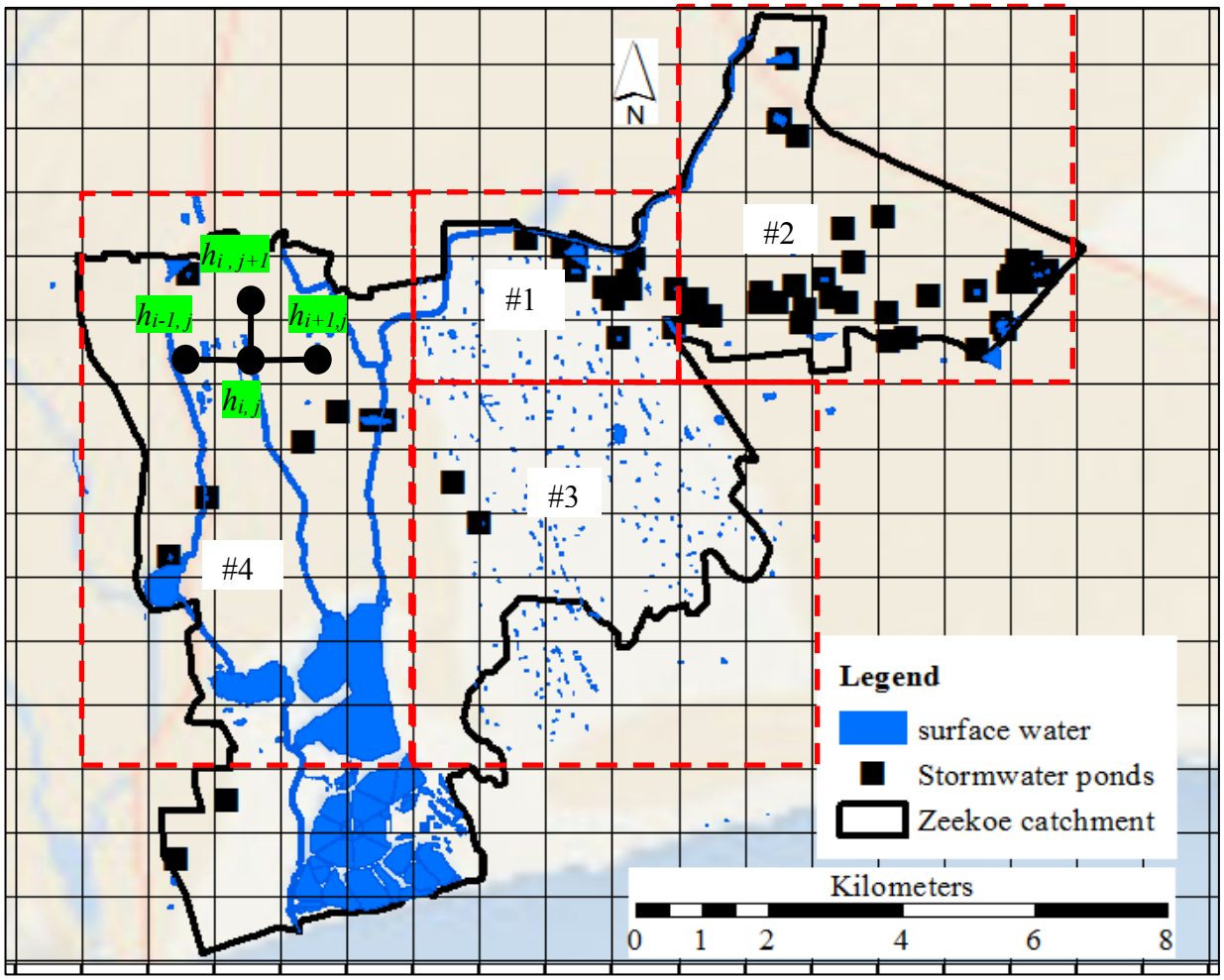


Figure 5-7 A discretised section of Zeekoe Catchment (*After* CCT 2015)

5.2.3 Groundwater abstraction

Groundwater abstraction modelling aimed to determine the appropriate number of abstraction boreholes (N_w), distance of the boreholes from stormwater ponds (D_p) and suitable abstraction rates (Q_w). In the set-up of the abstraction model, the supplementary groundwater resource was assumed to be equal to the stormwater transferred to the aquifer through the infiltration process as discussed in Section 5.1. In the model simulation, the objective was to maximise the quantity of water abstracted based on the estimated stormwater resource transferred to the aquifer through the infiltration process. Since the focus of the study was SWH and in this case, with recharge and recovery from groundwater aquifer, it was necessary to ensure that the abstracted water is from the stormwater ponds. Secondly, it was desirable that the SWH in this manner (*i.e.* recharge and recovery from groundwater aquifer) provided water quality improvement as this would assist the necessary treatment process. This is to be expected since the groundwater aquifer is, in fact, a sand filter. The trade-off between the maximising the quantity of the harvested stormwater (the assumption is that harvesting water from regions closest to the ponds are likely to be from the

stormwater ponds) and the desirable water quality improvement for biological pollutants such as *E.coli* that was determined to be very high in the stormwater. The optimisation aided in the determination of the number of abstraction boreholes, the distance of the boreholes from the associated stormwater ponds and suitable borehole pumping rates in the model set-up and simulation. Ultimately, the main goal was to limit the extraction of water in and around the ponds to avoid excessive drawdown that would destabilise the groundwater balance and potentially causing subsidence. The determination of the optimal values was based on the following criteria:

i. Visual inspection was used as guidance to ensure that the groundwater flow paths originated from stormwater ponds;

ii. The mean groundwater tables are generally deepest at the end of the dry summer (*i.e.* March at about 5 m below ground level). They rise to the surface in many areas with the natural recharge from winter rainfall (Adelana *et al.,* 2006). This seasonal groundwater level fluctuations provided the basis for setting initial conditions and hydraulic heads in the model;

iii. The groundwater abstraction rates were set such that the interference between the various drawdown curves was minimised;

iv. The total abstraction quantity was made approximately equal to the anticipated infiltration with the modified ponds as estimated in PCSWMM; and

v. The retention time of the water in the aquifer was kept at around one year to ensure die-off of the bulk potentially pathogenic organisms as established by Doll (2017), a study associated with this research.

5.3 A trial section of the study area

5.3.1 Groundwater abstraction in the trial section

SWH with recharge and recovery from groundwater aquifer was initially assessed with a trial section of a single stormwater pond (Edith Stephens Wetland shown in Figure 5-8 and 5-9) in a research collaboration with a MSc student from ETH Zurich. The selection of Edith Stephens Wetland was mainly due to the availability of data (both quantity, *i.e.* inflow and outflow; and water quality at various locations in the wetland) compared with other stormwater ponds in the study area. In the model (set-up in MATLAB and MODFLOW), the placement of boreholes to abstract infiltrated stormwater as groundwater was based on the criteria listed in Section 5.2.3. The boreholes were initially placed randomly, with the final positions determined during the modelling process based on visual observation of the origin of the flow field. A separation distance of 400 m from the stormwater pond and abstraction rates of 1.2 L/s per borehole resulted in most of the water flow fields originating from the stormwater pond as shown in Figure 5-10.

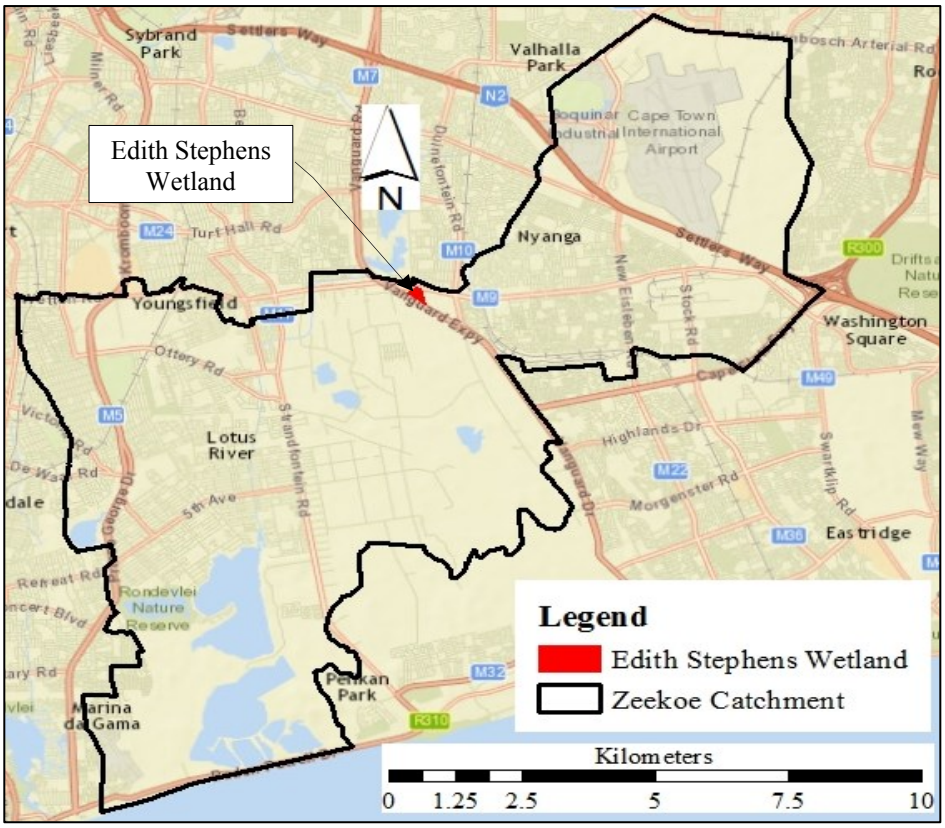


Figure 5-8 Trial section with the Edith Stephens Wetland (*after* CCT 2015)

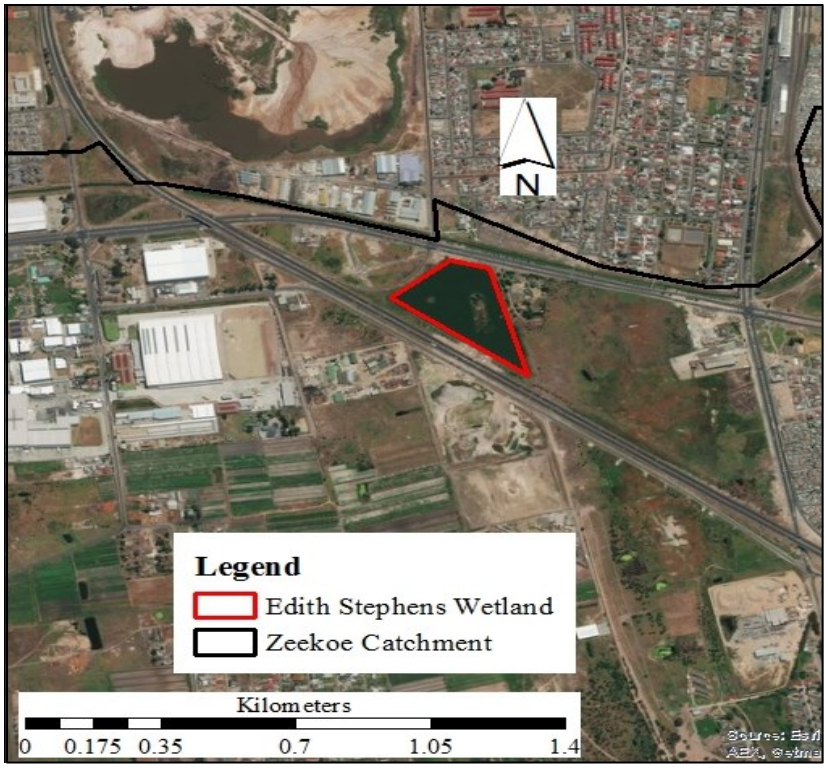


Figure 5-9 Edith Stephens Wetland (*after* CCT 2015)

In the determination of the ideal abstraction rate, the modelling of the rate of withdrawal was increased stepwise with visual inspection of the flow fields and location of the stormwater ponds (Figure 5-10). When the abstraction rates were raised beyond 5.8 L/s per borehole, the origin of groundwater flow was increasingly originating from elsewhere. The maximum abstraction rates were thus determined as 5.8 L/s per borehole.

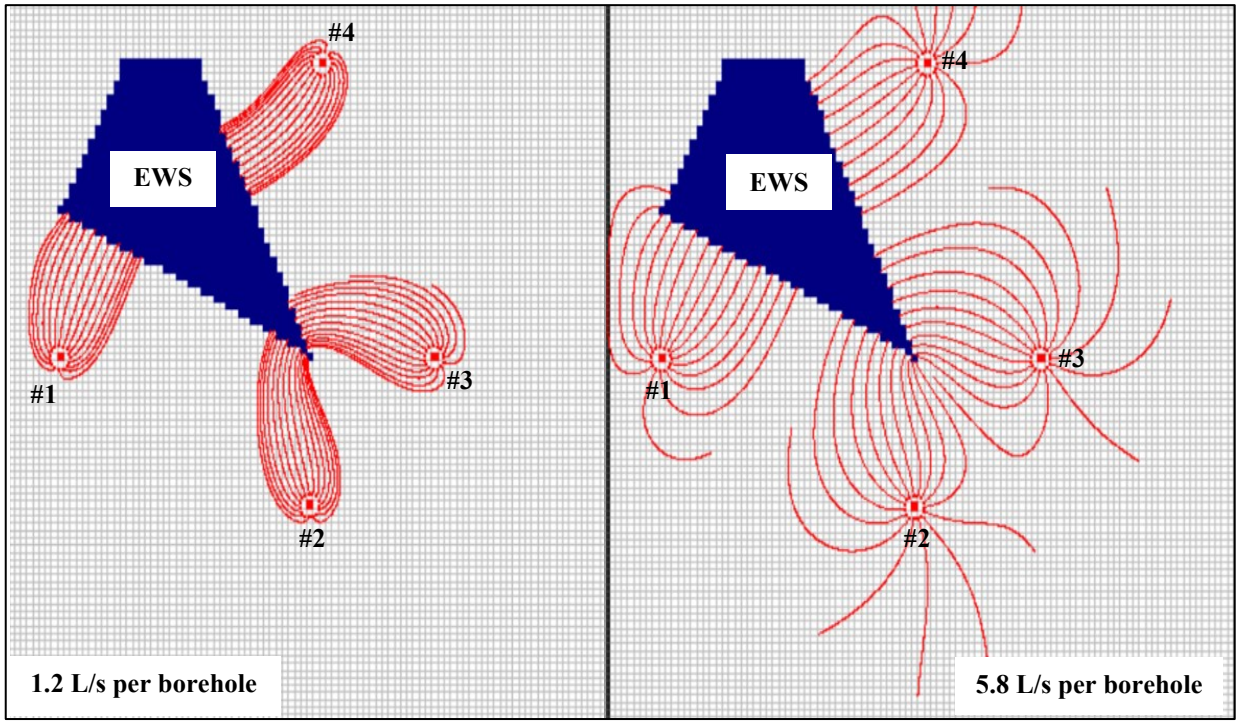


Figure 5-10 Impact of abstraction rates on the origin of the groundwater flow fields at the Edith Stephens Wetland (EWS) (Doll, 2017)

5.3.2 Water quality assessment in the trial section

SWH using aquifer storage typically provides an opportunity for water quality improvement through infiltration, adsorption (the process whereby pollutants bind to the surface of fine sand particles), biodegradation and volatilisation (the conversion of some compounds to gases or vapour). To determine the potential for water quality improvement, the pollution decay associated with groundwater transport from the stormwater pond to abstraction boreholes was assessed in the model. Various sources (*e.g.* Zimmerman *et al.,* 2016; Schulze *et al.*, 2005) provide pollution decay equations and rates. In the model, only *E.coli* (an indicator organism for faecal pollution) was considered (Delleur, 2007). A commonly used pollution decay is the first order relationship presented in Equation 5-6 (Delleur, 2007).

$$C_t = C_o\, e^{-\lambda t} \qquad \textbf{Equation 5-6}$$

Where C_t – concentration or quantity at time t; C_o – initial quantity at the start of assessment (t=0); λ – pollution decay rate (day^{-1}). The units of C_t and C_o depend on the pollution.

Equation 5-6 is a simplification of the process; *E.coli* removal typically depends on various factors including *inter alia* the availability of nutrients in the water, the exposure to UV-Radiation and temperature (Delleur, 2007). Nevertheless, simplification was adequate for the study as the goal was to provide indicative water quality improvement opportunity from the process of stormwater recharge and recovery. The primary parameter required in the model was a decay rate (Delleur, 2007). Potential decay rates for this study are listed in Table 5-1.

Table 5-1 Various *E.coli* decay rates

Description of conditions	Decay rate	Source
Laboratory condition, light exposure, seawater	14.7 - 107	Chan *et al.,* 2015
Laboratory condition, darkness, seawater	0.85 - 1.5	Chan *et al.,* 2015
Literature study	0.025 - 0.051	Engelbrecht, 1998
Laboratory, groundwater	0.046 - 0.092	Filip *et al.,* 1988
In situ diffusion chamber, groundwater	0.42	Page *et al.,* 2010
In situ diffusion chamber, groundwater	0.691	Sidhu *et al.*, 2012
Field experiment, groundwater	0.15	Toze *et al.,* 2002

The slowest decay rate of 0.051 (Engelbrecht, 2006) was selected as a conservative value associated with slow organism inactivation and prolonged survival times. The conservative value would provide for the worst case conditions. Simulations were undertaken in the model with Equation 5-6 to determine transport and decay of the *E.coli* as an indicator organism using an abstraction rate of 5.8 L/s per borehole and distance of 400 m. The values used in the simulation of pollution transport was based on the monthly grab samples collected by CCT from various locations in Edith Stephens Wetland. The data collected at inlet and outlet from 2006 – 2017 are presented in Figure 5-11. The very high *E.coli* values are consistent with major pollution sources such as on-site sanitation upstream of the Edith Stephens Wetland and direct discharge of grey and black water into the drainage channel from informal settlements. Since the grab samples are not collected at regular intervals (*i.e.* the sample collection date in the month was not consistent, and some values were missing), the data was only used to provide an indication of the river health and values for modelling purposes. Based on the data in Figure 5-11, the model was run with conservative value of 3,400,000 CFU/100ml (the maximum value). The results of the modelling of water quality improvement are presented in Figure 5-12.

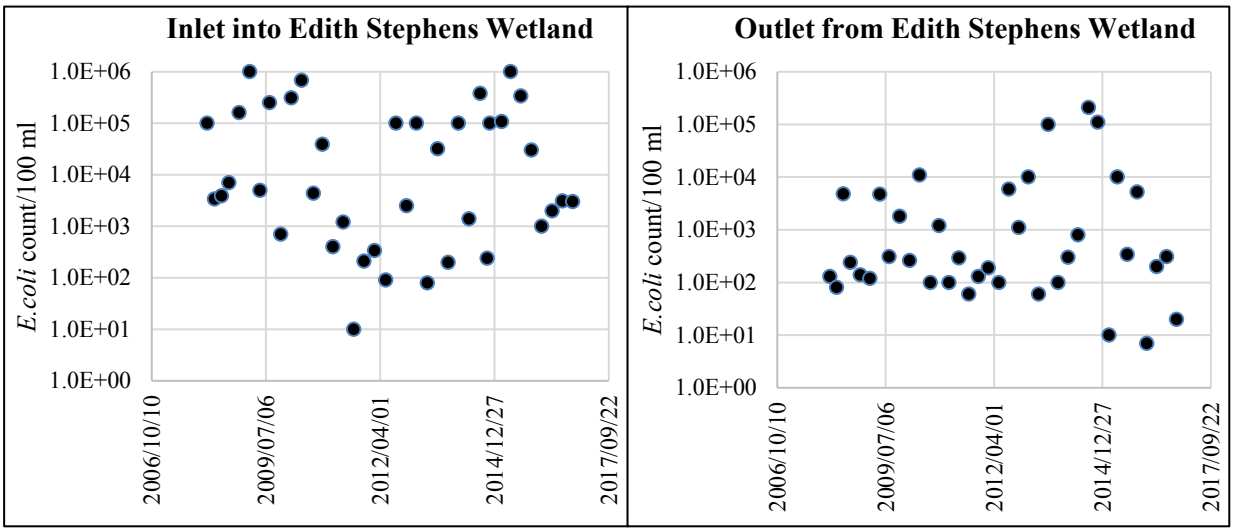


Figure 5-11 *E.coli* measured at Edith Stephens Wetland (CCT, 2017)

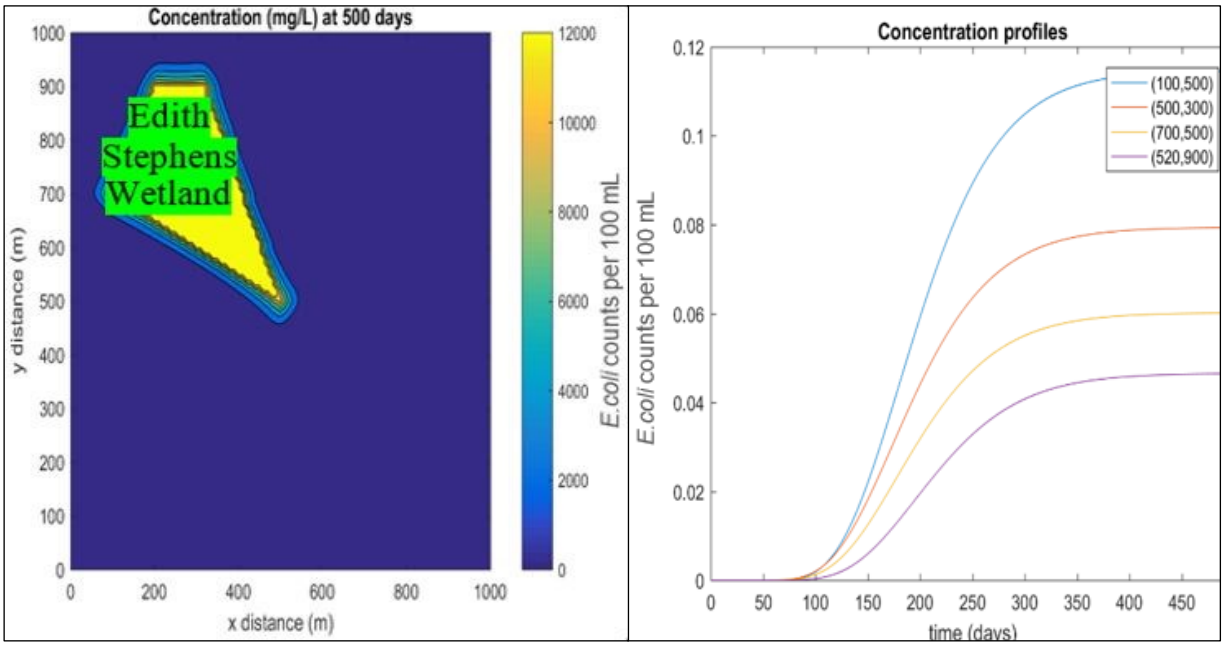


Figure 5-12 *E.coli* counts in the Edith Stephen Wetland (left) and at the boreholes abstracting at 5.8 L/s (right) with pumping commencing at Day 0 (Doll, 2017)

The results from the trial section with a single pond (Edith Stephens Wetland) and *E.coli* as an indicator organism for faecal pollution, show that that the sandy aquifer in the study area has the potential to remove very high levels of *E.coli,* from 1 x 10^3 to 1 x 10^5 counts per 100 millilitres (Figure 5-11) to values below one count per 100 millilitres (Figure 5-11). With the South African National Drinking Water Standards (SANS 241:2015) requiring zero count *E.coli*, this indicates that stormwater harvested from groundwater storage would theoretically be suitable for potable water uses with minimal additional disinfection treatment.

5.4 Catchment-scale

5.4.1 Groundwater abstraction model

The findings from the trial section model (Section 5.3) were extended to a catchment scale model. Additional information on typical borehole yield rates for the study area were obtained from CCT (Figure 5-13) and various studies on the Cape Flats Aquifer (CFA) such as Vandoolaeghe, (1989); Fraser *et al*., (2001); DWA, (2008); and Mauck, (2017).

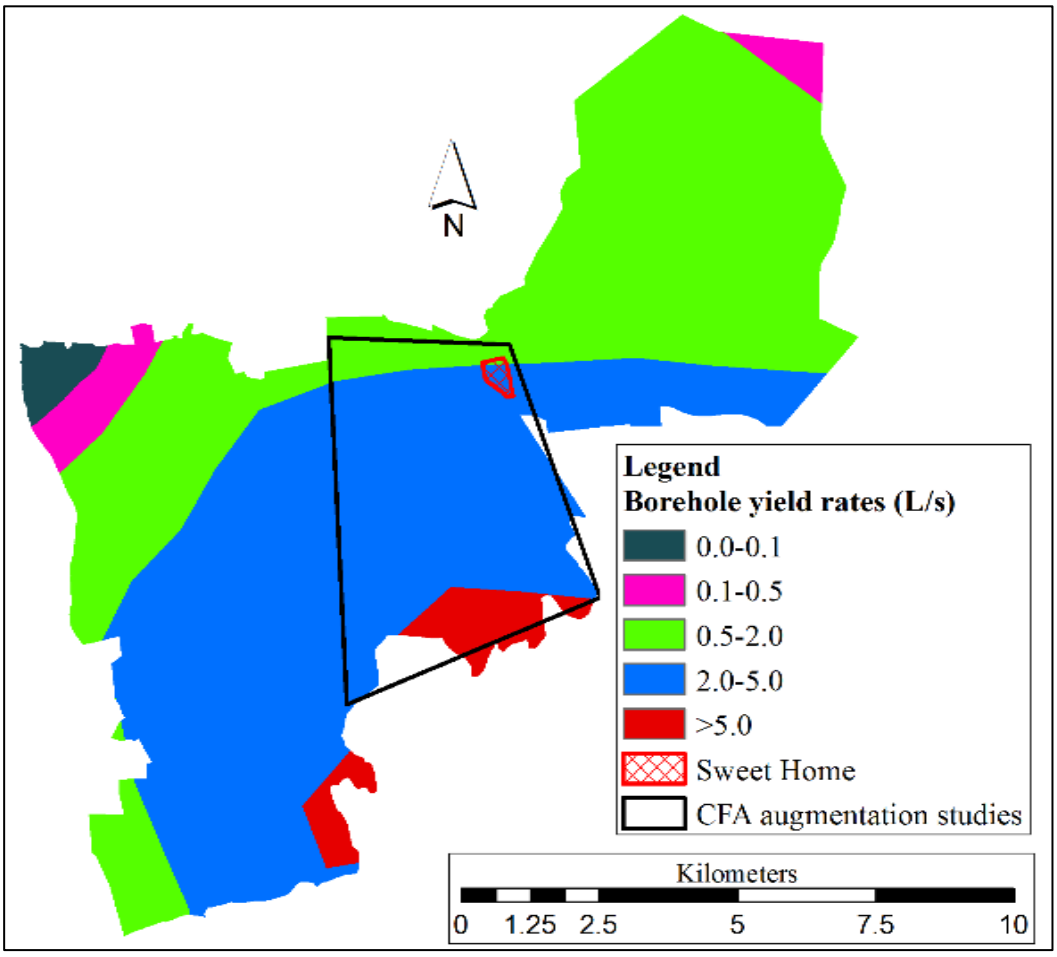


Figure 5-13 Borehole yield rates in the study area (*after* CCT, 2005)

Vandoolaeghe (1989) determined that a total of 10 Mm^3 / annum could be abstracted with 27 boreholes each pumping at an abstraction rate of 12 L/s from the CFA augmentation study area shown in **Figure 5-13**. Another study (*i.e.* Fraser *et al*., 2001) suggested a total groundwater yield of 18 Mm^3 /annum for the same area with additional boreholes. Abstraction rates of 6 L/s per borehole were determined to be most suitable with higher values potentially extending the groundwater cone of depression to the coastline and resulting in possible seawater intrusion (DWA, 2008). In a more recent study at Sweet Home (also shown in Figure 5-13), six scenarios

were assessed consisting of three arrangements of 9, 18 and 27 boreholes with abstraction rates between 3 L/s and 5 L/s (Mauck, 2017). One of the key aims of the study was flood mitigation through the drawdown of the water table to values lower than a threshold of 1.5 m below the surface through groundwater abstraction (Mauck, 2017). The study determined that an abstraction rate of 3 L/s would not draw down the water table to below the 1.5 m threshold for flood mitigation in all the three borehole arrangements. With the borehole pumping rates increased to 5 L/s, the simulated groundwater drawdown exceeded the 1.5 m threshold only 5% of the time for the 18 boreholes and completely for the 27 boreholes (Mauck, 2017). The values from the trial section (Section 5.3), CCT and various reference (*i.e.* Vandoolaeghe, 1989; Fraser *et al*., 2001; DWA, 2008; and Mauck, 2017) were interpolated in ArcGIS across the study area to generate the borehole yields as shown in **Figure 5-13**. In the catchment-scale model, the borehole yields in **Figure 5-13** were simulated for each of the four rectangular elements with 'red dash lines' labelled #1, #2, #3 and #4 in Figure 5-7. A plan showing the location of the ponds and placement of the abstraction boreholes for Computational Area #1 are given in Figure 5-14 and 5-15 respectively.

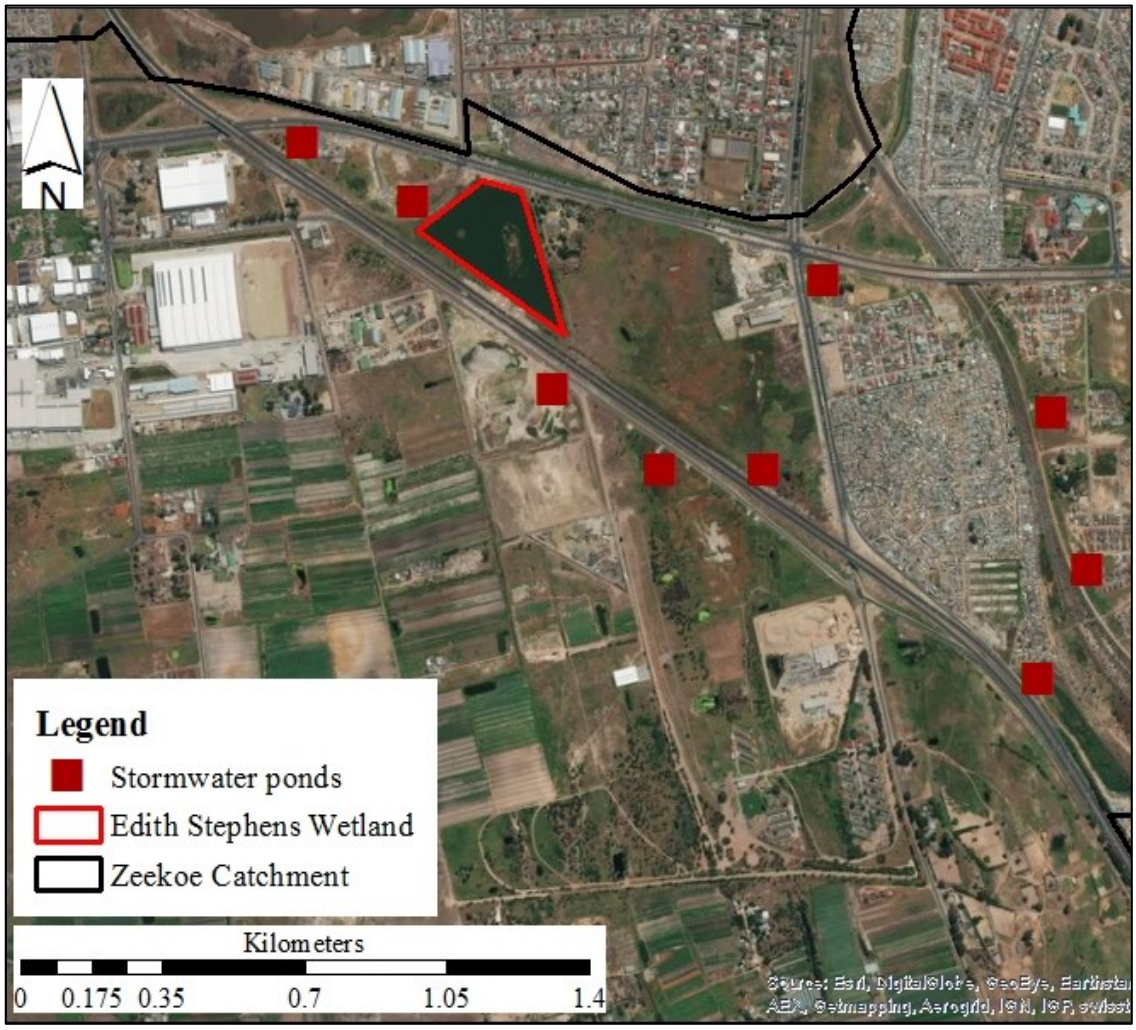


Figure 5-14 Location of the stormwater ponds in Area #1

The modelling of the groundwater abstraction in this study was implemented in two main steps, *i.e*. a manual trial and an optimisation process. The manual trial consisted of initially placing four boreholes per pond in each of the Computational Areas #1, #2, #3 and #4 (Figure 5-7). The boreholes were initially placed randomly around the stormwater ponds and each simulated with an abstraction rate of 5.8 L/s as determined in the trial section (Section 5.3). The number of boreholes and abstraction rates were then adjusted as discussed in Section 5.3 until the flow fields started to come from the stormwater ponds. An optimisation procedure was then implemented in MATLAB (MATLAB, 2010) with a genetic algorithm as proposed in Mahinthakumar & Sayeed, (2006) and discussed in Section 5.2.2 to provide the final borehole positions and abstraction rates. The modelled phreatic flow field in Computational Area #1 showing the flow paths from the ponds to the proposed abstraction boreholes is presented in Figure 5-15.

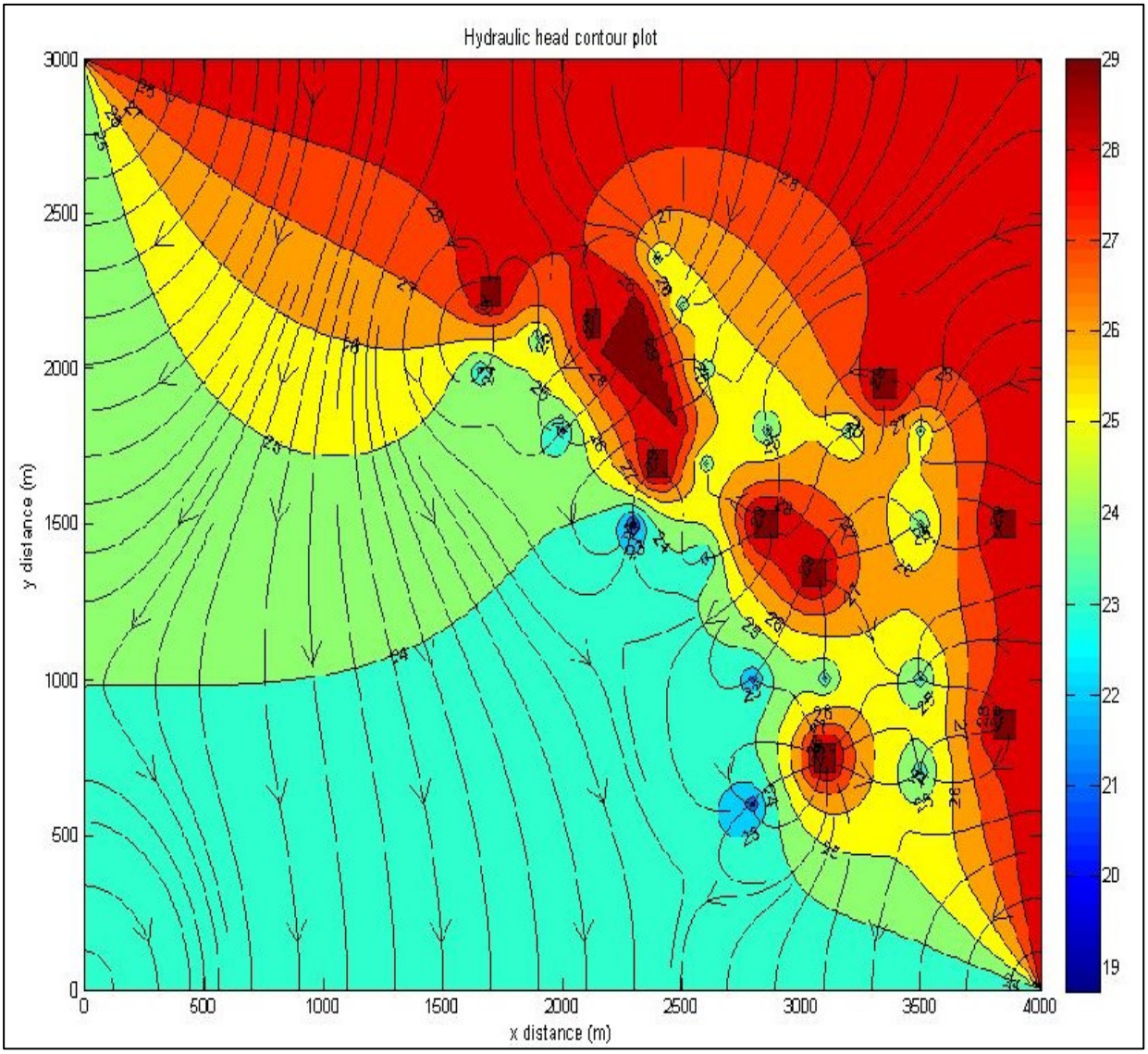


Figure 5-15 Modelled phreatic flow fields from the stormwater ponds to the boreholes

The MATLab code and modelled phreatic flow fields for Computational Area #2, #3 and #4 have been included in the appendices. A summary of the results including the parameters used, the final optimised modelled number of boreholes, abstraction rates per borehole and mean annual groundwater yields for each Computational Area are presented in Table 5-2.

Table 5-2 Range of model domain parameters and potential groundwater yield

	Computational Area			
Parameters	**#1**	**#2**	**#3**	**#4**
Domain size (km^2)	12	36	45	36
Conductivity (cm/hr)	4.8	11.1	11.1	10.3
Porosity (%)	33	37	37	40
Aquifer depth (m)	30	20	40	50
Number of boreholes	20	20	40	60
Distance of well from ponds(m)	400	400	400	400
Mean abstraction rate per borehole (m^3/day)	300	500	500	700
Mean abstraction rate per borehole (L/s)	3.5	5.8	5.8	8.1
Potential total annual groundwater yield (Mm^3/year)	2	4	7	15

The results in Table 5-2 show that depending on aquifer parameters in each Area in Figure 5-4, *i.e.* domain size, conductivity, porosity and aquifer depth, the abstraction rates per borehole ranged from 3.5 – 8.1 L/s to ensure that the flow fields were drawn largely from the areas around the stormwater ponds. When the abstraction rates increased beyond these values, the groundwater flow fields started drawing from outside the pond region.

5.4.2 Water quality assessment with the catchment-scale model

5.4.2.1 Overview

An assessment was undertaken to determine the likely water quality improvement associated with stormwater recharge and recovery. The CCT collects grab samples from various locations in the study area, as shown in **Figure 5-16** to test for various water quality parameters including *inter alia E.coli*, Total Suspended Solids TSS), pH, Electrical Conductivity (EC), Nutrients (*i.e.* Total Persulphate Oxidisable Nitrogen (TPON), Nitrate and Nitrite, Total Phosphorus) and Algae (*i.e.* Chlorophyll-a and Phaeophytin). Since the timing of the sampling and testing of the water quality parameters in the study area was irregular (*i.e.* the sample collection date in the month was not consistent, and some values were missing), the data could only be used to provide a rough indication of values for modelling purposes. The assessment was mainly undertaken with pathogen indicator organisms *i.e. E.coli* as they were determined to be very high and exceeding even the intermediate contact guideline of 1000 counts/100ml (Haskins, 2014). Nutrient

concentrations were largely below 10 mg/L with levels mostly around a mean value of 1 mg/L. With the relatively low values, no modelling was undertaken to determine water quality improvement with respect to nutrients.

5.4.2.2 Pathogens

The theoretical assessment of stormwater quality improvement with *E.coli* as the indicator organism for pathogens was modelled for the Areas #1, #2, #3 and #4 in Figure 5-4 based on the data collected by CCT at locations shown in Figure 5-16 with transport and decay Equation 5-6.

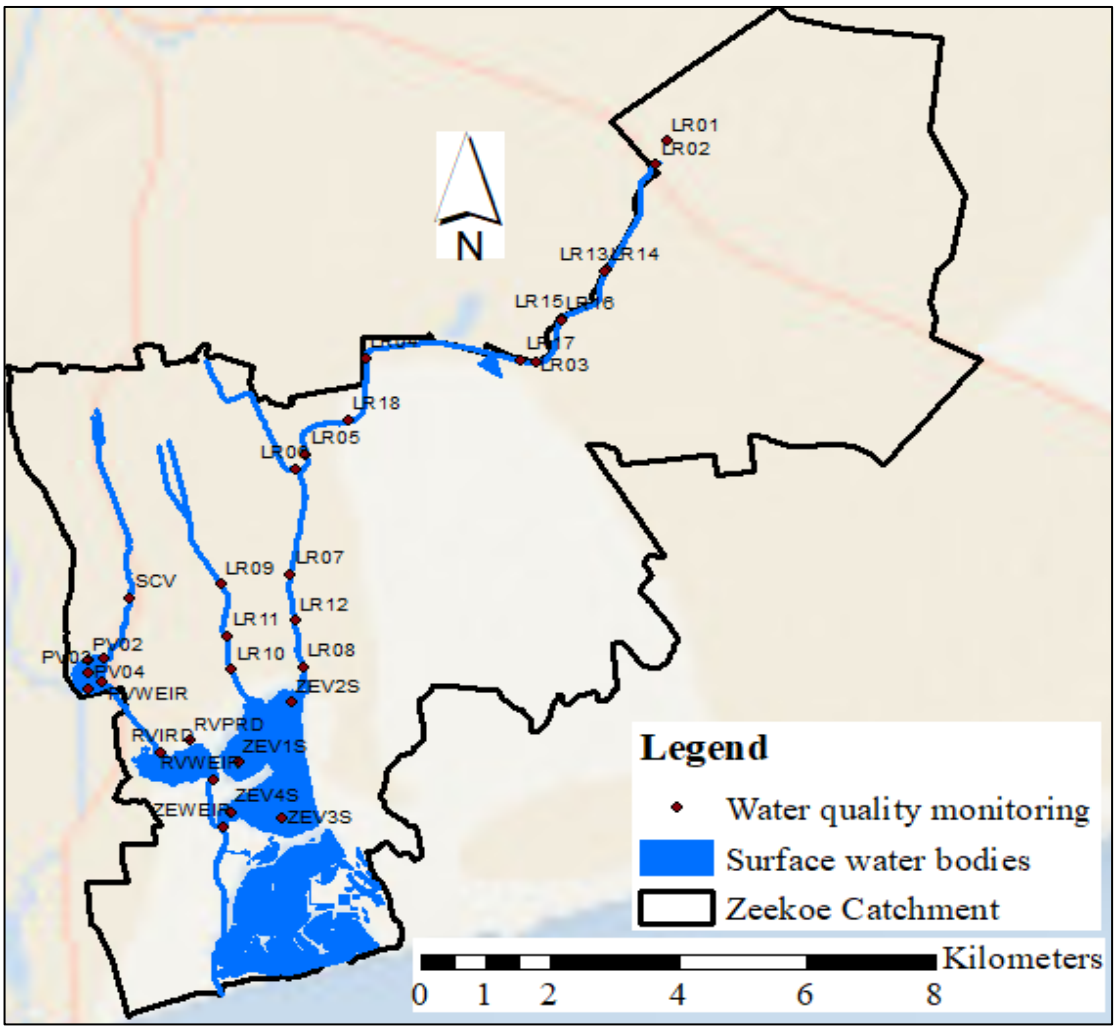


Figure 5-16 Locations of water quality motoring in the study area (*after* CCT, 2017)

For Area #1 in Figure 5-14 and phreatic flow shown in Figure 5-15, the modelling was based on values at the inlet of Edith Stephens Wetland (Figure 5-11) and Equation 5-6. The modelling was based on the procedure discussed in Section 5.3.2. The results from the water quality modelling

based on final positions of boreholes (Figure 5-15) are given as curves (Figure 5-17) indicating *E.coli* counts with respect to time of flow to reach the abstraction boreholes. The values from the catchment-scale model with 10 stormwater ponds and 20 abstraction boreholes as shown in Figure 5-15 indicate that that the sandy aquifer in the study area has the potential to remove very high levels of *E.coli, i.e.* 1 x 10^6 counts per 100 mL to values below 8 counts per 100 mL.

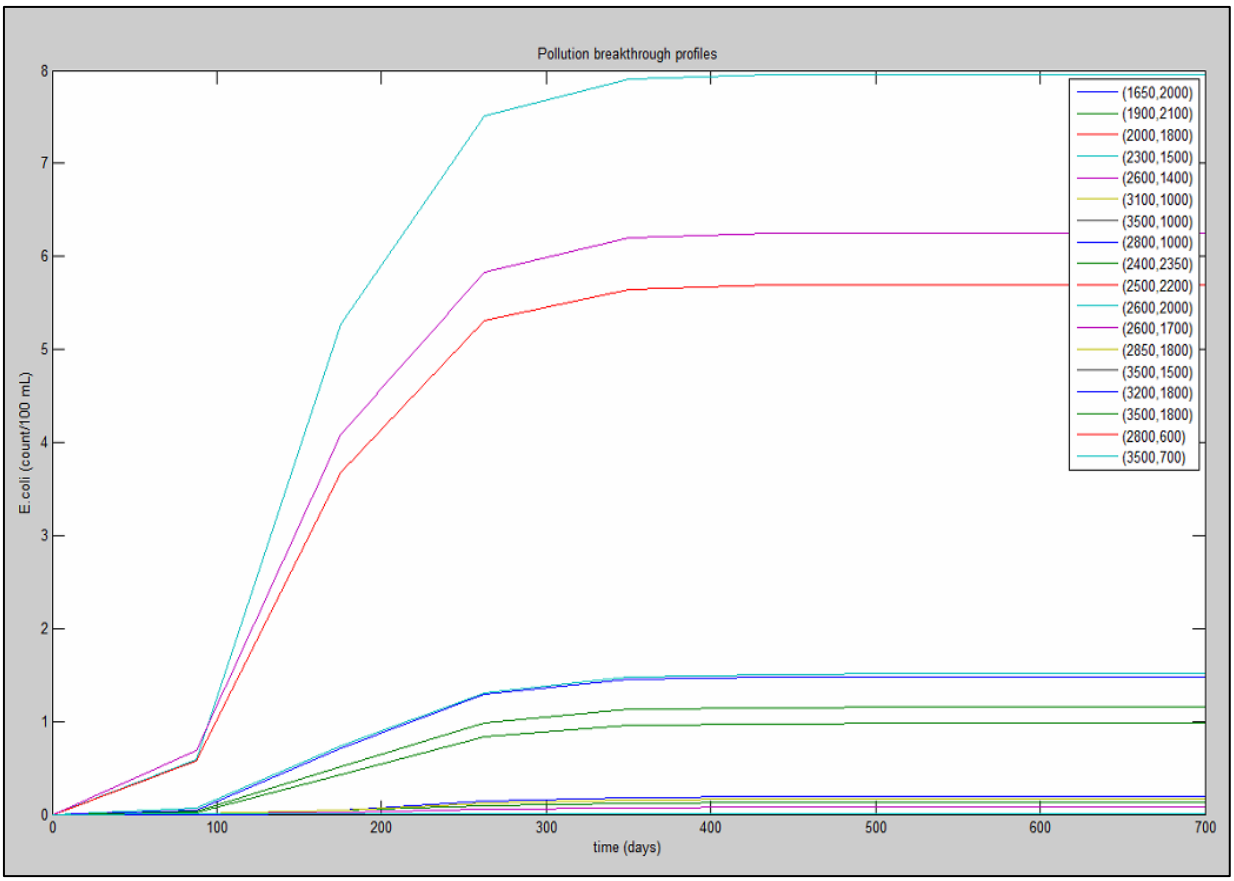


Figure 5-17 Estimated *e.coli* counts in the abstraction boreholes following commencement of pumping

In Areas #1 and #2, *E.coli* counts were detected at boreholes after about 100 days, and the rate of accumulation increased rapidly over a 200 day period. It stabilised in the range of one to eight counts per 100 millilitres at about 350 days (about one year) as shown in Figure 5-17. In Areas #3 and #4, *E.coli* counts were detected at boreholes after about 200 days, and rate of accumulation increased rapidly over a 400 day period and stabilised at about 600 days (1.5-year mark).

5.4.2.3 Heavy metals

Samples to test the presence of heavy metals in stormwater were collected at various locations in the study area as shown in **Figure 5-18** in a research collaboration with Nesre Redi – a master's student at the Swiss Tropical and Public Health Institute – Switzerland.

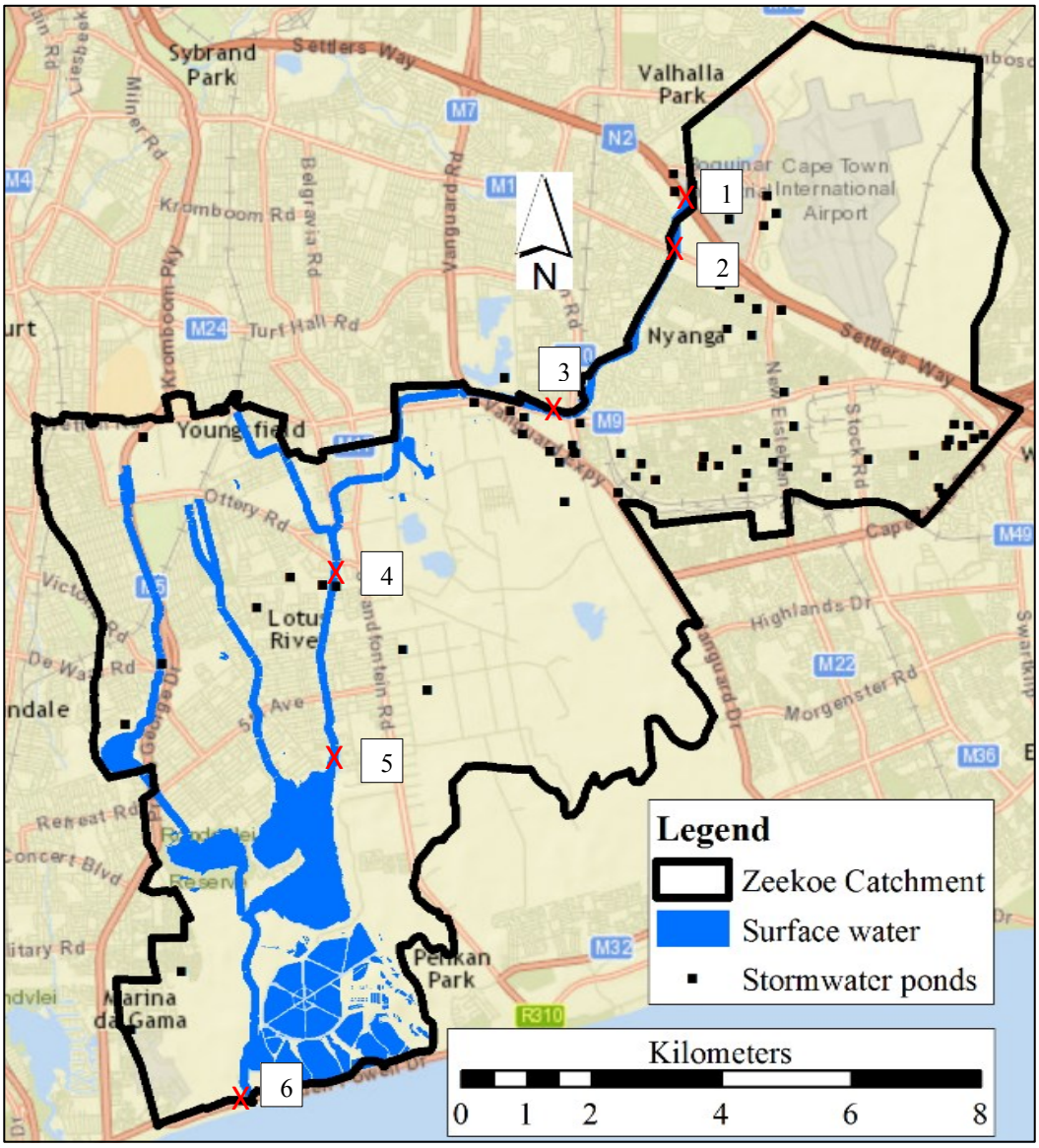


Figure 5-18 Locations of heavy metal sampling

The data on heavy metals collected in the study included Arsenic (Ar), Cadmium (Cd), Chromium (Cr), Lead (Pb) and Mercury (Hg). These have been linked to chronic diseases such as cancer (WHO, 2008; USEPA, 2016). A total of 35 samples per heavy metal were collected over a one-week period from 20th – 24th June 2016 as shown in Table 5-3.

Table 5-3 Heavy metal in the stormwater drainage of the study area

Heavy Metals	Sample Date	Locations where samples were collected for testing heavy metal concentration					
		1	2	3	4	5	6
Arsenic (µg/L as As)	20/6	<3	<3	<3	<3	<3	<3
	21/6	<3	<3	-	<3	<3	22
	22/6	<3	8	<3	<3	<3	<3
	23/6	<3	<3	<3	<3	<3	<3
	24/6	<3	<3	<3	16	7	13
Cadmium (µg/L as Cd)	20/6	<1	<1	<1	<1	<1	<1
	21/6	<1	<1	-	<1	<1	<1
	22/6	<1	<1	<1	<1	<1	<1
	23/6	<1	<1	<1	<1	<1	<1
	24/6	<1	<1	<1	<1	<1	<1
Chromium (µg/L as Cr)	20/6	<7	9	10	<7	<7	<7
	21/6	<7	20	-	<7	13	12
	22/6	<7	<7	<7	<7	<7	<7
	23/6	<7	<7	<7	<7	<7	<7
	24/6	<7	<7	<7	<7	<7	<7
Lead (µg/L as Pb)	20/6	<7	<7	<7	<7	<7	<7
	21/6	<7	<7	-	<7	<7	<7
	22/6	<7	<7	<7	<7	<7	<7
	23/6	<7	<7	<7	<7	<7	<7
	24/6	<7	<7	<7	<7	<7	<7
Mercury (µg/L as Hg)	20/6	<5	<5	<5	<5	<5	<5
	21/6	<5	<5	-	<5	<5	12
	22/6	<5	<5	<5	<5	<5	22
	23/6	<5	18	<5	14	<5	<5
	24/6	<5	<5	<5	<5	<5	<5

< - heavy metal concentration below indicated values

From the collected data, it seemed that the concentration of heavy metals was generally low. However, a significant presence of heavy metals was detected on some days, an indication of specific point source pollution. In a study by Davis *et al.*, (2003), it was shown that continuous loading of even low concentrations of heavy metals over an extended period, *e.g.* 20 years could result in concentrations exceeding levels permitted for human use. A strategy for sustainable management of the environment and soils in South Africa has been provided in the Government

Gazette for the *Protection and Remediation of Contaminated Soils* (Department of Environmental Affairs, 2012). In this strategy, the limits of soil contamination were provided as shown in Table 5-4 (Department of Environmental Affairs, 2012)

Table 5-4 Limits of soil contamination (Department of Environmental Affairs, 2012)

	Arsenic	Cadmium	Chromium	Lead	Mercury
Land uses protective of water (mg/kg)	5.8	7.5	6.5	20	0.93
Informal residential (mg/kg)	23	15	6.5	110	0.93
Standard residential (mg/kg)	48	32	13	230	1.0
Commercial/industrial (mg/kg)	150	260	40	1900	6.5
Protection of ecosystem (mg/kg)	580	37	260	100	4.1

The estimation of heavy metal accumulation in areas around the stormwater ponds was based on Equation 5-7 that has been used in other studies such as Marsalek *et al.*, (2001); Davis *et al.*, (2003); Weiss *et al.*, (2008).

$$\frac{C_s}{t} = C_w \frac{A_r}{A_i} \frac{MAR}{d.\rho} \qquad \textbf{Equation 5-7}$$

Where C_s/t – annual accumulation rate of metal mass/soil mass (mg/kg per year); C_w – concentration of metal in the runoff water (mg/m^3); A_r/A_i – the ratio of runoff area catchment to infiltration area; MAR – the mean annual rainfall (mm); d – the thickness of the soil layer (mm); ρ – the soil bulk density (kg/m^3); and t – time (years).

In the determination of the accumulation of the heavy metals in the study area and the period before concentrations would possibly exceed levels permitted for human use, it was assumed that heavy metals were retained in the top 150 mm soil layer (Weiss *et al.*, 2008). The bulk density of the soil was determined by collecting samples from the study area and analysed in the laboratory and found to have a mean of 1541 kg/m^3 with minimum and maximum values of 1467 and 1616 kg/m^3 respectively. The annual rate of accumulation as metal mass/soil mass (mg/kg per year) was then estimated from Equation 5-7 and the results presented in Tables 5-5 to 5-8. The time in years for the metal concentration to exceed the limits of soil contamination was computed from the calculated heavy metal annual accumulation rate (mg/kg per year).

Table 5-5 Period for accumulated metals to exceed limits – Area #1

	Arsenic	Cadmium	Chromium	Lead	Mercury
Mean annual runoff volume (mm/year)	335	335	335	335	335
Ar/Ai	72:1	72:1	72:1	72:1	72:1
Concentration (µg/L)	3	1	7	7	8
Annual accumulation rate (mg/kg per year)	0.31	0.11	0.72	0.72	0.83
All land uses protective of water resources (years)	19	73	9	28	1
Informal residential (years)	74	145	9	152	1
Standard residential (years)	155	310	18	318	1
Commercial/industrial (years)	484	2519	55	2629	8
Protection of ecosystem health (years)	1873	358	360	138	5

Table 5-6 Period for accumulated metals to exceed limits – Area #2

	Arsenic	Cadmium	Chromium	Lead	Mercury
Mean annual runoff volume (mm/year)	440	440	440	440	440
Ar/Ai	41:1	41:1	41:1	41:1	41:1
Concentration (µg/L)	5	1	11	7	9
Annual accumulation rate (mg/kg per year)	0.36	0.08	0.88	0.54	0.72
All land uses protective of water resources (years)	16	97	7	37	1
Informal residential (years)	64	193	7	203	1
Standard residential (years)	133	413	15	424	1
Commercial/industrial (years)	415	3353	46	3501	9
Protection of ecosystem health (years)	1,603	477	297	184	6

Table 5-7 Period for accumulated metals to exceed limits – Area #3

	Arsenic	Cadmium	Chromium	Lead	Mercury
Annual runoff volume (mm/year)	175	175	175	175	175
Ar/Ai	140:1	140:1	140:1	140:1	140:1
Concentration (µg/L)	3	1	7	7	5
Annual accumulation rate (mg/kg per year)	0.32	0.11	0.74	0.74	0.53
All land uses protective of water resources (years)	18	71	9	27	2
Informal residential (years)	73	143	9	150	2
Standard residential (years)	152	305	18	313	2
Commercial/industrial (years)	476	2476	54	2585	12
Protection of ecosystem health (years)	1,841	352	354	136	8

Table 5-8 Period for accumulated metals exceed standard – Area #4

	Arsenic	Cadmium	Chromium	Lead	Mercury
Annual runoff volume (mm/yr)	480	480	480	480	480
Ar/Ai	101:1	101:1	101:1	101:1	101:1
Concentration (µg/L)	3	1	9	7	5
Annual accumulation rate (mg/kg per year)	0.62	0.21	1.86	1.45	1.04
All land uses protective of water resources (years)	9	36	3	14	1
Informal residential (years)	37	72	3	76	1
Standard residential (years)	77	154	7	159	1
Commercial/industrial (years)	241	1255	21	1310	6
Protection of ecosystem health (years)	933	179	139	69	4

A_r/A_i – the ratio of runoff catchment area to the infiltration area

5.4.3 Summary of results for stormwater harvesting from groundwater storage

Potential for SWH utilising aquifer storage has been discussed in this chapter. In the assessment, it was determined that the physical characteristics in the Zeekoe Catchment, *i.e.* flat terrain, pervious sandy soils and unconfined aquifer would support abstraction rates of 3.5 – 8.1 L/s from 140 boreholes to provide a mean annual groundwater yield of 28.51 – 32.61 Mm^3. With the South African National Drinking Water Standards (SANS 241:2015) providing a zero count of *E.coli* per 100 mL, the findings from the Catchment-scale model show that the stormwater harvested from groundwater storage would theoretically be adequate for potable water uses with minimal additional disinfection treatment. Other studies such as Lim *et al.* (2015) have also shown that microbial pollution in stormwater from groundwater storage was significantly reduced to levels where the water could directly be used for some indoor residential needs with a limited level of contact, *e.g.* washing machine and toilet flushing. Another study, *i.e.* Vanderalm *et al.*, (2010) showed that stormwater recovered from an aquifer after a mean residence time of 240 days was suitable for non-potable water applications. However, continuous monitoring and provision for post-recovery disinfection for pathogen and aeration for iron removal are recommended where necessary. Overall, SWH from groundwater storage provides larger water quantities at a better water quality than the surface storage option.

6. Potential demand for stormwater

In this Chapter, the objective was to identify and estimate the potable and various non-potable water demands in the catchment for purposes of determining the reliability and adequacy to be supplied with stormwater. The estimation method used and results are provided in nine sections: Sections 6.1 to 6.5 cover the methods used to identify and quantify the non-potable water demands in the Zeekoe Catchment *i.e.* urban agriculture; public open spaces; residential gardens; and flushing toilets. Section 6.6 describes the method used to determine optimal storage requirement and assess the volumetric reliability of stormwater supply to meet non-potable demand. Sections 6.7 and 6.8 cover the stormwater quality and proposed water treatment for potable and non-potable and Section 6.9 is a summary of Chapter 6.

6.1 Urban agriculture

Urban agriculture is a significant land use in the study area with about 30% areal coverage (*i.e.* 25.6 km^2 of the 88.8 km^2 catchment area) as shown in Figure 6-1.

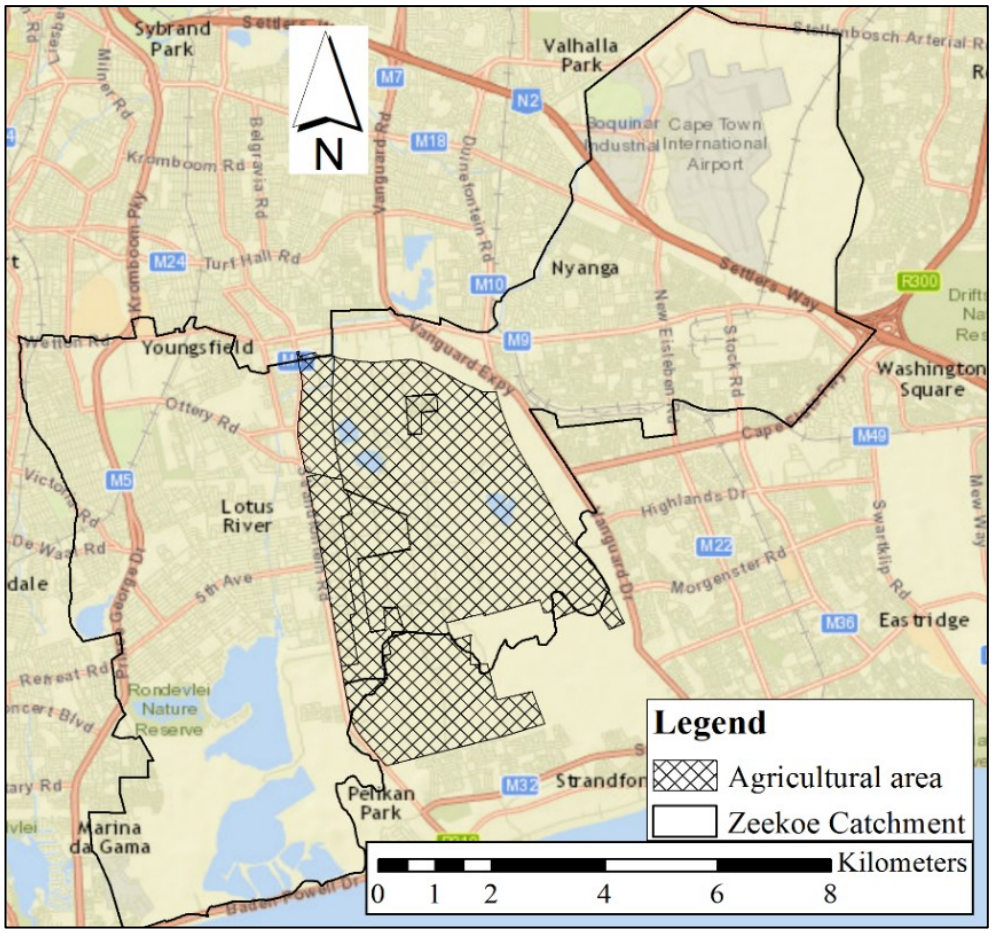


Figure 6-1 Urban agriculture in the study area (*after* CCT, 2016)

The Agriculture irrigation water demand may be estimated from the Crop Water Requirement (CWR) needed to meet precipitation deficit (FAO, 2012). The Food Agricultural Organisation (FAO) has developed the CROPWAT model to assist in the estimation of CWR based on climatic and soil data compiled by the FAO Agrometeorological Group for over 100 countries (Smith, 1992). CROPWAT 8.0 was suitable for the study as it can estimate CWR and generate irrigation schedules with multiple crops. In CROPWAT Version 8.0, the CWR was calculated with Equation 6-1 (FAO, 2012).

$$CWR = (k_c \, ET_o \, d - R_{eff}) \qquad \textbf{Equation 6-1}$$

Where *CWR* – Crop Water Requirement (mm); k_c – crop coefficient; *ETo* – Evapotranspiration (mm day^{-1}); *d* – days in a month (days); R_{eff} – effective rainfall (mm)

The k_c values are available in the CROPWAT model for various crops and periods, *i.e.* initial, middle and end of the plant growth stages. The American Society of Civil Engineers (Jensen *et al.,* 1990) and European Community (Choisnel *et al.,* 1992) evaluated various ET estimation procedures under different climatic conditions and confirmed that the Penman-Monteith method was most suitable in both arid and humid climates. The FAO then developed the modified FAO Penman-Monteith method (Equation 6-2), now widely used in the design of irrigation systems (Allen, 2000).

$$ET_o = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma (1 + 0.34 u_2)} \qquad \textbf{Equation 6-2}$$

Where *ETo* – evapotranspiration (mm day^{-1}), R_n – net radiation at crop surface (MJ m^{-2} day^{-1}), *G* – soil heat flux density (MJ m^{-2} day^{-1}), *T* – air temperature at 2 m height (^{o}C), u_2 – wind speed at 2 m height (m s^{-1}), e_s – saturation vapour pressure (kPa), e_a – actual vapour pressure (kPa), (e_s - e_a) – the saturation vapour pressure deficit (kPa), Δ – slope of the vapour pressure curve (kPa ^{o}C^{-1}), and γ – psychrometric constant (kPa ^{o}C^{-1}).

Using the Equation 6-2 with 24-h time steps, *G* is typically presumed to be 0 and e_s is computed as $(e_o(T_{max}) + e_o(T_{min}))/2$ where e_o is the saturation vapour function, *Tmax* and *Tmin* are the daily maximum and minimum air temperature (Allen, 2000). *ETo* was calculated using data from South Africa Weather Services (SAWS) and 'New LocClim' – FAO software for the estimation of various agro-climatic data based on spatial interpolation of existing data in the FAO database. The primary data for the study area required in the estimation of ETo with Equation 6-2 and the ETo values computed with CROPWAT are presented in Table 6-1.

Table 6-1 Data for computation of evapotranspiration

	Tmax °C	Tmean °C	Tmin °C	RH max %	RH mean %	RH min %	Vapour. pressure kPa	Wind speed m/sec	Sun shine hours	Rad MJ/m^2.day	ETo mm/day
Jan	26.1	20.3	15.6	90	68	46	1.67	7.3	10.44	28	6.0
Feb	26.3	20.3	15.5	92	69	45	1.71	7.4	10.29	25.3	5.6
Mar	25.3	19.2	14.1	94	70	46	1.63	5.7	9.11	20.4	4.7
Apr	23	16.8	11.8	95	73	50	1.48	5.5	7.23	14.6	3.5
May	20.2	14.3	9.3	96	76	55	1.38	4.5	5.52	10.4	2.3
Jun	18.1	12.5	7.8	97	77	57	1.16	4.2	6.07	9.2	2.1
Jul	17.3	11.8	7	96	76	55	1.12	4.3	6.02	9.7	2.0
Aug	17.7	12.3	7.5	95	75	55	1.12	4.1	6.43	12.8	2.5
Sep	19.2	13.6	8.6	94	72	50	1.2	4	7.27	16.9	3.2
Oct	21.2	15.6	10.6	92	70	47	1.28	4.5	8.54	22.1	4.3
Nov	23.5	17.8	13.1	89	68	46	1.43	5.5	9.57	25.7	5.3
Dec	24.8	19.5	14.8	90	68	45	1.58	6.1	10.44	28.3	5.8

The computed ETo values (with the modified FAO Penman-Monteith) were compared with estimates from Class A pan, Blaney-Criddle and Hargreaves methods discussed in Section 4.1.3 and results are as shown in Figure 6-2.

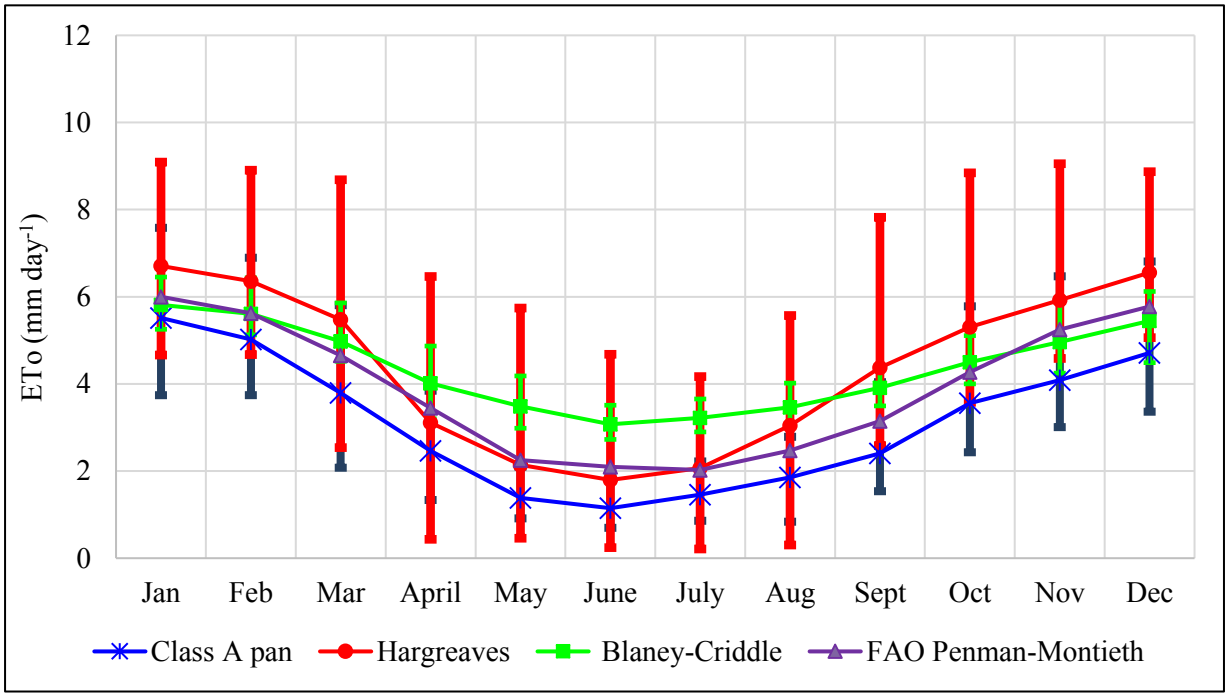


Figure 6-2 Estimated evapotranspiration values

The plot in Figure 6-2 shows that the modified FAO Penman-Monteith method provided a better match with respect to the Class A pan results than other empirical methods. The agricultural area in the study area depicted in Figure 6-1 mainly produces fresh vegetables comprising over 50 different types of crops (CCT, 2012). In this study, the CWR was only estimated for the five most representative crops widely grown in the area, *i.e.* potatoes, cabbages, small vegetables, green beans and tomatoes over two planting cycles per year. CWR was estimated using Equation 6-1 and equal to the mean precipitation deficit from the historical rainfall data provided by the South African Weather Services (SAWS) for the period modelled 2006 – 2015. Table 6-2 shows the output from the CROPWAT model showing irrigation requirements for the two planting cycles.

Table 6-2 Mean agriculture irrigation requirement for the two planting cycles

	Planting cycle 1											
Crops	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1. Potato	0	78	97.2	17.2	0	0	0	0	0	0	0	0
2. Cabbage	0	112.3	66.3	11.9	6.1	0	0	0	0	0	0	0
3. Small Vegetables	62.3	142.3	59.3	0	0	0	0	0	0	0	0	0
4. Green beans	0	81.8	85.5	2.8	0	0	0	0	0	0	0	0
5. Tomato	0	94.7	92.6	22.8	4.1	0	0	0	0	0	0	0
Net scheme irrigation requirement												
Irr. req. for actual area (l/s/h)	0.23	0.49	0.28	0.05	0.02	0	0	0	0	0	0	0
in mm/day	1	4.2	2.4	0.2	0	0	0	0	0	0	0	0
in mm/month	31.2	117.4	72	6.8	1.3	0	0	0	0	0	0	0
	Planting cycle 2											
Crops	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1. Potato	0	0	0	0	0	0	0	18.2	48.1	108.5	158.3	41.8
2. Cabbage	70.1	0	0	0	0	0	0	31.9	23.1	75.5	155.7	217.5
3. Small vegetables	0	0	0	0	0	0	0	47.9	58.6	39.4	0	0
4. Green beans	0	0	0	0	0	0	0	20.6	48.6	84.8	0	0
5. Tomato	0	0	0	0	0	0	0	24	33.4	104.8	172.7	145.4
Net scheme irrigation requirement												
Irrigated area. (% of total area)	14	0	0	0	0	0	0	100	100	100	38	38
in mm/day	0.3	0	0	0	0	0	0	1.2	1.6	2.1	2.1	1.7
in mm/month	9.8	0	0	0	0	0	0	36	48.2	66	61.5	52.9

The CWR values estimated from the mean precipitation deficit of the modelled period (2006 – 2015) are presented in Figure 6-3 in mm/month of rainfall deficit, with the values for the wettest year (2013) and driest year (2015) respectively providing the minimum and maximum limits represented with the range bars.

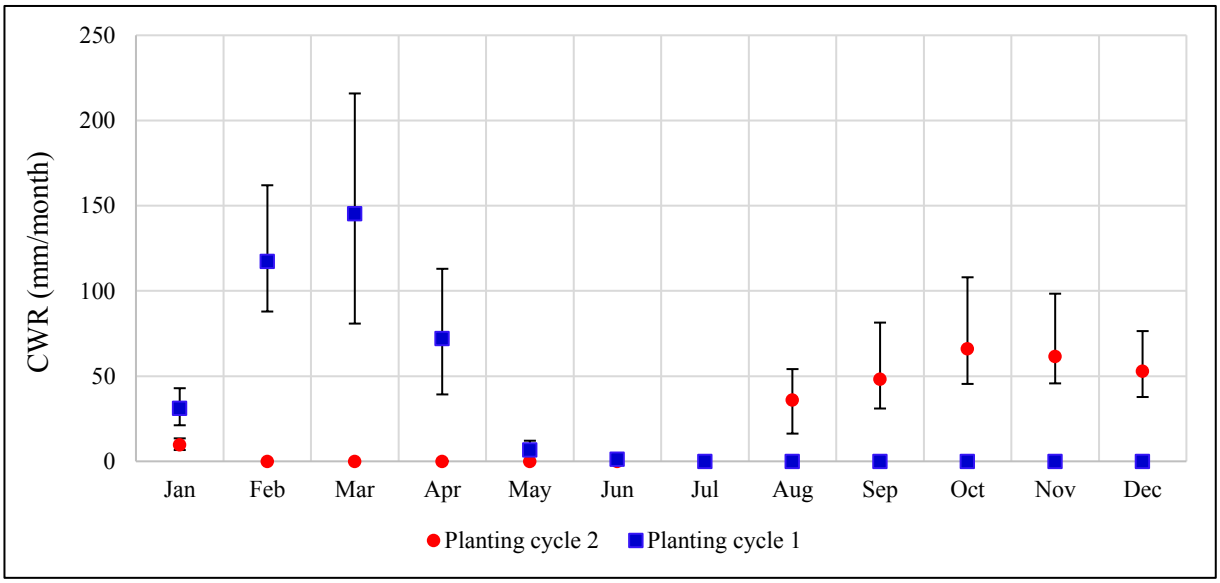


Figure 6-3 Mean monthly CWR estimates for the modelled period (2006 – 2015)

The aggregated mean monthly CWR estimates of both planting cycles for the modelled period (2006 – 2015) are given in Figure 6-4 in volumetric units (m^3), with the values for the wettest year (2013) and driest year (2015) respectively providing the minimum and maximum limits represented with the range bars.

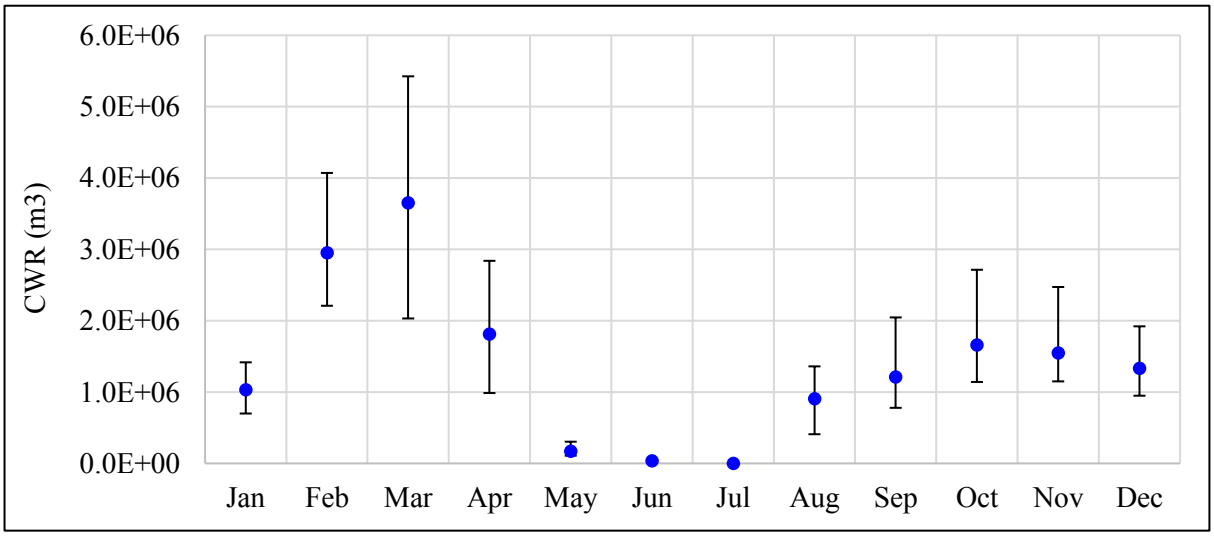


Figure 6-4 Aggregated mean monthly CWR estimates for the period (2006 – 2015)

The mean annual CWR requirement for both planting cycles for the modelled period (2006 – 2015) was estimated as 16.2 Mm^3 and the minimum and maximum values calculated based on the wettest year (2013) and driest year (2015) were 10.5 Mm^3 and 24.6 Mm^3 respectively. The CWR was also estimated for the same crops in the future based on data from climate change models available at the UCT with the precipitation deficit estimated from the mean value of the future rainfall data from climate change prediction models for the period 2090 – 2100. The output from CROPWAT showing irrigation requirements needed to meet precipitation deficit for the two planting cycles in the future is provided in Table 6-3.

Table 6-3 Future irrigation water requirement (2090 – 2100)

	Precipitation deficit (mm/month)											
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Planting cycle 1	38.7	133	134	58.5	27.7	1.7	0.6	0	0	0	0	0
Planting cycle 2	11.3	0	0	0	0	0	1.6	46.2	23.4	103.9	63.9	51.2
Total	50	133	134	58.5	27.7	1.7	2.2	46.2	23.4	103.9	63.9	51.2

The aggregated mean monthly CWR values for the modelled period (2090 – 2100) are given in Figure 6-5, including the minimum and maximum values calculated based on the wettest year (2092) and driest year (2095).

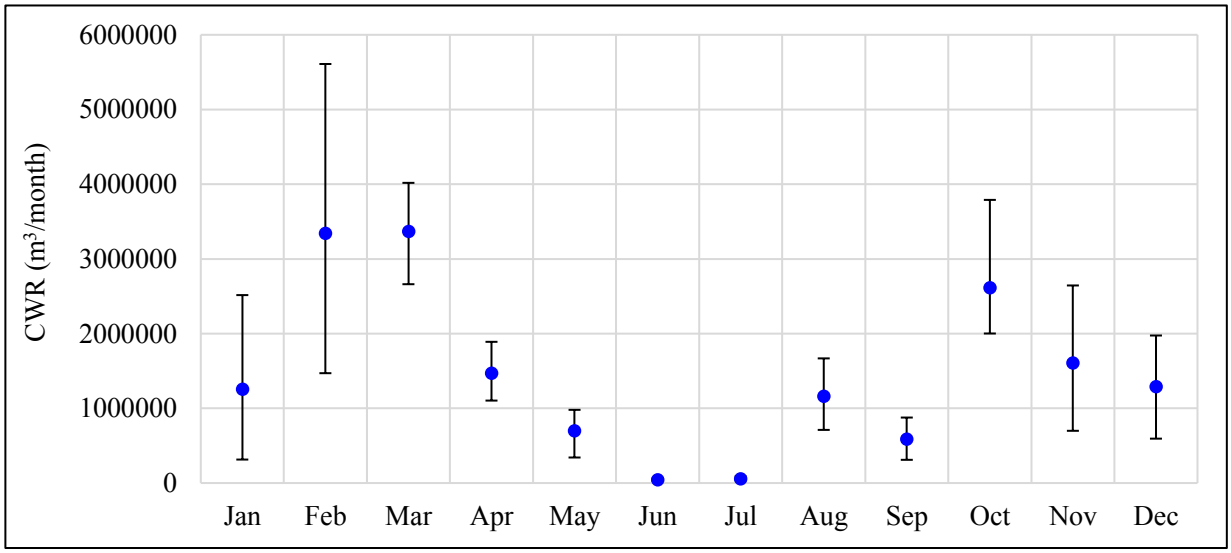


Figure 6-5 Aggregated mean monthly CWR

The mean annual CWR requirement of both planting cycles for the modelled period (2090 – 2100) was estimated as 17.5 Mm^3 and the minimum and maximum values calculated based on the wettest year (2092) and driest year (2095) were 11.3 Mm^3 and 26.5 Mm^3 respectively. The estimated values for agriculture demand in the study area were converted to volume per area per year (m^3 ha^{-1} yr^{-1}) and compared with typical annual mean agriculture water allocation to

farmlands in the same regions as the study area *i.e.* Western Cape Province in South Africa. An 'order-of-magnitude check' was used to determine whether the estimated values were within a similar range. For example, the annual mean agriculture water allocation to the upper Berg River farmlands is in the range of 4000 – 6000 m^3 ha^{-1} yr^{-1} (Nieuwoudt *et al.,* 2008). Based on the CROPWAT estimations, the annual mean CWR value for the agriculture in the study area covering the total of 2560 ha was 6,400 m^3 ha^{-1} yr^{-1} based on the historical data (2006 – 2015). The minimum and maximum values based on historical data were 4,100 and 9,600 m^3 ha^{-1} yr^{-1} respectively. The CWR value is projected to increase to a mean annual CWR value of 6,800 m^3 ha^{-1} yr^{-1} in the future (2090 – 2100) with minimum and maximum values of 4400 and 10300 m^3 ha^{-1} yr^{-1} respectively because of climate change. The minimum and maximum CWR values based on the historical data (2006 – 2015) were higher but comparable with the annual mean agriculture water allocation in the Berg River farmlands.

6.2 Public open spaces

Public open spaces are scattered in various locations in the study area as shown in Figure 6-6**.**

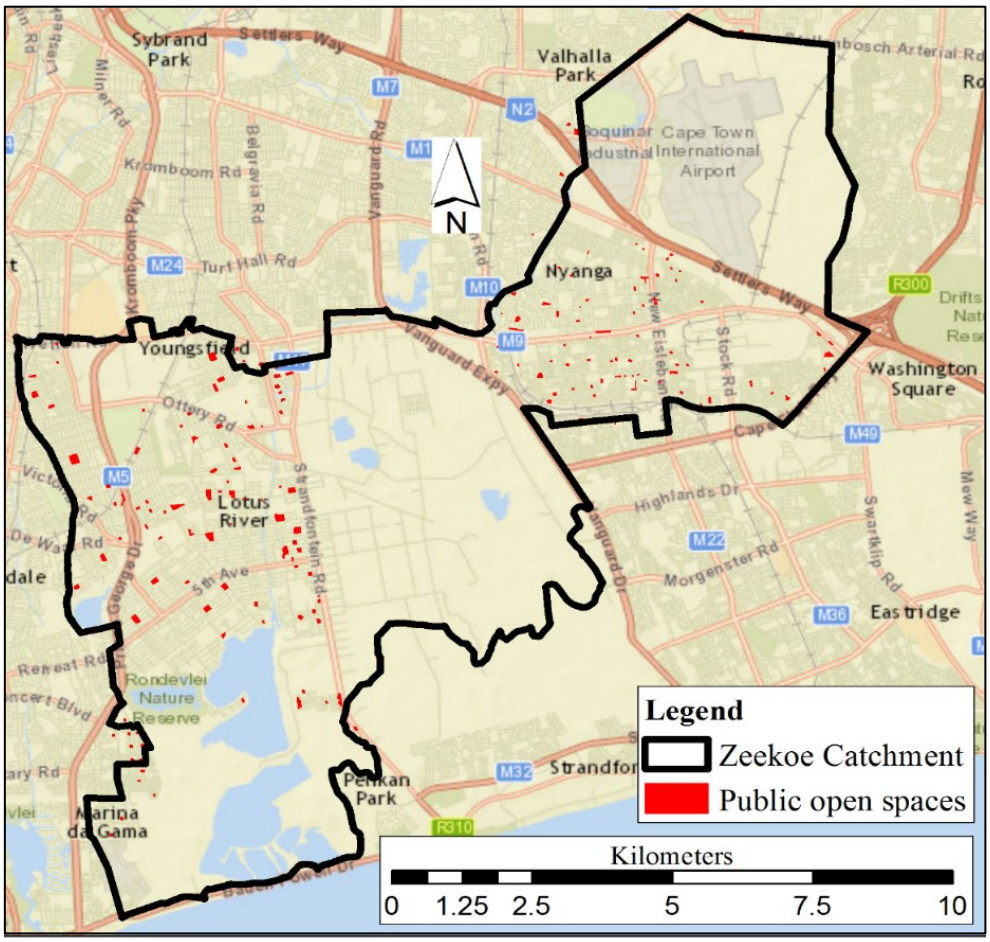


Figure 6-6 Various public open spaces in the study area (CCT, 2012)

In the determination of the irrigation water demand, a regularly maintained, well-watered and actively growing perennial grass was considered as a proxy for all the public open spaces scattered across the study area as shown in Figure 6-6. The irrigation demand for public open spaces was then estimated from Equation 6-1 for the modelled period (2006 – 2015) (Figure 6-7), including the maximum and minimum for the wettest year (2013) and driest year (2015).

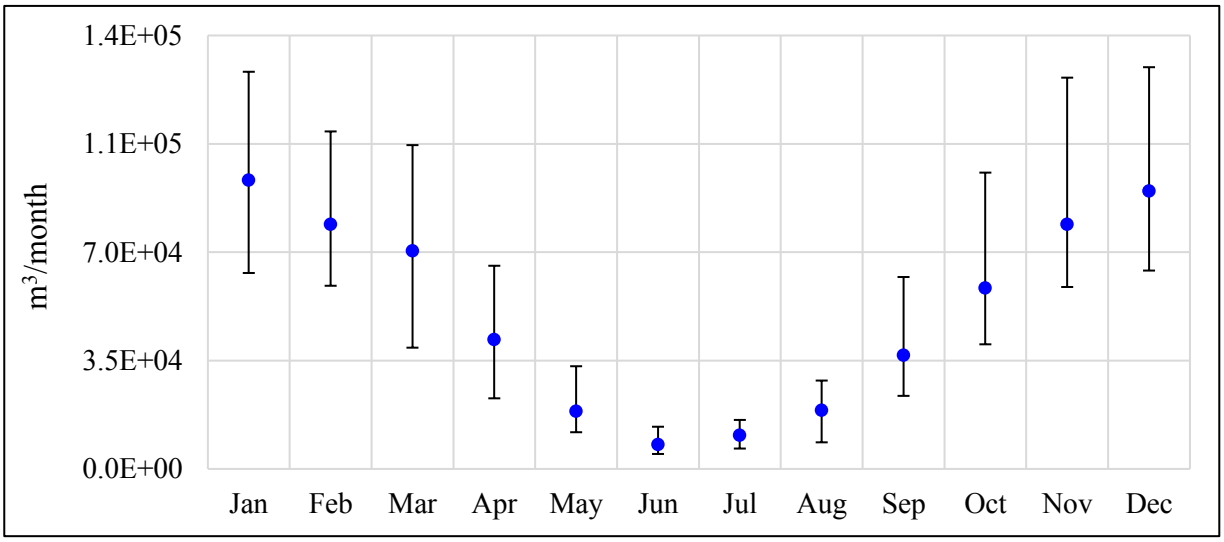


Figure 6-7 Public open spaces monthly irrigation requirement

The mean annual CWR for irrigation of public open spaces for the modelled period (2006 – 2015) covering a total of 2 km^2 was estimated at 0.61 Mm^3 and the minimum and maximum values calculated based on the wettest year (2013) and driest year (2015) were 0.93 Mm^3 and 0.41 Mm^3 respectively. The mean annual CWR requirement for irrigation of public open spaces in the future based on data from the selected climate change prediction models for the modelled period (2090 – 2100) was estimated as 0.82 Mm^3 and the minimum and maximum values calculated based on the wettest year (2092) and driest year (2095) were 0.54 Mm^3 and 1.26 Mm^3 respectively. The estimated water demand for public open spaces in the study area was also converted to volume per area per year (m^3 ha^{-1} yr^{-1}) and compared with the mean annual irrigation demand of a 63.7 ha golf course that already harvests and re-uses stormwater for irrigation as an 'order of magnitude' check. The mean annual irrigation demand of the 63.7 ha golf course was estimated as 2500 m^3 ha^{-1} yr^{-1} (mainly stormwater pumped from the drainage canals) and used for irrigation and cleaning at the golf course (Bodenstein, 2017). The annual mean CWR value for public open spaces for the study area was estimated as 3000 m^3 ha^{-1} yr^{-1} based on the historical data (2006 – 2100) projected to increase to 4100 m^3 ha^{-1} yr^{-1} in future based on data from climate change prediction model. The estimated water demand for public open spaces was higher than the mean annual stormwater re-use at the golf course likely due to the assumption used in the study area of a regularly maintained, well-watered and actively growing perennial proxy grass. Also, it is likely that not all of the golf course site is irrigated *e.g.* sand traps and tree verges would normally require watering.

6.3 Residential garden irrigation

In the estimation of the water use for residential garden irrigation, it was necessary to disaggregate the domestic water demand into specific end uses. Domestic demand can be categorised as indoor or outdoor (Jacobs & Haarhoff, 2004). Residential garden irrigation is an outdoor demand together with others such as swimming pools and car washing (Jacobs & Haarhoff, 2004). The following steps were taken to determine the residential garden irrigation demand of the study area:

i. An inspection of the historical Google Earth imagery of the area was undertaken to determine the level of land use change over time between 2005 and 2015 (Figure 6-8). The assessment showed that there was not much change in the number of houses on separate stands as shown in a portion extracted from the study area (Figure6-8). Thus no adjustment was undertaken over the model period 2006 – 2015.

Figure 6-8 Historical Google Earth imagery extract of a section in the study area

ii. The comprehensive household data collected in the South African Census of 2011 was used to determine the number of houses in the study area. An extract from the census database for the study area is presented in Table 6-4.

Table 6-4 Number and types of houses per suburb in the study area (StatsSA 2011)

	Houses on a separate stand	Traditional structure	Flat/ cluster	Semi-detached house	backyard/ informal
Gugulethu	10947	126	1410	1530	15561
Nyanga	8595	30	1134	792	5439
Crossroads	5346	21	27	21	5241
Philippi	25401	183	1659	543	36627
Lotus River	6591	18	1056	591	636
Parkwood	1248	6	717	42	444
Grassy Park	4029	9	384	84	204
Zeekoevlei	117	3	0	0	9
Pelican Park	2250	33	27	57	888
Wynberg	1977	24	2100	879	147
Wetton	723	-	57	66	12
Ottery	1749	0	141	105	210
Royal Cape	183	-	0	3	12
Elfindale	663	-	96	24	57
Southfield	1791	3	186	102	72
Seawinds	465	3	6	738	177
Lavender Hill	2310	18	1338	267	1182
Vrygrond	2568	39	138	321	2172
Plumstead	4566	27	1671	672	144
Total	81519	543	12147	6837	69234

iii. CCT provides an online interactive map at http://emap.capetown.gov.za/egisviewer/ with a provision to measure lengths and areas of features on properties. The houses (Table 6-4) were grouped according to property sizes to coincide with water use bands of CCT. The categories are <200 m^2, 200 – 500 m^2, 500 – 1000 m^2, 1000 – 1500 m^2, 1500 – 2000 m^2, and > 2000 m^2. The mean property size in each category was estimated and used as a representative area per suburb as presented in Table 6-5. In the measurement process, it was determined that stand areas less than 200 m^2 for all suburbs and houses in the 200 – 500 m^2 category for some of the suburbs such as Nyanga, Gugulethu, Crossroads and Philippi did not have residential garden areas for irrigation. The houses without residential garden areas were thus excluded

from the analysis. With the elimination of the properties without residential garden areas, a total of 69,329 houses in the entire study area were considered for estimation of the irrigation demand and design of a water supply reticulation system.

Table 6-5 Mean stand area per category

Suburb	Mean stand area per category (m^2)					
	< 200	200 - 500	500 - 1000	1000 - 1500	1500 - 2000	> 2000
Crossroads	158	297	658	1384	1761	2079
Elfindale	124	374	708	1159	1610	2859
Grassy Park	161	371	667	1211	1764	2317
Gugulethu	146	386	641	1236	1613	2893
Lavender Hill	151	297	650	1130	1558	2323
Lotus River	170	360	650	1190	1653	2428
Nyanga	145	351	656	1235	1899	2486
Ottery	154	377	675	1131	1681	2014
Parkwood	159	430	634	1209	1707	2205
Pelican Park	142	333	646	1066	1916	2321
Philippi	150	317	680	1271	1751	2308
Plumstead	140	386	621	1180	1990	2799
Seawinds	140	315	594	1227	1773	2320
Southfield	160	376	608	1209	1903	2317
Wetton	154	373	633	1131	1681	2014
Wynberg	139	337	665	1225	1651	2403
Zeekoevlei	126	312	630	1133	1727	2321

The total irrigation demand was estimated using Equation 6-1 for urban agriculture based on the mean property size in each category and suburb, typical residential garden areas and the total number of houses on separate stands. A regularly maintained, well-watered and actively growing perennial grass as was assumed – as for public open spaces. The mean monthly CWR values for the residential garden for the modelled period (2006 – 2016) are provided in Figure 6-9, including the maximum and minimum for the wettest year (2013) and driest year (2015). The CWR for residential garden irrigation for study period (2006 – 2016) was estimated as 9.86 Mm^3 with minimum and maximum values as 6.6 Mm^3 and 15.1 Mm^3 for the wettest (2013) and driest (2015) years respectively. The predicted CWR requirement for residential garden irrigation in the future based on climate change models for the modelled period (2090 – 2100) was estimated at 13.3 Mm^3 with minimum and maximum values as 8.8 Mm^3 and 20.5 Mm^3 with the minimum and maximum for wettest year (2092) and driest (2095) years respectively.

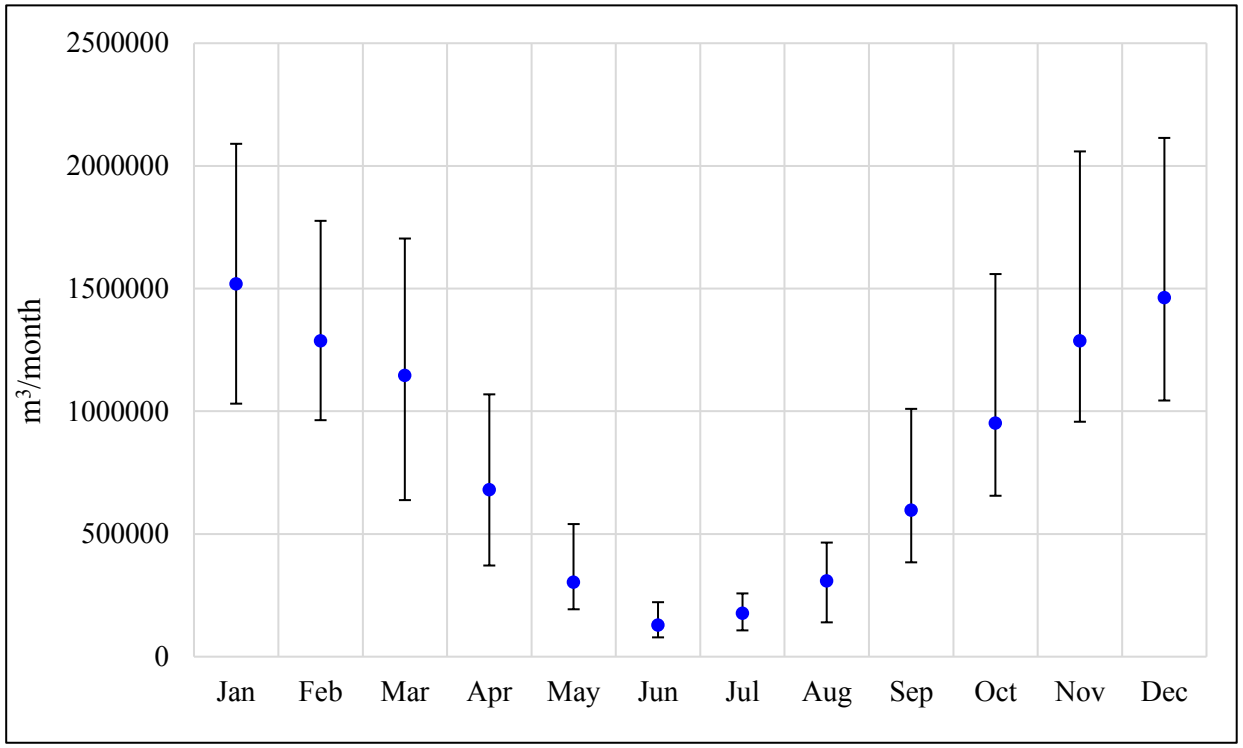


Figure 6-9 Mean monthly residential garden irrigation demand for the study area

6.4 Water-use estimation for toilet flushing

The unit water demand analysis method (Rinaudo, 2015), one of the approaches discussed in Section 2.6.4, was adopted for the estimation of the demand for toilet flushing. The toilet flushing demand was estimated from the number of houses in the study area (Table 6-4), the number of people in a household (Table 6-6), and the expected mean number of flushes per person. Studies such as Smith, (2010) and van Zyl *et al.,* (2008) have determined a typical frequency of toilet use (flushes/day) as four flushes/person/day for medium to high-income and three flushes/person/day for low income. The number of people and level of income as a percentage of households per suburb are as shown in Table 6-6. Based on the unit water demand analysis method, the total annual amount of water for toilet flushing was estimated at 3.8 Mm^3 for the study area for the current development. Since this was an indoor water demand, it is not really impacted by seasonality.

Table 6-6 Household sizes and income groups as a percentage (StatsSA, 2011)

Suburbs	HH size	Low income	Middle income	High income
Gugulethu	3.3	48%	50%	1%
Nyanga	3.6	50%	49%	1%
Crossroads	3.4	58%	42%	1%
Philippi	3.1	51%	48%	1%
Lotus River	4.3	24%	69%	8%
Parkwood	4.8	28%	70%	2%
Grassy Park	4.1	21%	66%	13%
Zeekoevlei	3.4	12%	48%	40%
Pelican Park	3.8	27%	60%	12%
Wynberg	2.8	18%	59%	22%
Wetton	3.8	10%	60%	30%
Ottery	3.5	21%	48%	30%
Plumstead	2.9	13%	55%	32%
Elfindale	3.1	20%	46%	34%
Southfield	3.3	14%	56%	29%
Lavender Hill	5.1	40%	59%	1%
Seawinds	4.8	28%	71%	1%
Vrygrond	4.8	51%	49%	1%

6.5 Summary of non-potable demand estimation

In this study, it was determined that the Zeekoe Catchment presents a realistic opportunity for stormwater supply to non-potable water demands including irrigation of urban agriculture, residential gardens, public open spaces and toilet flushing. The potential use of stormwater was considered as a supplementary source to existing resources (*i.e.* water from dams) and initially limited to non-potable water demands to minimise the need for costly water treatment. The mean annual urban agricultural demand of 16.3 Mm^3 presents a significant non-potable water demand that could be readily supplied by stormwater. The irrigation of public open spaces with an estimated mean annual demand of 0.6 Mm^3 also presents an opportunity for non-potable water supply with stormwater. Some parks and golf courses in the study area are already being supplied with stormwater for irrigation and cleaning activities. For domestic water demand *i.e*. garden irrigation and toilet flushing, the mean annual demand was estimated to be about 9.8 Mm^3 and 3.8 Mm^3 respectively. The households with a garden area for irrigation were included in the estimation of non-potable water demand as they would benefit from economies of scale, *i.e.* supply of a large volume of stormwater.

6.6 Storage requirement for stormwater supply

6.6.1 Demand categories

The non-potable water demands were categorised as Sc 1 – agriculture; Sc 2 – residential garden irrigation and toilet flushing; and Sc 3 – residential garden irrigation, toilet flushing and irrigation of public open spaces. The estimated mean annual demand volumes of the three scenarios, *i.e.* Sc 1, Sc 2 and Sc 3 over the ten-year period (2006 – 2016) was 16 Mm^3, 14 Mm^3 and 14 Mm^3 respectively. The mean monthly demand volumes over the periods are presented in Figure 6-10.

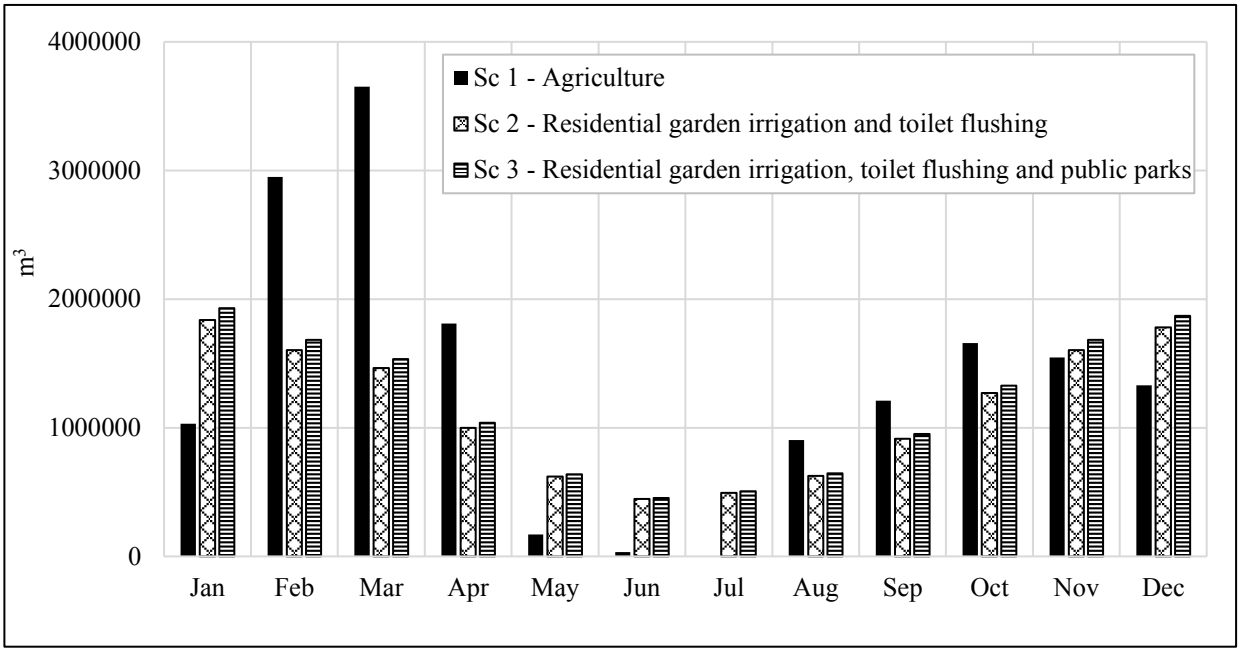


Figure 6-10 Mean monthly estimated demand volumes

6.6.2 Assessment of storage for stormwater harvesting and supply

A Volumetric Reliability (VR) assessment was undertaken to determine the optimal storage requirement and reliability of stormwater supply to meet demand. According to Mitchell *et al.,* (2008), VR is the ratio of the volume of water supplied to total water demand in a given study period as determined by Equation 6-3 (Fewkes & Butler, 2000; Palla *et al.,* 2011):

$$VR = \frac{\sum_{t=1}^{T} Y_t}{\sum_{t=1}^{T} D_t} \qquad \textbf{Equation 6-3}$$

Where: Y_t = yield in m^3 at time t; D_t = water demand in m^3 at time t; T = analysis period.

The yield was estimated with Equation 4-12 (Mitchell *et al.*, 2008) as discussed in Section 4.4 and the optimal total storage capacity was estimated in a 1 Mm^3 incremental stepwise manner with the determination of VR using at Equation 6-3. The optimal capacity was assessed against the need for balancing storage required for SWH to determine the reliability and adequacy using Equation 6-3. The capacity in the Zeekoevlei and Rondevlei (Figure 3-10) of 5 Mm^3 and 1 Mm^3 respectively was also assessed against the need for balancing storage for SWH. The objective of the assessment was to determine the benefits of the additional capacity with regard to maximising yield and minimising spillage.

6.6.3 Optimal storage requirement for the non-potable water

The ideal storage required in the study area to account for the mismatch in the availability of stormwater and the various demand scenarios was estimated in a stepwise manner with capacity provided in the vleis. The simulation was based on the YAS model (Equation 4-12) with RTC on the stormwater ponds and the vleis (Zeekoevlei and Rondevlei). The results, presented in the Figure 6-11, show that SWH with 1 Mm^3 balancing storage (current capacity available in the stormwater ponds) was only adequate to supply 44%, 60% and 58% for demands in Sc 1, Sc 2 and Sc 3 respectively (Figure 6-11). Increasing the storage (*i.e.* using the available capacity in the vleis) increased yield and decreased spillage in the various demand options (Figure 6-11).

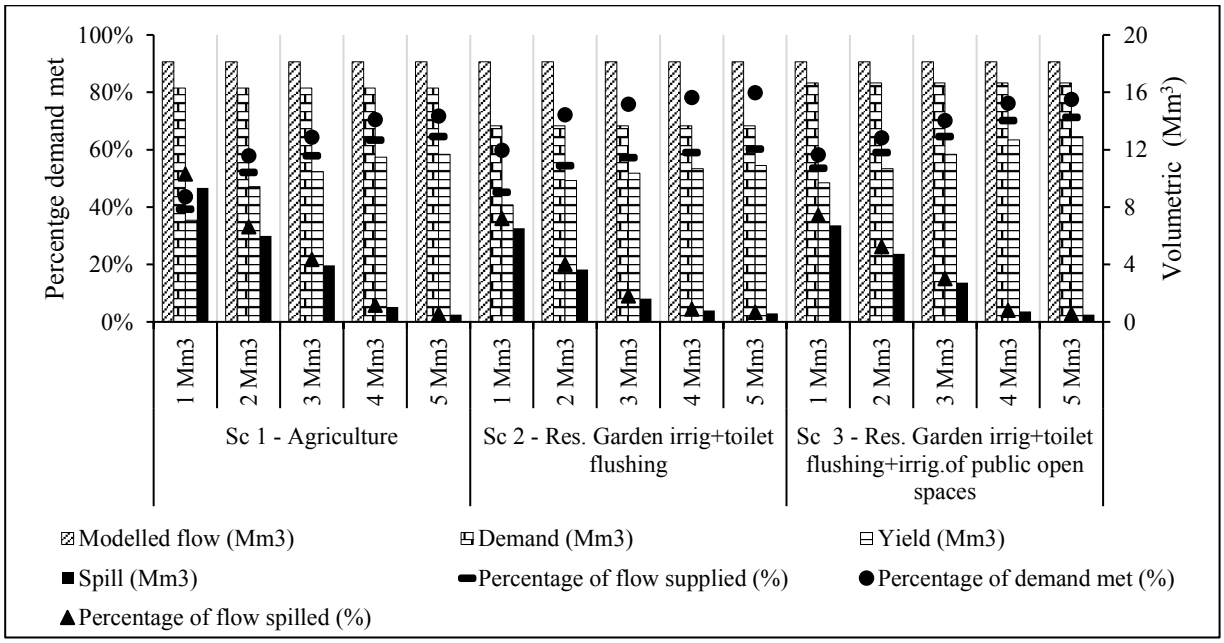


Figure 6-11 Assessment of storage for the various demand scenarios

It was determined that after 4 Mm^3, there was limited improvement and insignificant additional benefit for the various demand options as shown in Figure 6-12. Thus, it was determined that a balancing storage of 4 Mm^3 was adequate for the modelled stormwater volume to meet a

significant portion of expected demand with minimal spillage. Since the stormwater ponds could only provide a total of 1 Mm^3 with application of RTC and the physical expansion unlikely due to land limitations typical in urban areas, enlarging the vleis (Zeekoevlei and Rondevlei) to provide the additional storage seems to be the most promising option. However, the vleis are currently used for other purposes such as recreation – including sailing – and to maintain ecology requiring a permanent pool of water. The ecological sensitivity and recreational activities in the vleis was the driver for the consideration of alternative storage options such as groundwater through Managed Aquifer Recharge and Recovery (MAR&R).

6.6.4 Stormwater harvesting and supply for potable demand

Stormwater could also be abstracted from the two most downstream vleis (Zeekoevlei and Rondevlei), fully treated to potable water standards and injected into the local potable water distribution system. Alternatively, the abstracted water from the vleis could be pre-treated at a new proposed WTP in the study area and then pumped to one of the existing water treatment plants (WTPs) as indicated in Figure 3-11. A similar assessment as discussed in Section 6.6.3 was undertaken with SWH to supply potable water. The results with various storage and pump capacities with corresponding yield and spillage are presented in Figure 6-12.

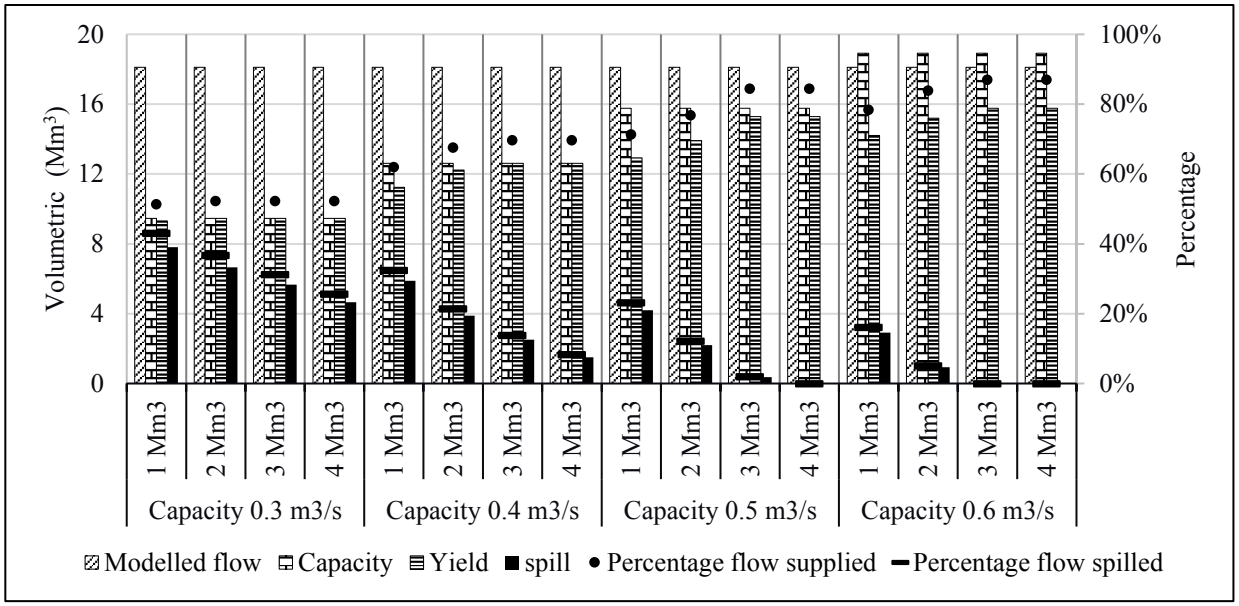


Figure 6-12 Selection of pump capacity for stormwater transmission

It was determined that for potable water that is required all the year round, the yield was not that sensitive to changes in storage volume (Figure 6-12) but linked rather to the capacity of the water delivery system. Since the influence of the local balancing storage was limited, optimisation was based on treatment, pump and pipe capacity to maximise yield and minimise spillage. As presented in Figure 6-12, the most suitable plant was with capacity of 0.5 m^3/s since above this,

there was limited increase in yield and reduction of spillage. The proposed transmission pipeline plan is shown in Figure 3-11 whilst a trial design (in Appendix 8) indicates a need for a DN 450 PN 20 pipe, and two on-line booster stations.

6.7 Stormwater quality

6.7.1 Overview

The CCT collects grab samples at several points in the study area including at the locations proposed for stormwater harvesting *i.e.* Zeekoevlei and Rondevlei as shown in Figure 6-13.

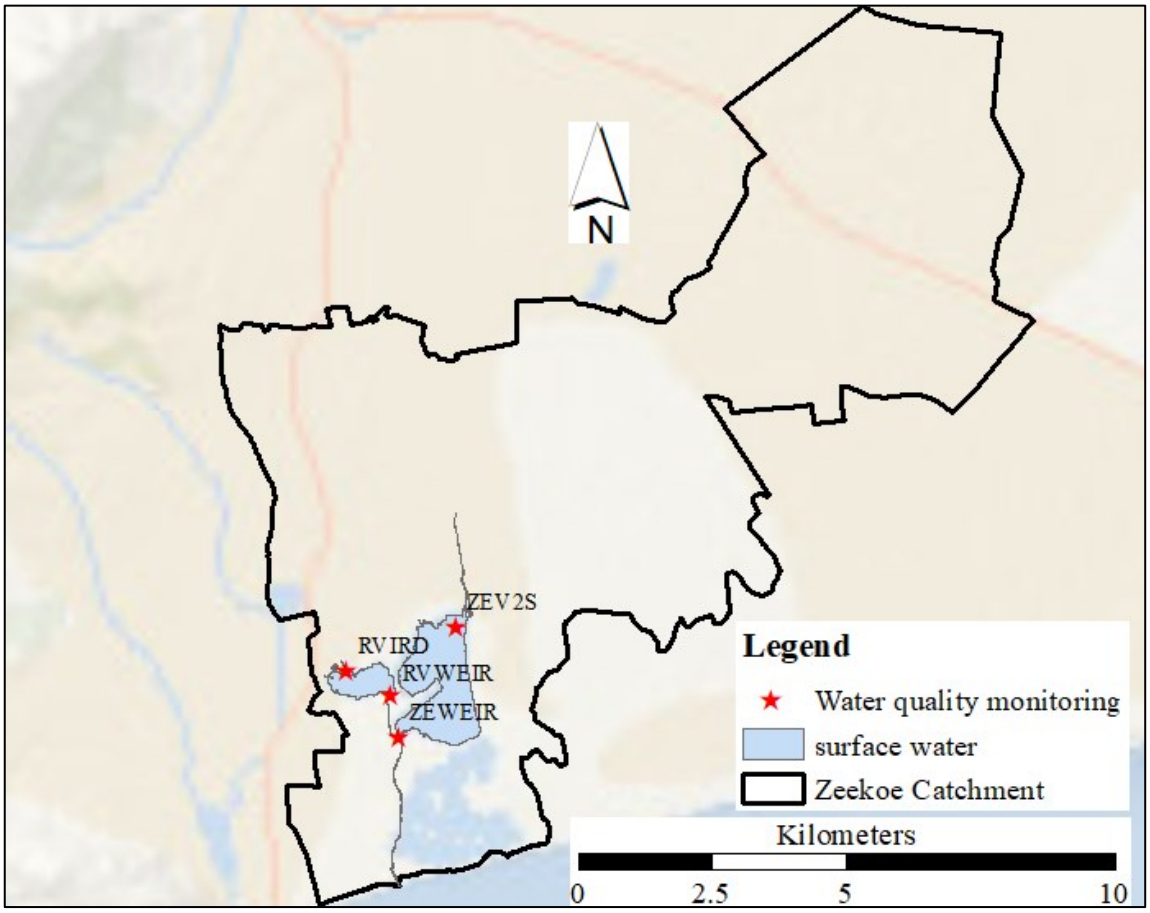


Figure 6-13 Water quality monitoring sites in Zeekoevlei and Rondevlei (*after* CCT, 2015)

Stormwater quality data was collected and compared with the South African Water Quality Guidelines for Irrigation (Volume 4: Agricultural Water Use: Irrigation Second Edition, 1996) and National Drinking Water Standards (SANS 241:2015) to determine the treatment requirement. An extract from Volume 4: Agricultural Water Use: Irrigation Second Edition, 1996 and National Drinking Water Standards is presented in Tables 6-7 and 6-8.

Table 6-7 South African Water Quality Guidelines for Irrigation

Parameter	Risk	Limits	Parameter	Risk	Limits
E.coli (count/100ml)	health	1 – 1000	Arsenic mg/L	health	0.1 – 2
Conductivity (at 25ºC mS/m)	aesthetic	40 – 540	Cadmium mg/L	health	0.01 – 0.05
Total Dissolved Solids (mg/L)	aesthetic	260 – 3500	Chromium mg/L	health	0.1 – 1
pH		6.5 – 8.4	Lead mg/L	health	0.2 – 2

Table 6-8 Drinking water standards for SANS 241:2015

Parameter	Risk	Limits	Parameter	Risk	Limits
E.coli (count/100ml)	Acute	Not detected	Arsenic µg/L	Chronic	≤ 10
Conductivity (at 25oC mS/m)	aesthetic	≤ 170	Cadmium µg/L	Chronic	≤ 3
Total Dissolved Solids (mg/L)	aesthetic	≤ 1200	Chromium µg/L	Chronic	≤ 50
Nitrate + Nitrite (mg/L)	Acute	≤ 11	Lead µg/L	Chronic	≤ 10
Nitrate (mg/L)	Acute	≤ 10	Mercury µg/L	Chronic	≤ 6

6.7.2 Pathogen pollution and treatment needs

Water quality data on pathogen pollution with *E.coli* as the indicator organism was obtained from CCT for ten years (2007 – 2016). The data was analysed to determine the extent to which water treatment would be required for potable and non-potable water uses based on the South African Water Quality Guidelines for Irrigation and National Drinking Water Standards (Table 6-7) respectively. The data shows a very high level of pathogen pollution at the inlets, however, there were significantly lower counts at the outlet as shown in Figure 6-14 and 6-15.

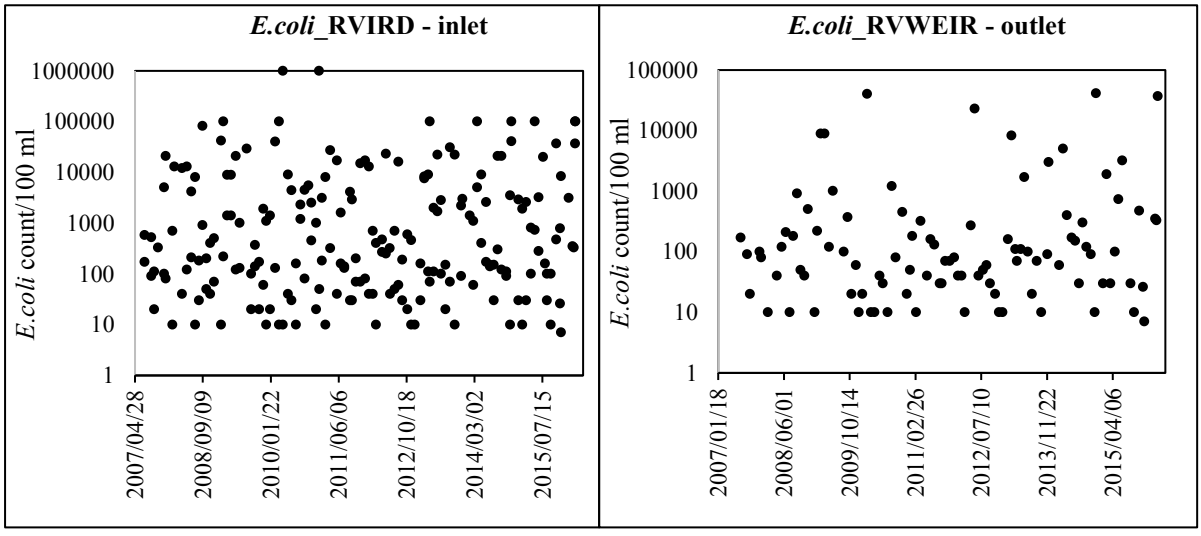


Figure 6-14 *E.coli* time-series measured at Rondevlei (*after* CCT 2017)

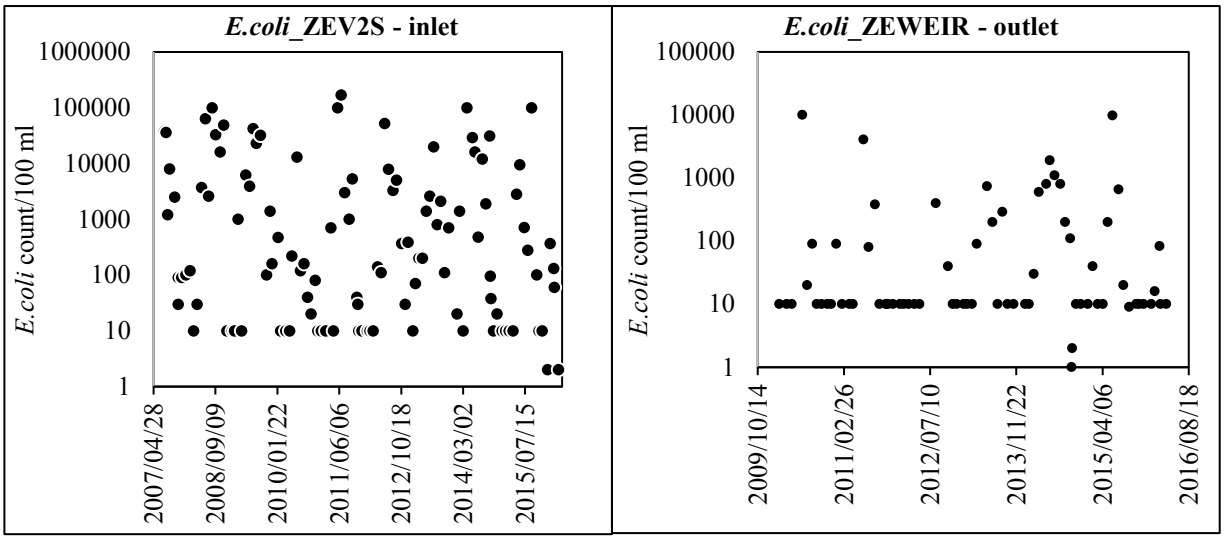


Figure 6-15 *E.coli* time-series measured at Zeekoevlei (*after* CCT 2017)

The very high levels of *E.coli* indicate that the surface water would require significant treatment even for non-potable uses. The associated costs are discussed in Chapter 7.

6.7.3 Nutrients

The CCT also test for several nutrients. The results show relatively low levels of nutrients as presented in Figure 6-16.

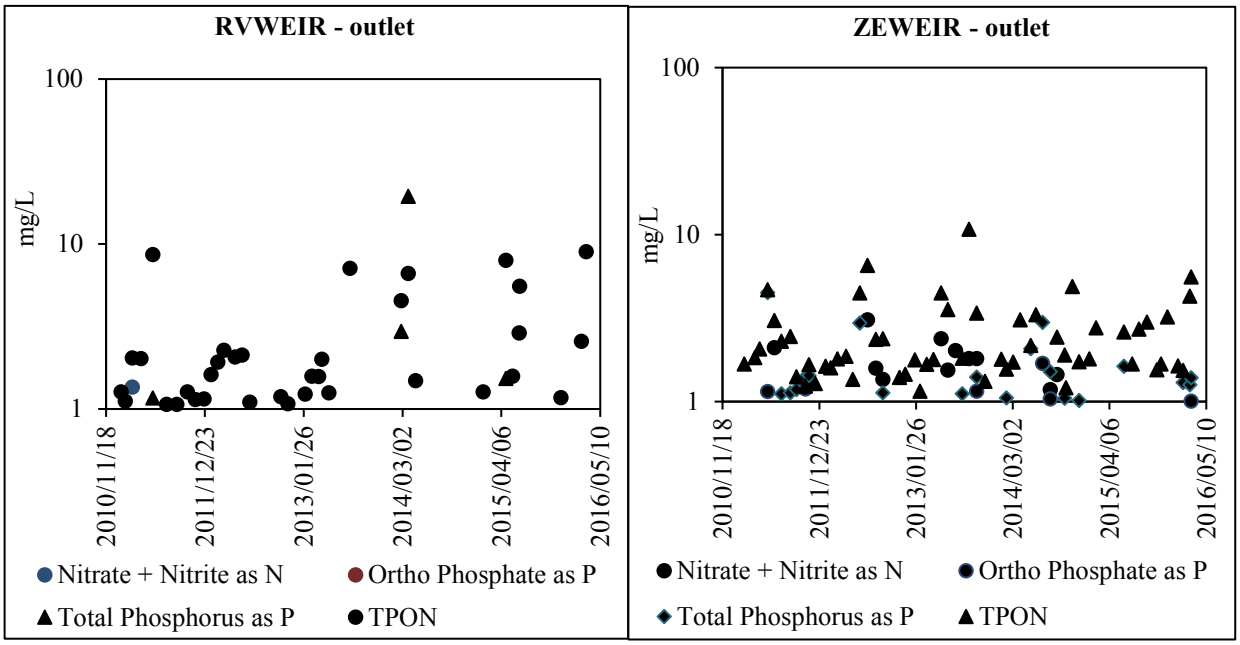


Figure 6-16 Nutrient time series measured at the outlet of the vleis

With the relatively low nutrient concentrations, no additional treatment was proposed for non-potable uses. In any case, the use of stormwater with nutrients for agriculture, residential gardens and public open spaces would be beneficial as it would decrease the amount of fertilizer required (Candela *et al.,* 2007).

6.7.4 Electrical conductivity and total dissolved solids

The monthly grab samples presented in Figure 6-17 show relatively low Electrical Conductivity (EC) levels compared with what is allowable under the drinking water standards (Table 6-8). The CCT does not collect data on Total Dissolved Solids (TDS) but there is usually a relatively good correlation of this with EC (DWA, 1996), typically estimated with Equation 6-4.

$$TDS\ (mg/L) = EC\ (mS/m\ at\ 25^{o}C) \times 6.5 \qquad \textbf{Equation 6-4}$$

Where TDS – Total Dissolved Solids (mg/L)

EC – Electrical conductivity (mS/m)

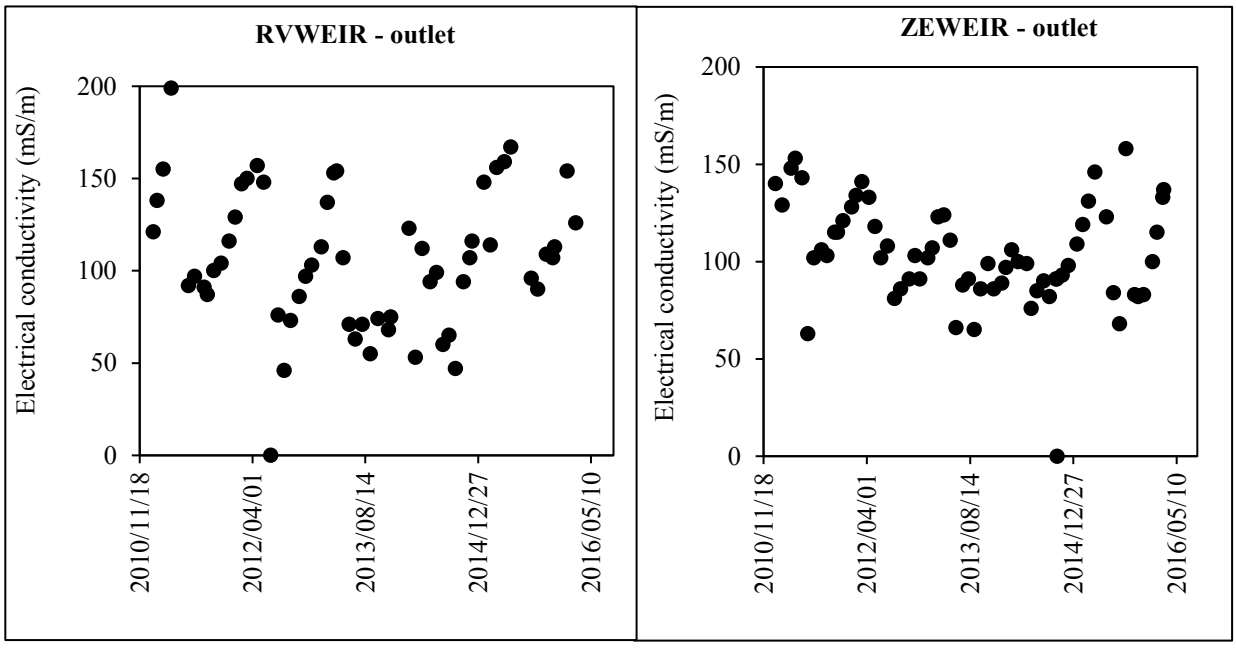


Figure 6-17 Total dissolved solids and Electrical conductivity at the outlet of the vleis

To confirm the relationship presented by Equation 6-4, water samples were collected over a one-week period from 12th – 16th February 2018 at the inlets to Rondevlei (#1) and Zeekoevlei (#2 and #3), the outlet before sewage outfall (#4) and after sewage outfall (#5) as presented in the Figure 6-18. The data is presented in Figure 6-19 and 6-20.

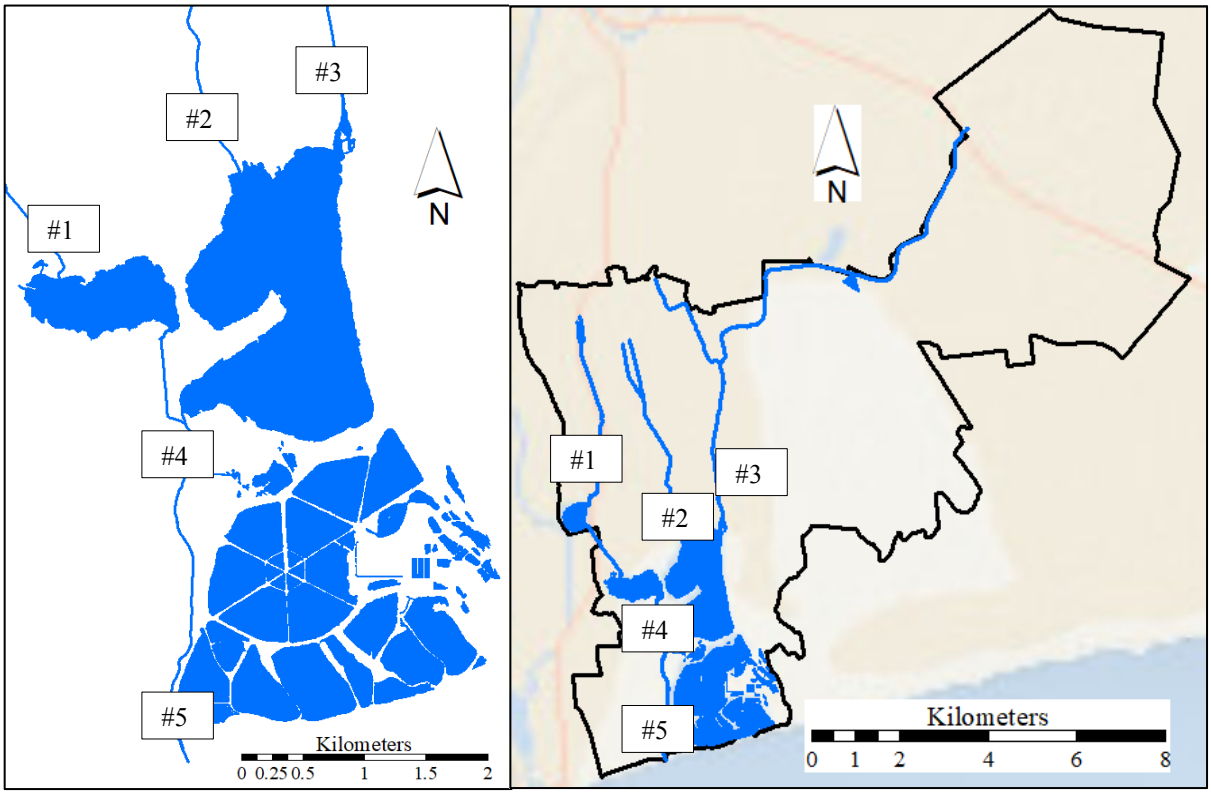


Figure 6-18 Water quality test sites for TDS and EC

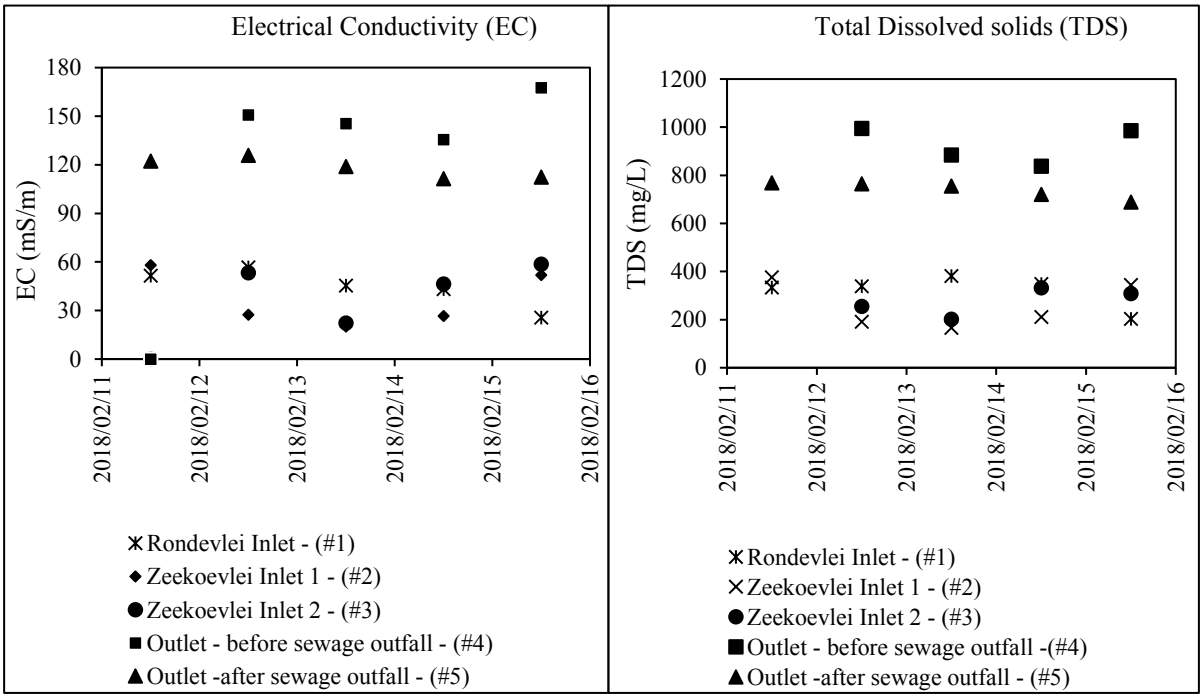


Figure 6-19 Total dissolved solids and electrical conductivity at the vleis

Linear relationships were generated for the TDS and EC values measured at the various locations. It was determined that the conversion factor in Equation 6-4 for the study area was in the range of 5.8 – 7.1 with a correlation coefficient of 0.41 – 0.94.

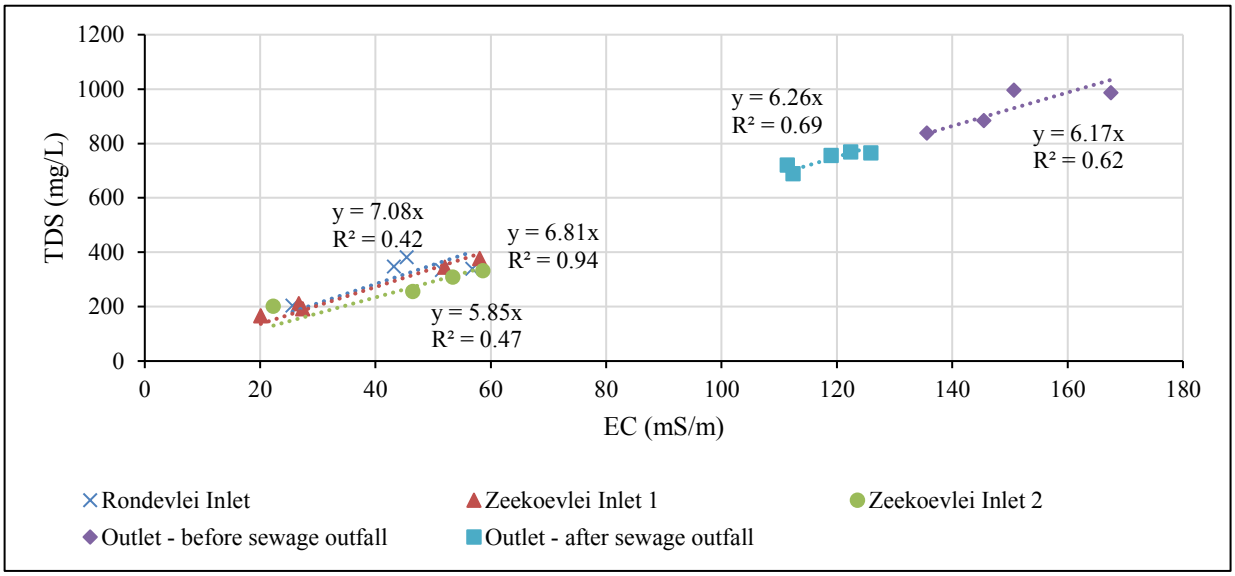


Figure 6-20 Linear relationships for TDS and EC

No specific treatment is proposed for TDS and EC as the measured values were relatively low compared with the South African Standards in Table 6-7 and Table 6-8 respectively.

6.7.5 Heavy metals

Stormwater samples were collected from 20th – 24th June 2016 at various points in the study area including at locations #3 and #4 in **Figure 5-18** in a research collaboration with an MSc student from Swiss TPH. They were then tested for various heavy metals including Arsenic (Ar), Cadmium (Cd), Chromium (Cr), Lead (Pb) and Mercury (Hg) (Table 6-9).

Table 6-9 Data for heavy metal in the surface water

	Inlet of Zeekoevlei (µg/L)					outlet* (µg/L)				
Date	Ar	Cd	Cr	Pb	Hg	Ar	Cd	Cr	Pb	Hg
20/6	<3	<1	<7	<7	<5	<3	<1	<7	<7	<5
21/6	<3	<1	13	<7	<5	22	<1	12	<7	12
22/6	<3	<1	<7	<7	<5	<3	<1	<7	<7	22
23/6	<3	<1	<7	<7	<5	<3	<1	<7	<7	<5
24/6	7	<1	<7	<7	<5	13	<1	<7	<7	<5

*Combined outlet from Zeekoevlei, Rondevlei and the Cape Flats Waste Water Treatment Plant

With the grab samples indicating relatively low concentration of heavy metals in the vleis compared with the South African Standards in Tables 6-7 and 6-8 respectively, no specific treatment was proposed for heavy metals. The sporadic high concentration detected in samples on some days and locations is likely to be an indication of intermittent and specific point source pollution that needs to be identified and eliminated to avoid very costly treatment process.

6.8 Water treatment

In this study, potable and non-potable needs have been identified and quantified as potential demands that could be supplied with stormwater. The general quality of stormwater was also determined and the need for treatment established for both potable and non-potable demands. For stormwater supply to non-potable water uses, the water would need partial treatment and distribution through a separate reticulation system (sometimes called '*dual*' or '*third-pipe*' reticulation). Alternatively, the harvested stormwater could be fully treated to potable water standards and distributed through the CCT reticulation system either locally or after partial treatment and conveyance to an existing water treatment plant where it would be blended with the raw water stream from the external reservoirs. The treatment processes would be as follows.

- **Surface Storage** – abstraction; screening (to remove large suspended objects); and disinfection (typically chlorination 'shock treatment' to significantly reduce the very high pathogen levels (Scarlett *et al.*, 2015). Other water treatment processes would include rapid sand filtration; 'pH correction – addition of alkali, *e.g.* lime and de-chlorination to values less than 100 ppm (Ibrahim *et al.*, 2015).
- **Groundwater storage** – if the intention is to use the water for non-potable purposes or to transfer to one of the existing WTPs for blending and treatment, it is likely that no specific treatment is required.
- **For potable water uses** (irrespective of source) – full conventional treatment with sand/ultra-filtration and final disinfection with ultra-violet radiation or ozonation to ensure effective biocidal activity.

6.9 General overview and summary of results

Potable and various non-potable demands in the catchment were identified and estimated for purposes of determining the reliability and adequacy to be supplied with stormwater. The non-potable demands included irrigation of urban agriculture, public open spaces, residential gardens and toilet flushing. It was estimated that the mean annual irrigation demand for urban agriculture covering a total of 25.5 ha was 6,300 m^3 ha^{-1} yr^{-1} based on the historical data (2006 – 2015). The minimum and maximum values based on the historical data were 4100 Mm^3 and 9600 Mm^3 respectively. The mean annual irrigation demand for agriculture is projected to increase to a mean annual value of 6,800 m^3 ha^{-1} yr^{-1} in the future (2090 – 2100) based on data from climate change prediction models. The mean and maximum urban agriculture demand values based on the

historical data (2006 – 2016) were slightly higher but comparable to the annual mean agriculture water allocation in the upper Berg River farmlands which was in the range of 4000 – 6000 m^3 ha^{-1} yr^{-1} (Nieuwoudt *et al.*, 2008). The over-estimation in the drier years is likely due to the Penman-Monteith method used in the analysis as it was determined to over-estimated evapotranspiration values compared to from Class A pan.

The mean annual water requirement for irrigation of public open spaces in the study area for the modelled period (2006 – 2016) covering a total of 2 km^2 was estimated as 3000 m^3 ha^{-1} yr^{-1} based on the historical data (2006 – 2100) projected to increase to 4000 m^3 ha^{-1} yr^{-1} in future based on data from climate change prediction models. In the study area, there is a 63.7 ha golf course that already harvests and re-uses stormwater for irrigation. On mean, around 2500 m^3 ha^{-1} yr^{-1} of stormwater is pumped from the drainage canals and used for irrigation and cleaning at the golf course (Bodenstein, 2017). The estimated irrigation demand value for the public open gardens was also higher than the mean annual stormwater re-use at the golf course. The over-estimation is likely due to the assumption of a regularly maintained, well-watered and actively growing perennial proxy grass. It is also likely that not all the golf course area is irrigated.

The mean annual water requirement for residential garden irrigation in the study area for the modelled period (2006 – 2016) was estimated as 10 Mm^3 and minimum and maximum values calculated based on the wettest year (2013) and driest year (2015) were 7 Mm^3 and 15 Mm^3 respectively. The mean annual CWR requirement for residential garden irrigation in the future based on data from the selected climate change prediction models for the modelled period (2090 – 2100) was estimated as 13 Mm^3 and the minimum and maximum values calculated based on the wettest year (2092) and driest year (2095) were 9 Mm^3 and 20 Mm^3 respectively. With SWH for non-potable uses, a storage volume of 20 – 30% of mean annual flow was required to balance temporal mismatch in stormwater and demand. RTC on ponds provided storage equal to 5.5% of the mean annual flow volume to meet 44 – 67% of demands and 37 – 51% spill. RTC on both ponds and vleis provided storage equal to 22% of mean annual flow volume to meet 70 – 79% of demand and 4 – 11% spill.

Stormwater use for potable water demand is not as sensitive to changes in storage, thus the performance of the system was largely linked to the plant capacity for stormwater abstraction, treatment and supply. The suitable plant capacity was determined in an optimisation process to maximise harvested stormwater volumes and minimising spillage.

Overall, the study provided insight into opportunities for stormwater use with partial or full treatment for non-potable or potable water demands respectively. The study has also provided a useful understanding of the potential scale and magnitude of the available non-potable water needs. If the non-potable water needs are supplied with low-quality stormwater, it will reduce the demand on existing resources that have significantly been constrained. The associated costs have been determined and discussed in Chapter 7. In the economic analysis (Chapter 7), capital, operation and maintenance cost estimates have been provided for stormwater treatment to potable water standards and corresponding seawater desalination included for comparison purposes and assessment.

7. Economic analysis

In this chapter, the viability of stormwater harvesting (SWH) was assessed based on an economic analysis comparing the costs of abstraction and supply from surface and groundwater storage for both potable and non-potable water demand. The costs were also compared with other proposed alternative sources in Cape Town. At conception of the research, it was initially planned that the economic analysis would include additional benefits from SWH systems such as amenity to the local community and ecosystem services. Due to inadequate expertise to investigate these benefits, the economic analysis was limited to the estimation of costs and comparison with other sources. Some marginal estimates linked to amenity and ecosystem services derived from literature have been added in Appendix 12. A detailed investigation of additional benefits associated with SWH has been recommended in future studies. The economic analysis presented in this chapter is based on unit costs. The Unit costs in R/kL were estimated and compared with the indicative costs of competing water sources presented in the City of Cape Town 'Water Outlook Report' of May 2018 (CCT, 2018).

7.1 Method

To account for the time value of money in the economic analysis, it was necessary to match past, present and future costs. The economic analysis was based on the Net Present Value (NPV) method using Equation 7-1 as recommended in Swartz *et al.* (2013).

$$PV = \frac{FV}{(1+i)^n} \qquad \textbf{Equation 7-1}$$

Where P – Present Value; FV – Future Value; i = Interest rate; n = Number of years

The interest rate was determined from suitable proxies for public sector projects discount rate such as: the long-term rate on corporate bonds, the post-tax savings rate, and the cost of long-term state borrowing as real values *i.e.* nominal values adjusted for inflation (DEAT, 2002; Van Vuuren and Van Dijk, 2006). Figure 7-1 shows that the discount rate determined from the South African Government 10-year bond expressed as real values, *i.e.* adjusted for inflation (Van Vuuren and Van Dijk, 2006). For this study period (2006 – 2017), the mean 10-year government bond and inflation rates over the study period (2006 – 2017) were determined to be 8% and 5% respectively, giving a discount rate of 3% (*i.e.* the difference between the mean 10-year government bond and inflation rate) were used. To account for the potential uncertainty associated with the selected discount rate, the analysis was repeated with a 4.5% (*i.e.* the maximum difference between the 10-year government bond and inflation rate). The expected life

expectancy in Equation 7-1 to account for the frequency of replacement of the various components and equipment in the water supply system is given in Table 7-1. The life expectancy of the various components and equipment is associated with wear and tear. It is different from the design period that is linked to the length of time a facility will be able to meet demand. For this study, typical design periods and life expectancies of 30 and 50 years (Mackenzie, 2010) were selected and used in the economic analysis.

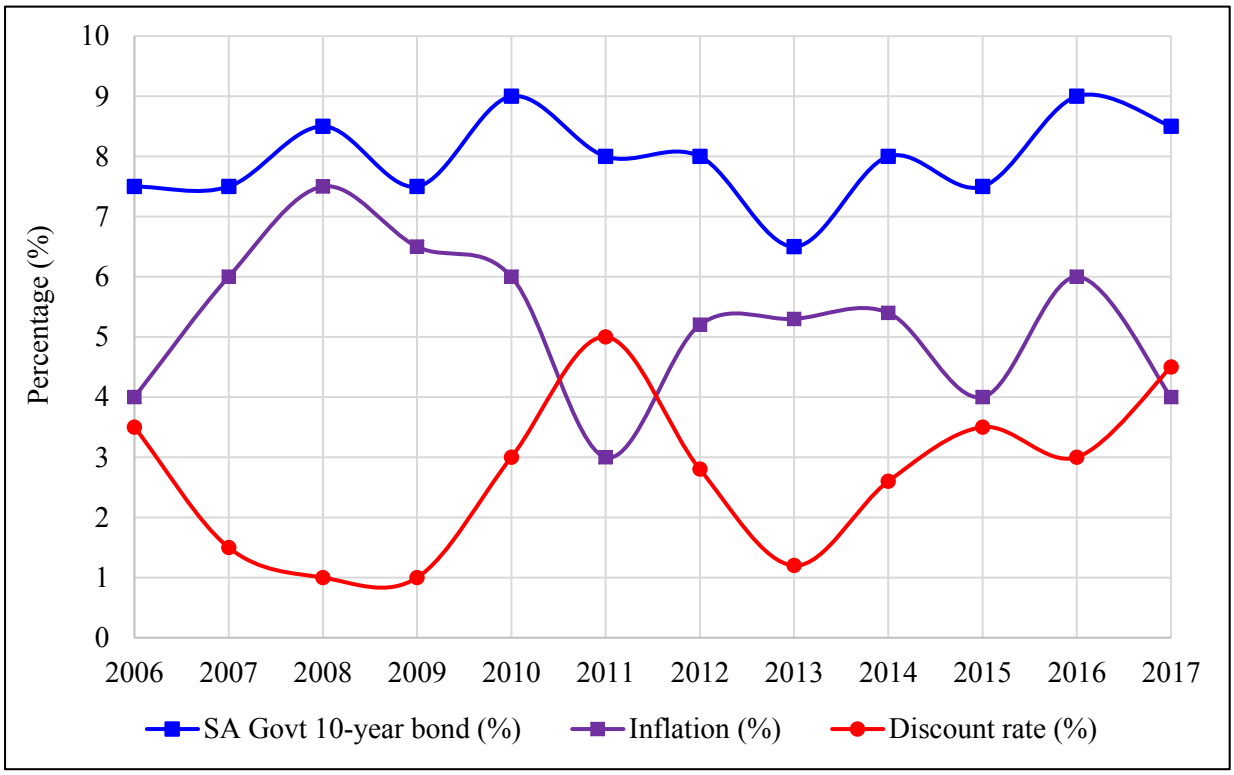


Figure 7-1 Discount rate over a 10-year period (*after* StatSA, 2018)

Table 7-1 Design period and typical life expectancies in years (*after* Mackenzie, 2010)

Type of facility	Characteristics	Design period	Life expectancy
Large dams and pipelines	Difficult and expensive to replace	100	100+
Wells	Easy to refurbish/replace	15–25	25+
Fixed facilities	Expensive to enlarge/replace	20–25	50+
Equipment	Easy to refurbish/replace	10–15	10–20
Distribution systems, *e.g.* dual reticulation system	Replacement is expensive	20–25	60+

7.2 Project components

The various project components were identified from references such as Twort *et al.*, (2000); Begum *et al.*, (2008); McArdle *et al.*, (2011); Gerrity *et al.*, (2014); Blersch & Plessis, (2017); Dillon *et al.*, (2010) USEPA, (2016) and are summarised in Table 7-2.

7.3 Capital costs

Capital cost estimates were derived from the Department of Water Affairs (DWA) costing benchmarks for typical water supply projects (DWA, 2010) and the South African Department of Cooperative Governance and Traditional Affairs (DoCOGTA, 2010). The design of a typical water transmission and reticulation system was undertaken in EPANET 2 integrated with PCSWMM Version 7. The design of the reticulation system was based on Strategy 3 – residential garden irrigation, toilet-flushing and the irrigation of public open spaces discussed in Chapter 6 with an estimated mean annual stormwater yield of 12.5 Mm^3/year. The discount rate (%) and the design period were estimated as discussed in Section 7.1. The resulting estimated capital costs for construction and installation of water supply systems including abstraction, treatment, transmission and distribution are presented in Table 7-3.

Table 7-2 Water resource cost estimation

Process	Component	Surface water		Groundwater	
		Non-potable	Potable	Non-potable	Potable
Abstraction	Intake works	✓	✓	-	-
	Boreholes	-	-	✓	✓
Water Treatment	Primary treatment	✓		-	-
	Conventional	-	✓	-	-
	Final disinfection	-	✓	-	✓
	Desalination	-	-	-	-
	Bio-retention cell	-	-	✓	✓
Reservoirs	Clear water well	✓	✓	✓	✓
	Reservoir	✓	✓	✓	✓
Transmission in decentralised system	Pump	✓	✓	✓	✓
	Pipeline	✓	✓	✓	✓
Dual reticulation	New connections	✓	-	✓	-

Table 7-3 Major capital cost categories for the various water resources

Process	Component	Cost in thousand Rands ('000)			
		Surface water		Groundwater	
		Non-potable	Potable	Non-potable	Potable
Abstraction capacity of 43 ML/day	Intake works (2 No)	15,567	14,567	-	-
	Boreholes (140 No)	-	-	15,763	15,763
Water Treatment	Primary treatment	66,373	-	-	-
	Conventional	-	170,574	-	-
	Final disinfection	-	13,645	-	13,645
	Desalination	-	-	-	-
	Bio-retention cell	-	-	55,737	55,737
Reservoirs	Clear water well - 3200 m^3	4,653	4,653	4,653	4,653
	Supply reservoir in Steel - 1588 m^3	2,534	2,534	2,534	2,534
Transmission in decentralised system	Pump (0.5 m^3/s)	2,958	2,958	2,958	2,958
	16.55 Km DN600 PN 16	28,468	28,468	28,468	28,468
Dual reticulation	New connections	240,040	-	240,040	-
Total (2017 ZAR)		**359,597**	**237,403**	**350,157**	**110,116**

In the option where the partially treated stormwater is conveyed to an existing water treatment plant for blending with the raw water stream from the external reservoirs, the cost of the transmission pipeline was estimated using Equation 7-2 and 7-3 (Bester *et al.*, 2010).

For pipeline $$Cost = L(0.0026\,D^2 + 2.8788\,D - 198)$$ **Equation 7-2**

For pump station $$Cost = 91169\;Q^{0.544}$$ **Equation 7-3**

Where *Cost* is the value in ZAR; L – Total length of the pipeline (m); D – diameter (mm); Q – Total volume of water pumped (L/s); costs are presented in Table 7-4.

Table 7-4 Costs of transmission pipeline and pump stations

Components	Cost in thousand Rands ('000) (2017 ZAR)
30.76 Km DN600 DI PN 25	75 832
3 No. pump stations	8 039
Total	**83 871**

7.4 Operation and maintenance costs

Various approaches for estimation of operation and maintenance costs are possible including: as a percentage of investment costs; or based on past performance of similar utilities as costs per unit volume of water produced (Boshoff *et al.*, 2009). A combination of both these options were adopted in the study, for example, the costs of electricity and chemicals were calculated per m³ of water produced, the estimation of labour costs was based on a mean number of employees per connection and the overheads as a percentage of the total cost.

The energy requirements for water treatment were estimated from studies where energy intensity for water supply components have been compiled (*e.g.* Pabi *et al.*, 2013; Meldrum *et al.* 2013). The estimation of the unit energy costs was based on CCT electricity tariffs for 2017 as shown in Table 7-5.

Table 7-5 Electricity Tariffs 2017 (CCT, 2017)

Large Power User	Time of use	Units	Low voltage (500-1000 kVA)	Medium voltage (>1 MVA)
Service		ZAR/day	98.84	96.9
Energy	High Peak	ZAR/kWh	3.91	3.81
	High Standard	ZAR/kWh	1.38	1.34.
	High- off peak	ZAR/kWh	0.87	0.85
	Low- Peak	ZAR/kWh	1.46	1.42
	Low- Standard	ZAR/kWh	1.09	1.06
	Low- Off Peak	ZAR/kWh	0.79	0.77
Small Power User (<500 kVA)		**Units**	**Small Power User 1 (>1000 kWh/ month)**	**Small Power User 2 (<1000 kWh/ month)**
Service		ZAR/day	52.01	4.1
Energy		ZAR/kWh	1.48	2.60

The estimation of chemical costs was based on the rate of chemical usage and costs as applied at a water treatment plant in Cape Town as shown in Table 7-6. For ethical reasons, the name of the WTP will not be revealed.

Table 7-6 Rate of chemical usage and costs (CCT, 2017)

Chemicals	Actual usage (kg/kL)	Chemical prices (R/kL)
Chlorine	0.00174	27.6
Lime	0.02521	45.1
Aluminium sulphate	0.04917	120
Carbon Dioxide	0.00971	49
PAC	0.00384	95.8

Maintenance typically comprises of both planned preventive and corrective costs and was accounted for by drawing on the researcher's personal experience working with a water utility, consultation with other professional engineers, and various manuals *e.g.* van Zyl, (2014); CPHEEO (2005); and the infrastructure asset management guideline by the RSA Department of Provincial and Local Government, Boshoff *et al.* (2009).

7.5 Total cost analysis with NPV

In the total cost analysis with NPV (including capital, operation and maintenance) associated with the two sources (*i.e.* surface water and groundwater) and two supply requirements (*i.e.* potable and non-potable), the summary of costs in ZAR/kL are presented in Figure 7-2.

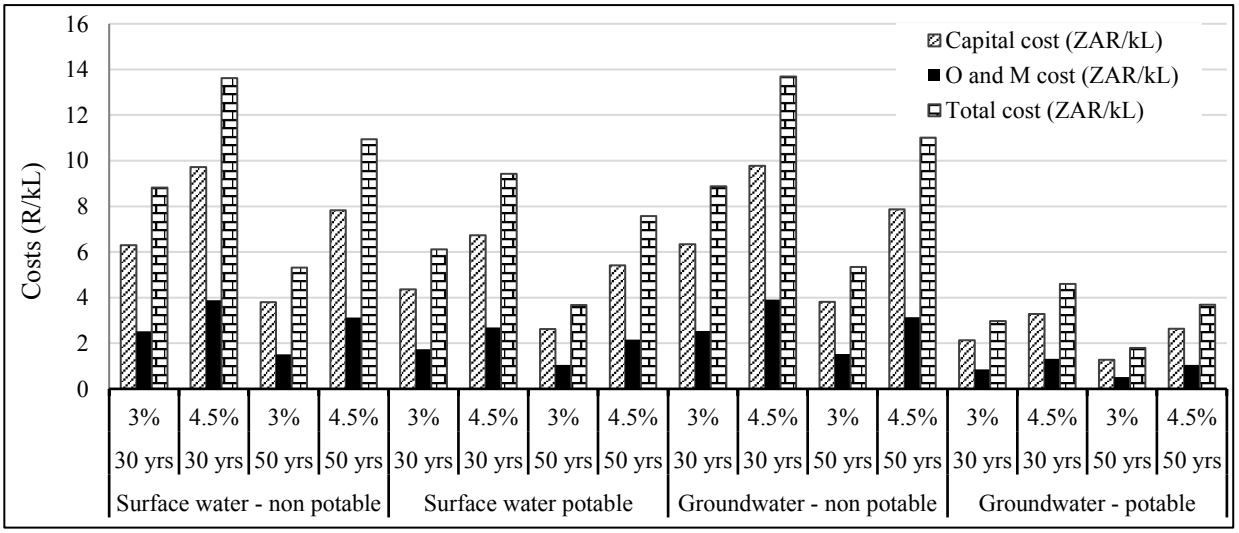


Figure 7-2 Costs for supply from various systems per kL

The results from the economic analysis were compared with indicative unit costs of water from proposed new sources published in the CCT Water Outlook Report of 2018 (CCT, 2018) as presented in Figure 7-3.

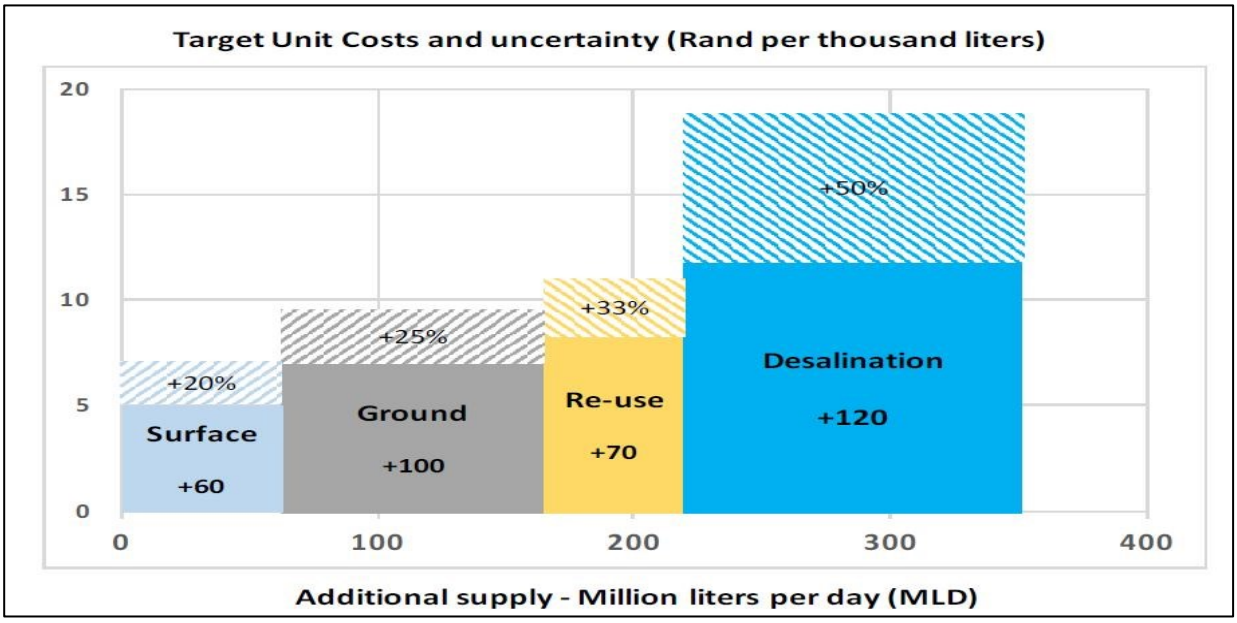


Figure 7-3 Costs of water from proposed systems in Cape Town per kL (CCT, 2018)

The CCT Water Outlook Report of 2018 indicates that the indicative unit costs of water from existing and various proposed new sources as follows: existing reservoirs (R5 – R6/kL); groundwater (R7 – R10/kL); reclaimed water (R8 – R11/kL); and large-scale desalination (R12 – R19/kL) rising to R35/kL for small-scale desalination. From this comparison, it appears that SWH is competitive with these alternatives, with MAR&R combined with potable water demand using the existing CCT water reticulation system being the most cost effective approach.

8. Conclusions and recommendations

This chapter provides a concluding overview of the areas investigated and summary of findings, study contribution, and recommendations. Section 8.1 presents overview of the areas investigated highlighting the proposed stormwater harvesting (SWH) options (*i.e.* from the surface and groundwater storage) and water demands (*i.e.* potable and non-potable). Section 8.2 highlights other benefits including water quality improvement, amenity and biodiversity. Section 8.3 presents the study contribution and Section 8.4 provides recommendations for further studies.

8.1 Overview of investigated areas

In this study, the prospects of catchment-scale SWH was investigated with a focus on available storage in the study area, *i.e.* stormwater ponds and vleis (with storage enhancement using RTC), and groundwater aquifer (with MAR through stormwater ponds modelled as bio-retention cells). The areas investigated include the following:

i. The use of RTC techniques on surface water storage to allow for extended detention of water to provide balancing storage required for SWH. RTC was also essential for flood control through pre-emptive draining before storm events.

ii. Augmentation of groundwater using stormwater where ponds are designed to promote infiltration into the local aquifer. In this case, SWH was from the stormwater ponds.

iii. Assessment of the extent to which stormwater supply could be relied upon to meet selected non-potable water demands in the study area, *e.g.* urban agriculture, residential gardens, public open spaces, and toilet flushing.

iv. The full treatment of stormwater – both locally and at a remote existing WTP was also investigated to determine the opportunity for supply to potable water demand.

v. An economic analysis was also undertaken to determine the viability of SWH for the various options. These could then be compared with other proposed sources in Cape Town *e.g.* existing reservoirs; groundwater; reclaimed water; small-scale and large-scale desalination (CCT, 2018)

8.1.1 Stormwater harvesting from surface water storage

In the assessment of the prospects for SWH from a catchment with seasonal rainfall largely in the winter period, large storage was required to balance the temporal mismatch in the availability of the resource and demand. Large storage was particularly necessary for non-potable water uses, *e.g.* irrigation of agriculture, residential gardens and public open spaces, as the demands were mainly in the dry summer period. To provide the required storage, an investigation was carried out into the use of the available stormwater ponds for both flood control and water supply, using Real-Time-Control (RTC) techniques. The use of RTC on the stormwater ponds provide an

opportunity to utilise the available 1 Mm^3 capacity (about 5.5% of the mean annual modelled volume of stormwater). An assessment was undertaken to determine the reliability and adequacy of the storage to balance the mismatch in availability of stormwater and the three demand options, *i.e.* Sc1 (agriculture), Sc2 (Residential garden irrigation and toilet flushing) and Sc3 (Residential garden irrigation, toilet flushing and irrigation of public open spaces). The storage in the ponds was only able to supply 44%, 60% and 58% of the demands in Sc1, Sc2 and Sc3 respectively. The corresponding spill (water lost as overflow) was 51%, 35% and 37% of the modelled mean annual stormwater volume (*i.e.* 18 Mm^3). To increase yield and reduce spillage, the storage in the vleis was assessed in stepwise incremental volumes of 1 Mm^3 to determine the optimal storage required to account for the mismatch in the availability of stormwater and demand. It was determined that at 4 Mm^3 storage (22% of the modelled mean annual stormwater volume), 70%, 79% and 76% of the non-potable demands in Sc1, Sc2 and Sc3 were met respectively. The corresponding spill (water lost as overflow) was 11%, 7% and 4% of the modelled mean annual stormwater flow (*i.e.* 18 Mm^3). There was minimal increase in demand met and reduction of spillage with capacity of 5 Mm^3. In general, it was determined that stormwater supply to non-potable demand was sensitive to balancing storage and required a capacity of 20 – 30% of stormwater volume to maximise demand met and minimise loss through spillage.

Treatment and use of stormwater for potable water purposes significantly reduced the water lost as overflow since the water could be used virtually immediately. A similar assessment as discussed for non-potable water supply was undertaken for potable water. It was determined that local balancing storage had limited influence to demand met and the optimisation of the SWH system was based on plant capacity to maximise yield and minimise spillage.

Other factors such as land use and climate change would also affect the volume of water to be harvested in the study area in the future. Assessment of the impact of climate change utilising the 26 climate change prediction models available suggest a future annual mean reduction in rainfall of 40 – 200 mm with an increase in mean daily temperature of 3 – 5^{o}C by 2100 compared with the study period (2006 – 2016). Climate change will thus likely result in an annual mean decrease in stormwater yield of 3 – 9 Mm^3 (15 – 50% of the mean annual modelled flow). Land use in the study area is highly variable with built-up areas mainly consisting of residential (formal and informal) and light industrial land-uses. The study area also comprises extensive pervious areas including considerable agricultural land, nature reserves, sports fields and public open spaces. The mean imperviousness for the entire study area was estimated to be 45%. An assessment of the impact of land use change considered both the planned developments and a hypothetical increase in imperviousness. An assessment with a mean imperviousness of 50% (allowing for the planned developments for 2040) and 75% (hypothetical future) showed a significant impact on the availability of the stormwater resource as surface water with an increase of some 29% and 91% respectively. To match the increase in stormwater due to the increase in imperviousness, additional storage would be required to minimise loss through spillage. Since surface storage (stormwater ponds and vleis) is severely limited in the study area – a very common situation in urban areas – an assessment was undertaken on the possibility of utilising

groundwater storage. The study area had considerable aquifer storage with sandy soils that could support surface to groundwater infiltration.

8.1.2 Stormwater harvesting from groundwater storage

The Zeekoe Catchment is particularly well located in an area with suitable features for MAR&R from groundwater storage, *i.e.* availability of large unconfined aquifer that ranges between 20 and 55 m deep, pervious soils (sandy soils) and reasonably flat terrain (catchment slope less than 3%). The catchment also has relatively high typical borehole abstraction rates compared with other areas in Cape Town, *i.e.* in the range of 2 – 5 L/s with some sections greater than 5 L/s (CCT, 2005). Some studies such as Vandoolaeghe, (1989), Fraser *et al*., (2001) and Mauck, (2017) suggest typical borehole abstraction rates in the range of 3 – 12 L/s. The mean annual natural infiltration for the 89 km^2 was estimated to be in the range of 20 – 21 Mm^3. With the 61 stormwater ponds available in the study area adapted to the enhanced surface to groundwater transfer, the mean annual infiltration increased the groundwater resource to 29 – 33 Mm^3. The actual additional groundwater resource due to stormwater infiltration was 9 – 12 Mm^3 (about 30% increase). The impact of land use change was also assessed with a hypothetical future general catchment imperviousness of 75%. It was determined that the natural mean annual infiltration volume would decrease to 10 – 13 Mm^3. Deliberate recharge of aquifer with stormwater to enhance groundwater augmentation would increase the groundwater resource to about 21 Mm^3. The results from modelling various potential groundwater abstraction scenarios in the Zeekoe Catchment show that, depending on the aquifer parameters, *i.e.* conductivity, porosity and aquifer depth; the suitable borehole pumping rates ranged from 3.5 – 8.1 L/s from 140 boreholes in the 89 km^2 Catchment.

8.1.3 Volumetric assessment of using both surface and groundwater storage

Stormwater harvesting with RTC from surface water storage (*i.e.* ponds and vleis) would provide a mean annual volume of 18 Mm^3 with a range of 12 Mm^3 to 25 Mm^3 for the driest (2015) and wettest (2013) years. With the adoption of MAR&R, some stormwater would be locally retained in the catchment through infiltration in the stormwater ponds and transferred to groundwater storage. As a result, the mean annual stormwater flow to the downstream surface storage where SWH is undertaken would reduce from 18 Mm^3 to 12 Mm^3. The stormwater volume would no longer be adequate for the identified non-potable demand options assessed in the study *i.e.* Sc 1, Sc 2 and Sc 3 of 16 Mm^3, 13 Mm^3 and 14 Mm^3 respectively. On the other hand, the enhanced infiltration would augment the groundwater resource from about 20 Mm^3 to 28 – 33 Mm^3. The groundwater resource would be adequate and volumetrically viable for both potable and non-potable demands. At the current level of land use and without adoption of MAR&R, the mean annual surface water volume and natural groundwater infiltration would be about 18 Mm^3 and 20 Mm^3 respectively. Volumetrically, both resources are separately sufficient to supply the identified non-potable demands. Thus the use of both surface water and natural groundwater

resource would be sensible for non-potable demands. However, it was noted that future land use change with increasing imperviousness would likely impact the resources. For example, based on a hypothetical scenario where imperviousness increased to an extreme 75%, the natural groundwater resource would significantly reduce to a mean annual value of 10 Mm^3 while the surface water volume would increase to around 29 Mm^3. This would be a significant change from the historical conditions of the area that was largely marshland and not linked directly to the ocean. The land use change significantly reduces infiltration, thus losing the groundwater resource. The 10 Mm^3 natural groundwater resource would no longer be adequate to supply identified demand, and would thus require enhanced infiltration to augment the groundwater resource. The high runoff from the hardened earth surface would also increase the risk of flooding in the area. Further, the 29 Mm^3 surface water resource is also significantly large and would require equally large storage *e.g.* coastal reservoirs for implementation of an effective SWH system and to minimise loss through spillage. Clearly, facilities such as bio-retention cells where stormwater is deliberately infiltrated to augment the groundwater resource and mitigate floods are required in areas where land use change increases imperviousness. Overall, to maximise benefit from SWH, especially in a catchment such as Zeekoe with physical characteristics appropriate for MAR&R, it would be prudent to utilise groundwater storage. This would restore the area to conditions similar to pre-development conditions, and provide additional benefits including *inter alia* provide a groundwater resource, flood control and stormwater quality improvement.

8.1.4 Use of stormwater as potable or non-potable water

The studies identified in this research (*e.g.* Hatt *et al.*, 2004; Mitchell *et al.*, 2007; Goonrey *et al.*, 2009; Fisher-Jeffes, 2015; Rohrer, 2017) recommended that SWH and reuse be limited to non-potable water purposes. The recommendation was mainly based on the need to minimise potential health risks associated with the poor stormwater quality. Based on monthly grab samples of stormwater, the risks associated with poor water quality were significant in the study area as *E.coli* levels, in some cases, were greater than $1x10^6$ counts/100 mL. Consequently, any attempt to safely and cost-effectively exploit stormwater as a water resource would require appropriate catchment management to reduce the pollutant load; such as treatment of the water to a standard appropriate for the desired use and possibly the construction of a separate reticulation system ('*dual*' or '*third-pipe*' reticulation). The non-potable demands identified in the study, *i.e.* irrigation of urban agriculture, residential gardens and public open spaces were mainly in summer, thus mismatched with availability of the stormwater resource largely available in winter (Figure 3-15). In the yield-demand analysis, the impact from toilet flushing (an indoor water use) with regards to enhancing the performance of volumetric reliability and supply efficiency was negligible. Toilet flushing demand is not sufficient to account for much of the available SW resource and the result is considerable spillage in the rain season. For SWH to be cost-effective (compared to other sources *e.g.* waste water reuse and seawater desalination) and volumetrically (adequate yield to meet demand), supply should be for demands that are

available throughout the year *e.g.* potable water use. Clearly, optimal use of stormwater requires a shift in the use to potable water uses. Treatment to potable standards would also eliminate the potential public health risks from cross connections. It was also determined that the treatment to potable water standards is more cost-effective for SWH at a catchment scale (centralised system) than using the water for non-potable purposes as it eliminates the need for the costly dual reticulation. Accordingly, this study recommends SWH and reuse to be for potable water needs where the abstraction is from a single location and the distribution through the existing potable water system. In the case of SWH from groundwater storage, it was determined that abstraction from boreholes at 400 m from the ponds and travel time of 300 days would allow for a reduction in pollution associated with *E.coli* to values less than 10 counts/100 mL. SWH from groundwater storage could be supplied for non-potable water use without additional treatment. Disinfection would be required for potable water demands.

8.1.5 Water quality improvement

MAR&R provides water quality improvement benefits. The study area contains several informal settlements (slums, shanty towns), that generate wastewater and litter discharges into the drainage channels particularly in the upper reaches of the catchment. The CCT monthly grab samples of stormwater quality showed that the drainage system in the study area is highly impacted by pollution. In various studies (Hunt *et al.*, 2008; Fletcher *et al.*, 2014; Hathaway *et al.*, 2014), bio-retention cells have shown potential for considerable stormwater quality improvement. The selection of bio-retention cells as a potential infiltration device was aimed at benefiting from the water quality improvement. Water quality improvement will result from movement through the sandy aquifer associated with the study area as discussed in Chapter 5. A preliminary assessment suggested that a residence time of about a year should provide die-off of pathogens water to values less than 10 *E.coli* counts/100 mL pond. Other contaminants that are likely to be substantially reduced are nutrients and heavy metals. However, research is still required to determine whether the bio-retention principle can be used in situations like these where the units could be flooded for relatively long periods of time.

8.2 Study Contribution

This study has contributed towards identification of an alternative water resource by considering the possibilities of SWH from surface and groundwater storage for supply to potable and non-potable demands at a catchment scale. The study determined that the optimal use of stormwater requires a shift in the use to potable water uses. Treatment to potable standards would also eliminate the potential public health risks from cross connections. It was also determined that the treatment to potable water standards is more cost-effective for SWH at a catchment scale (centralised system) than using the water for non-potable purposes as it eliminates the need for the costly dual reticulation. Accordingly, this study recommends SWH and reuse to be for potable water needs where the abstraction is from a single location and the distribution through

the existing potable water system. The factors that were determined to be important and needed to be considered for the efficacious application of a SWH system included *inter alia* the availability of storage (surface or groundwater), the catchment characteristics (terrain, soil types, level of development, population density), and seasonal availability of the stormwater resource (winter or all year rainfall). The study also assessed the impact of land use and climate changes on the quantity of the stormwater. Having considered all these factors, this study has found that in the Zeekoe Catchment:

- SWH is a viable water resource volumetrically (sufficient quantity to meet a significant portion of water demand) and economically (cost effective compared to other non-conventional water resources *e.g.* seawater desalination).
- If stormwater from surface water storage is to be used for non-potable uses *e.g.* irrigation of agriculture, residential gardens and public open spaces in areas such as Cape Town with rainfall limited to winter period, storage in the range of 20 – 30% of mean annual modelled stormwater volume would be required to balance the mismatch between availability of the water resource and demand.
- Besides being a supplementary water supply, stormwater from groundwater storage may provide various additional benefits *e.g.* additional flood control (over and above designed capacity in stormwater ponds) and water quality improvement. The additional benefits were not identified with the surface water storage option.
- To maximise benefits from SWH with MAR&R, appropriate physical characteristics *e.g.* flat terrain, pervious soil types and unconfined aquifer need to be present. In the selection of groundwater abstraction rate and distance of boreholes from ponds, the study confirmed that at least one year residence period should be allowed to provide for a reduction in *E.coli* to values less than 10 counts/100 mL.
- The construction and operational costs of the SWH and distribution infrastructure are a major factor in the selection of the system scale (*i.e.* centralised or decentralised) and end-use (potable or non-potable demands). In this study, it was determined that the total cost for a dual reticulation system needed in the case of non-potable water supply made the unit costs (cost/kL) higher than for potable water.
- Based on discussions with CCT officials and several community members during the study, it was determined that SWH and reuse as non-potable water in a highly impacted urban catchment with pollution such as Zeekoe would likely be acceptable in the case where the threat of water scarcity is significant and tariffs associated with alternative options high.
- The 26 climate change prediction models suggest a future annual mean reduction in rainfall of 40 – 200 mm and an increase in mean daily temperature of 3 – 5^{o}C by 2100 compared with the study period (2006 – 2016). The impact of the reduction in rainfall and an increased temperature is likely to be a 15 – 50% reduction in stormwater yield.
- The use of both surface and groundwater storage was affected by land and climate change.

With increase in imperviousness, the natural groundwater resource would significantly reduce requiring MAR to sufficiently supply demands. Groundwater storage seems the most suitable option as it provided additional benefits such as large storage that minimise loss through spillage, flood control and stormwater quality improvement.

Overall, the study has provided insight into opportunities for stormwater use with partial or full treatment for non-potable or potable water demands respectively. The study has also provided a useful understanding of the potential scale and magnitude of the available non-potable water needs. It was also noted that reliability of the SWH was a function of storage capacity especially for supply to non-potable demands and the local rainfall regime. In the long dry periods, the demands supplied by SWH system would rely on the conventional water supply system. This would have economic implications for the water supply system, depending on how charging for stormwater provision and for the conventional system water provision are organised. If charging is not integrated, the operator of the conventional system may face a reduction on income during the rainy season and, at the same time, keeping the responsibility of maintaining all the infrastructure and at least part of the personnel required to providing water during long dry seasons. Furthermore, besides the relief on existing water resources by such an alternative water source, additional benefits, *e.g.* stormwater quality improvement were identified.

8.3 Identified challenges and recommendations for further research

This research mainly focused on the prospects for stormwater harvesting in Cape Town by the identification and assessment of suitable storage for balancing the available stormwater resource and demand to maximise supply and minimise loss. The study was mainly a quantitative assessment of the factors required for the successful implementation of SWH utilising surface and groundwater storage *e.g.* ponds, vleis and aquifer. The scope of the research was limited to the selected catchment and did not consider qualitative factors. The challenges associated with stormwater harvesting identified in literature *i.e.* Section 2.6.6 were also identified in the study. A key challenge for a desktop study on SWH was availability of data for hydrological, hydraulic and water demand modelling. The other challenges related to the catchment characteristics include topography, availability of storage, seasonal rainfall and variability of water demand. The areas recommended for future research are as follows:

- In the case of non-potable water demand, a detailed investigation would be required to determine perception and community acceptance of stormwater as a resource. The respondents and reactions need to be categorised according to the demographics (*e.g.* level of education, income and age group), preferred uses and under what conditions would the resources be accepted or considered (*e.g.* water scarcity and restrictions, high tariffs).
- A comprehensive study of SWH considering all the catchments in CCT to determine the total aggregated volumes available and benefit. Various other storage units would need to

be assessed *e.g.* coastal reservoirs to determine the suitability of installation and benefit.

- There is a need to investigate potential non-potable water demands in industry, commercial and institutions to determine if there are significant needs in rainy seasons that might make it unnecessary to treat the water to potable.
- Whether cost-savings that might be achieved through the joint installation of a dual reticulation system in a green-field development might change the relative economies of potable versus non-potable supply in favour of the latter.
- A qualitative assessment is required to determine the likely level of acceptance of SWH by local residents.
- A pilot study is required to determine the suitability of bio-retention cells and infiltration cells to promote infiltration in the study area to augment the groundwater resource. The study would also propose modifications suitable for a study area.
- Detailed exploration of additional benefits including *inter alia* amenity values such as increasing property values, biodiversity preservation and cooling to minimise 'urban heat' effect.

References

Achleitner, S., Möderl, M., & Rauch, W. (2007). CITY DRAIN ©: An open source approach for simulation of integrated urban drainage systems. Environmental Modelling & Software, Volume 22, Issue 8, August 2007, Pages 1184-1195.

ADB. (2017). Guidelines for the economic analysis of projects. Metro Manila, Philippines. ISBN: 978-92-9257-764-3.

Adelana, S., Xu, Y., & Adams, S. (2006). Identifying Sources and Mechanisms of Groundwater Recharge in the Cape Flats, South Africa: Implications for Sustainable Resources Management (pp. 1–13).

Adelana, S., Xu, Y., Petr, V., Yongxin, X., & Vrbka, P. (2010). A conceptual model for the development and management of the Cape Flats aquifer, South Africa. ISSN 0378-4738 (Print) Water SA Vol. 36 No. 4 July 2010. ISSN 1816-7950 (On-line) Water SA Vol. 36 No. 4 July 2010

Akram, F., Rasul, M. G., Khan, M. M. K., & Amir, M. S. I. I. (2014). A Review on Stormwater Harvesting and Reuse. International Journal of Environmental, Earth Science and Engineering, 8(3), 178–187.

Alcamo, J., Doll, P., Kaspar, F., Siebert, S. (1997). Global change and global scenarios of water use and availability: an application of WaterGAP 1.0. University of Kassel, CESR, Kassel, Germany.

Alcamo, J., Henrichs, T., Rosch, T. (2000). World water in 2025: global modeling and scenario analysis. In: Rijsberman, F.R. (Ed.). World Water Scenarios Analyses. World Water Council, Marseille, France.

AL-Jayyousi., O.R. (2004). Greywater reuse: knowledge management for sustainability. Desalination 167 27-37.

Allen, R. G. (2000). Using the FAO-56 dual crop coefficient method over an irrigated region as part of an evapotranspiration intercomparison study. Journal of Hydrology, 229(1–2), 27–41. doi:10.1016/S0022-1694(99)00194-8. ISBN: 0022-1694. ISSN:00221694.

Allen, R. G., Pereira, L., Raes, D., & Smith, M. (2006). FAO Irrigation and Drainage Paper Crop by. Rome, Italy.

Armitage, N., Fisher-jeffes, L., Carden, K., Winter, K., Naidoo, V., Spiegel, A., Mauck, B., & Coulson, D. (2014). Water Sensitive Urban Design (WSUD) for South Africa. Framework and Guidelines. Water Research Commission Report No TT 588/14. Pretoria.

Armitage, N., Vice, M., Fisher-Jeffes, L., Winter, K., Spiegel, A., & Dunstan, J. (2013). Alternative Technology for Stormwater Management: The South African Guidelines for Sustainable Drainage Systems. Water Research Commission Report No TT 558/13. Pretoria, South Africa. ISBN: 9781431204137.

Bach, P. M., Rauch, W., Mikkelsen, P. S., McCarthy, D. T., & Deletic, A. (2014). A critical review of integrated urban water modelling - Urban drainage and beyond. Environmental Modelling and Software, 54, 88–107. doi:10.1016/j.envsoft.2013.12.018. ISBN: 1364-8152. ISSN:13648152.

Baumann, D.D & Boland, J.J. (1998). The case for managing urban water. In: Baumann DD, Boland JJ and Hanemann WM (eds.) Urban Water Demand Management and Planning. McGraw- Hill Professional, New York

Begum, S., & Rasul, M. G. (2009). Reuse of stormwater for watering gardens and plants using Green Gully: A new stormwater quality improvement device (SQID). Water, Air, and Soil Pollution: Focus, 9(5–6), 371–380. doi:10.1007/s11267-009-9226-x. ISBN: 15677230 (ISSN). ISSN:15677230.

Begum, S., Rasul, M. G., & Brown, R. J. (2008). Stormwater treatment and reuse techniques : A review. 2nd International Conference on Waste Management, Water Pollution, Air Pollution, Indoor Climate, 144–149. ISBN: 9789604740178.

Belgiorno, V., Rizzo, L., Fatta, D., Della, Rocca, C., Lofrano, G., Nikolaou, A., Naddeo, V., Meriç, S. (2007). Review on endocrine disrupting- emerging compounds in urban wastewater: occurrence and re- moval by photocatalysis and ultrasonic irradiation for wastewater reuse. Desalination 215:166–176

Benetti, A. D. (2008) "Water reuse: issues, technologies, and applications," Engenharia Sanitaria e Ambiental, vol. 13, pp. 247-248, 2008.

Bester, A. J., Jacobs, H. E., Merwe, J. Van Der, & Fuamba, M. (2010). Unit cost-functions for value estimation of waterborne sewer infrastructure. Water Institute of South Africa 2010 International Biennial Conference & Exhibition, (October 2014), 11. Retrieved from http://www.ewisa.co.za/literature/files/BesterJacobs WISA article.pdf

Blaney, H. F., & Criddle, W. D. (1962). Determining consumptive use and irrigation water requirements. Technical Bulletin 1275, U.S. Dept. Agriculture, Agricultural Research Service, 59

Blersch, C. L., & Plessis, J. A. Du. (2017). Planning for desalination in the context of the Western Cape water supply system. Journal of the South African Insititution of Civil Engineering, 59(1), 11–21. doi:10.17159/2309-8775/2017/v59n1a2. ISSN:23098775. Retrieved from http://dx.doi.org/10.17159/2309-8775/2017/v59n1a2

Bodenstein, B. L. (2017). Stormwater harvesting and managed abatement for the Royal Cape Golf Course Research project in partial fulfilment of the requirements for a Supervisor :

Borsanyi, P., Benedetti, L., Dirckx, G., De Keyser, W., Muschalla, D., Solvi, A. M., Vanrolleghem, P. A. (2008). Modelling real-time control options on virtual sewer systems. Journal of Environmental Engineering and Science, 7(4), 395-410.

Boshoff, L., Childs, R., & Roberts, L. (2009). Guidelines for Infrastructure Asset Management in Local Government 2006 - 2009. Department of Provincial and Local Government.

Boskidis, I., Pisinaras, V., Petalas, C., and Tsihrintzis, V. (2012). Monitoring and modeling of two alluvial aquifers in lower Nestos river basin, Northern Greece. Journal of Environmental Science and Health. Part A, Toxic/hazardous Substances and Environmental Engineering, 47(12), 1849–68. doi:10.1080/10934529.2012.689552. ISSN:1532-4117.

Bougadis, J., K. Adamowski, and R. Diduch. (2005). Short-term municipal water demand forecasting, Hydrol. Processes, 19, 137–148, doi:10.1002/ hyp.5763.

Breen, P., Coleman, J., Deletic, A., Duncan, H., Fletcher, T., & Gerhart, T. (2006). Integrated Urban Water Management Modelling Review Stage 2 Report. eWater CRC Technical

Report In Snowdon, D., Hardy, M. J., & Rahman, J. M. (2011). Urban Developer : A model architecture for manageably building urban water cycle models spanning multiple scales. In 19th International Congress on Modelling and Simulation. Australia

Brown, T.C. (2004). The Marginal Economic Value of Streamflow from National Forests. Rocky Mountain Research Station, U.S. Forest Service, December 2004.

Brown, C., & Dallas, H. (1995). Eerste River, Western Cape: Situation assessment of the riverine ecosystem. Final report, June 1995,. Stellenbosch.

Brown, C., & Magoba, R. (2009). Rivers and Wetlands of Cape Town - caring for our rich aquatic heritage. Water Research Commission Report No TT 376/08. Pretoria. ISBN: 978-1-77005

Brown, R., Hunt, W.F., & Kennedy, S.G. (2011). Designing Bioretention with an Internal Water Storage (IWS) Layer. Design guidance for an innovative bioretention feature.

Brown, R., Keath, N., & Wong, T. (2008). Transitioning to Water Sensitive Cities: Historical, current and future transition states. In 11th International Conference on Urban Drainage, Edinburgh, UK, 31 August- 5 September 2008. Edinburgh, UK.

Brunner, P., Simmons, C. T., and Cook, P. G. (2009). Spatial and temporal aspects of the transition from connection to disconnection between rivers, lakes and groundwater. Journal of Hydrology, 376(1–2), 159–169.

Buchmiller, R., M. Focazio, L. Franke, W. Freeman, S, Gain, E. Josberger, and C. Tate. (2000). A Vision for the U.S. Geological Survey National Water-Use Information Program. Unpublished report prepared by the internal USGS planning team on future of the National Water-Use Assessment Program. U.S. Geological Survey: Reston, VA.

Bugan, R. D. H., Jovanovic, N., Israel, S., Tredoux, G., Genthe, B., Steyn, M., Allpass, D., Bishop, R., & Marinus, V. (2016). Four decades of water recycling in Atlantis (Western Cape, South Africa): Past, present and future. Water SA, 42(4), 577–594. ISSN:03784738.

Campisano, A., Modica, C. (2002). Modern technologies for real time control of wastewater systems, Proceedings of 16th European Junior Scientist Workshop 'Real Time Control and Measurement in Urban Drainage Systems', Valle dei Margi, Grammichele, Catania, Italy, 7–10 November 2002

Campisano, A., & Modica, C. (2014). Selecting time scale resolution to evaluate water saving and retention potential of rainwater harvesting tanks. In Procedia Engineering (Vol. 70, pp. 218–227). doi:10.1016/j.proeng.2014.02.025. ISSN:18777058.

Candela L, Fabregat S, Josa A, Suriol J, Vigués N, Mas J. (2007) Assessment of soil a groundwater impacts by treated urban waste- water reuse. A case study: application in a golf course (Girona, Spain). Sci Total Environ 374:26–35

CCT. (2005). State of Rivers Report - Greater Cape Townis Rivers. Cape Town. ISBN: 0620340266.

CCT. (2007). Water Services Development Plan for the City of Cape Town 2007/8 - 2011/2012. City of Cape Town, South Africa.

CCT. (2012). City of Cape Town GIS Database: General Land use Layer Shapefile. Strategic Information Department. City of Cape Town, South Africa. Cape Town, South Africa.

CCT. (2012). City space planning - Spatial development framework. Cape Town.

CCT. (2015). City of Cape Town GIS Database: Rainfall and Flow data monitoring points. Strategic Information Department. City of Cape Town, South Africa. Cape Town, South Africa.

CCT. (2015). City of Cape Town GIS Database: Water Quality data and monitoring points. Strategic Information Department. City of Cape Town, South Africa. Cape Town, South Africa.

CCT. (2017). Annexure 4 Revised Consumptive Tariffs , Rates and Basic Charges for Electricity Services , Water Services and Waste.

CCT. (2018). Water Outlook 2018 Report. Cape Town, South Africa. Retrieved from http://resource.capetown.gov.za/documentcentre/Documents/City research reports and review/Water Outlook 2018 - Summary.pdf

CHI. (2014). SWMM modeling with PCSWMM - Advanced modeling software for stormwater management, wastewater and watershed modeling.

Christou, A., Maratheftis, G., Elia, M., Hapeshi, E., Michael, C., & Fatta-Kassinos, D. (2016). Effects of wastewater applied with discrete irrigation techniques on strawberry plants' productivity and the safety, quality characteristics and antioxidant capacity of fruits. Journal of Mathematical Analysis and Applications, 441(2), 48–54. doi:10.1016/j.agwat.2016.04.027. ISBN: 0378-3774. ISSN:10960813.

Chiu, Y. R., Liaw, C. H., & Chang, H. H. (2008). GIS-based decision supporting system in hydraulic simulation and economic analysis for rainwater harvesting design. In Proceedings of the 2008 12th International Conference on Computer Supported Cooperative Work in Design, CSCWD (Vol. 2, pp. 1133–1138). Ieee. doi:10.1109/CSCWD.2008.4537139. ISBN: 9781424416509.

Choisnel, E., de Villele, O., Lacroze, F. (1992). A standardized approach for calculation of potential evapotranspiration for all countries of the European Community. European, Commission. Luxembourg, 176 pp

Clary, J., Jones, J., Urbonas, B., Quigley, M., Strecker, E., & Wagner, T. (2008). Can Stormwater BMPs Remove Bacteria? New Findings from the International Stormwater BMP Database. Stormwater Magazine May, (June), 1–14.

Clift, M., & Bourke, K. (1999). Study on Whole Life Cycle costing. Building Research Establishment (BRE LTD). In: Lampe, L., Barrett, M., Woods-Ballard, B., Andrews, A., Martin, P., & Kellagher, R. (2005). Performance and Whole Life Costs of Best Management Practices and Sustainable Urban Drainage Systems. Water Environment Research Foundation Project 01-CTS-21T

Coccia, G., Todini, E. (2011). Recent developments in predictive uncertainty assessment based on the model conditional processor approach. Hydrol. Earth Syst. Sci. 15, 3253–3274

Colas, H., Pleau, M., Lamarre, J., Pelletier, G., & Lavallee, P. (2004). Practical perspective on Real-Time Control. Water Quality Research Journal of Canada, 39(4), 466–478. ISBN: 1201- 3080.

Coombes, P., & Mitchell, G. (2006). Urban Water Harvesting and Reuse. In Australian Runoff Quality: A Guide to Water Sensitive Urban Design. Engineers Australia. doi:http://hdl.handle.net/1959.13/32154. ISBN: 0858258528.

Cosgrove, W.J., & Rijsberman, F.R., (2000b). Challenge for the 21st century: making water everybody's business. Sustain. Develop. Int. 2, 149–156.

CPHEEO. (2005). Operation and Maintenance of Water Supply Systems Central Public Health and Environmental Engineering Organisation World Health Organisation, January, 1–31.

Crawford, N.H. & Linsley, R.K. (1966). Digital Simulation in Hydrology: Stanford Watershed Model IV, Tech Report No. 39, Civil Engineering Department, Stanford University, Palo Alto, CA, July 1966.

CSIR. (2005). Guidelines for Human Settlement and Planning: Volume I (Vol. 2). Boutek Report No. BOU/E2001. CSIR Building and Construction Technology. Pretoria, South Africa. ISBN: 0798854987.

Darshdeep, J., & Litoria, S. (2009). Selection of suitable sites for water harvesting structures in soankhad watershed, punjab using remote sensing and geographical information system (RS & GIS) approach- a case study. Journal of the Indian Society of Remote Sensing, 37(1), 21–35.

Davis, A., Shokouhian, M., Sharma, H., Minami, C., & Winogradoff, D. (2003). Water quality improvement through bioretention: Lead, Copper and Zinc removal. Water Environment Research, 75(1), 73–82. doi:10.2175/106143005X94376.

Dawson, C. W., Abrahart, R. J., & See, L. M. (2007). HydroTest: A web-based toolbox of evaluation metrics for the standardised assessment of hydrological forecasts. Environ-mental Modelling and Software, 22(7), 1034–1052. doi:10.1016/j.envsoft. 2006.06.008. ISBN: 13648152 (ISSN). ISSN:13648152.

DCD. (1983). Guidelines for the Provision of Engineering Services for Residential Townships. The Blue Book. Department of Community Development. Pretoria, South Africa.

Debo T.N & Reese A.J (2003). Municipal Stormwater Management. Lewis Publishers. Florida

DECNSW. (2006). Managing Urban Stormwater: Harvesting and Reuse. Department of Environment and Conservation NSW. Sydney South, Australia. ISBN: 1741378753.

DEAT. (2002). Integrated Environmental Management Information series: Screening (Vol. 7). ISBN: 0958472890. ISSN:1537-6613.

Department of Environmental Affairs. (2012). National Norms And Standards For The Remediation Of Contaminated Land and Soil Quality, 561 Government Gazette 16 (2012). doi:http://dx.doi.org/9771682584003-32963. ISBN: 2008018466.

Delleur, J.W (2007). The Handbook of Groundwater Engineering. CRC Press, Taylor & Francis Group, Boca Raton London New York. Second Edition

De Wit, M., van Zyl, H., Crookes, D., Blignaught, J., Jayiya, T., Goiset, V., & Mahumani, B. (2009). Investing in Natural Assets A business case for the environment in the City of Cape Town. City of Cape Town. Retrieved August 25, 2015, from https://www.capetown.gov.za/en/EnvironmentalResourceManagement/publications/Pages/Reportsand.aspx#envresecon.

DHI Denmark. (2014). MIKESWMM by DHI. Retrieved January 20, 2015, from http://worldwide.dhigroup.com/dk/?redirect=www.dhi.dk

Dillon, P., Toze, S., Page, D., Vanderzalm, J., Bekele, E., Sidhu, J., & Rinck-Pfeiffer, S. (2010).

Managed aquifer recharge: Rediscovering nature as a leading edge technology. Water Science and Technology, 62(10), 2338–2345. doi:10.2166/wst.2010.444. ISBN: 0273-1223. ISSN:02731223.

Dinovo, E. (1995). Stormwater detention basins and residential locational decisions. Water Resources Bulletin.

Dixon, A., Butler, D & Fewkes, A. (1999). Water saving poten- tial of domestic water reuse systems using greywater and rainwater combination. Water Sci. Technol. 39 25-32.

Dobbie, M., Brookes, K., & Brown, R. (2012). Risk perceptions of Australian urban water practitioners towards alternative water systems, including stormwater harvesting and quality treatment systems. Retrieved May 29, 2016, from http://www.waterfor liveability.org.au/wp-content/uploads/Project6-SocietyInstitutions- Apr20121.pdf.

Dobes, L., & Bennett, J. (2009). Multi-Criteria Analysis: "Good enough" for government work? Agenda: A Journal of Policy Analysis and Reform, 16(3), 7–26. ISSN:13221833.

DoCOGTA. (2010). An Industry Guide to Infrastructure Service Delivery Levels and Units Cost. Pretoria, South Africa.

Dodgson, J. S., Spackman, M., Pearman, A., & Phillips, L. D. (2009). Multi-criteria analysis : a manual. Appraisal (Vol. 11). doi:10.1002/mcda.399. ISBN: 9781409810230. ISSN: 10579214.

Dolan, P., & Edlin, R. (2002). Is it really possible to build a bridge between cost-benefit analysis and cost-effectiveness analysis? Journal of Health Economics, 21, 827–843. doi:10.1016/S0167-6296(02)00011-5. ISBN: 0167-6296 (Print). ISSN:01676296.

Doll. (2017). Probable extent and pathway of groundwater and pollution from surface water. Master Thesis in Environmental Engineering ETH Zurich

Dotto, C. B. S., Bach, P. M., Allen, R., Wong, T., & Deletic, A. (2014). Towards Water Sensitive Urban Precincts: Modelling Stormwater Management Opportunities. In 13th International Conference on Urban Drainage (pp. 1–9). Sarawak, Malaysia.

Duchesne, S., Mailhot, A., Dequidt, E., & Villeneuve, J. P. (2001). Mathematical modeling of sewers under surcharge for real time control of combined sewer overflows. Urban Water, 3(4), 241–252. doi:10.1016/S1462-0758(01)00037-1. ISBN: 1462-0758. ISSN:14620758.

Duchesne, S., Mailhot, A., & Villeneuve, J.-P. (2004). Global Predictive Real-Time Control of Sewers Allowing Surcharged Flows. Journal of Environmental Engineering, 130(5), 526–534. doi:10.1061/(ASCE)0733-9372(2004)130:5(526). ISBN: 0733-9372. ISSN:0733-9372.

DWA. (1996). South African Water Quality Guidelines. Volume 7: Aquatic ecosystems. Aquatic Ecosystems (Vol. 7). ISBN: 0798853387.

DWA. (2004). Overview of the South African Water Sector. National Water Resource Strategy, 1, 1–35.

DWA. (2008). The assessment of water availabilty in the Berg Catchment (WMA19) by means of Water Resources Related Models (Vol. 5).

DWA. (2010). The Atlantis Water Resource Management Scheme: 30 years of Artificial Groundwater Recharge. Department of Water Affairs and Forestry. Pretoria, South Africa.

DWA. (2011). NWRS2 Annexure D National Strategy for Water Re-use.

DWA. (2012). Pre-feasibility and feasibility studies for augmentation of the western cape water supply system by means of further surface water developments. Cape Town, South Africa.

DWS. (2014). WC WSS Reconciliation Strategy Status Report October 2013. Cape Town, South Africa. Retrieved from https://www.dwa.gov.za/Projects/RS_WC_WSS/ Docs/WCWSS Status Report Oct2014 Final.pdf

DWS. (2015). Department of Water and Sanitation - Hydrological Services - surface water data monitoring. Pretoria, South Africa.

El Saliby, I., Okour, Y., Shon, H. K., Kandasamy, J., & Kim, I. S. (2009). Desalination plants in Australia, review and facts. Desalination, Elsevier B.V, 247(1–3), 1–14. doi:10.1016/j.desal.2008.12.007. ISSN:00119164.

Elliott, A., & Trowsdale, S. (2007). A review of models for low impact urban stormwater drainage. Environmental Modelling & Software, 22(3), 394–405. doi:10.1016/j.envsoft. 2005.12.005. ISSN:13648152.

Engelbrecht, J.F.P. 1998. Groundwater pollution from cemeteries. Proc. Biennial Conference, 4–7 May. Cape Town: The Waste Institute of Southern Africa.

eWater. (2013). Australia's national hydrological modelling platform: Integrated Water Resources Management (IWRM) combined with water policy and governance capability. eWater Source.

Fagan, J. E., Reuter, M. A., & Langford, K. J. (2010). Dynamic performance metrics to assess sustainability and cost effectiveness of integrated urban water systems. Resources, Conservation and Recycling, 54(10), 719–736. doi:10.1016/j.resconrec.2009.12.002. ISBN: 0921-3449. ISSN:09213449.

FAO. (2012). ETo Calculator. Land and Water Digital Media Series No 36. FAO, Rome, Italy.

Fewkes, A., & Butler, D. (2000). Simulating the performance of rainwater collection and reuse systems using behavioural models. Building Services Engineering Research and Technology, 21(2), 99–106. doi:10.1177/014362440002100204. ISBN: 0143-6244. ISSN:0143-6244.

Field. R, & Sullivan, D., (2003) Wet-Weather Flow in the Urban Watershed, Lewis Publishers, Boca Raton.

Fisher-Jeffes, L. (2015). The viability of rainwater and stormwater harvesting in the residential areas of the Liesbeek River Catchment, Cape Town. University of Cape Town.

Fisher-Jeffes, L., Carden, K., Armitage, N. ., & Winter, K. (2017). Stormwater harvesting: Improving water security in South Africa's urban areas. South African Journal of Science, 113(1–2), 2–5. doi:10.17159/sajs.2017/20160153. ISSN:19967489.

Fletcher, T., Deletic, A. B., & Hatt, B. E. (2004). A Review of Stormwater Sensitive Urban Design in Australia.

Fletcher, T. D., Andrieu, H., & Hamel, P. (2013). Understanding, management and modelling of urban hydrology and its consequences for receiving waters: A state of the art. Advances in Water Resources, 51, 261–279. doi:10.1016/j.advwatres.2012.09.001. ISBN: 0309- 1708. ISSN:03091708.

Fletcher, T. D., Deletic, A., Mitchell, V. G., & Hatt, B. E. (2008). Reuse of urban runoff in australia: A review of recent advances and remaining challenges. Journal of Environment Quality, 37(5 Suppl), S–116. doi:10.2134/jeq2007.0411. ISSN:0047-2425.

Fletcher, T., Shuster, W., Hunt, W. F., Ashley, R., Butler, D., Arthur, S., Trowsdale, S., Barraud, S., Semadeni-Davies, A., Bertrand-Krajewski, J.-L., Mikkelsen, P. S., Rivard, G., Uhl, M., Dagenais, D., & Viklander, M. (2014). SUDS, LID, BMPs, WSUD and more – The evolution and application of terminology surrounding urban drainage. Urban Water Journal, (September), 1–18. doi:10.1080/1573062X.2014.916314. ISSN:1573-062X.

Foster, S., Hirata, R., Gomes, D., D'Elia, M., & Paris, M. (2002). Groundwater quality protection: a guide for Water Utilities, Municipal Authorities and Environmental Agencies. World Bank Publication, Washington, D.C., USA. doi:10.2134/jeq1988. 0047242500170002038x. ISBN: 0821349511.

Fraser, L., & Weaver, J. (2000). Groundwater impact scoping for the Cape Flats Aquifer. Pretoria, South Africa.

Friedler E, Lahav E, Jizhaki H & Lahav T. (2006). Study of urban population attitudes towards various wastewater reuse options: Israel as a case study. J. Environ. Manage. 81 360-370.

Gaborit, E., Muschalla, D., Vallet, B., Vanrolleghem, P. A., & Anctil, F. (2013). Improving the performance of stormwater detention basins by real-time control using rainfall forecasts. Urban Water Journal, 10(4), 230–246. doi:10.1080/1573062X.2012.726229. ISBN: 1573-062X. ISSN:1573-062X.

Galán, J. M., A. López-Paredes, & R. del Olmo (2009), An agent-based model for domestic water management in Valladolid metropolitan area, Water Resour. Res., 45, W05401, doi:10.1029/2007WR006536.

Garcia, L., Barreiro-Gomez, J., Escobar, E., Tellez, D., Quijano, N., & Ocampo-Martinez, C. (2015). Modeling and real-time control of urban drainage systems: A review. Advances in Water Resources, 85(18), 120–132. doi:10.1016/j.advwatres.2015.08.007. ISSN:0309-1708.

García, L., Escobar, E., Tellez, D., Gómez, J., & Silva, N. (2014). On The Modeling And Real-Time Control Of Urban Drainage Systems : A Survey. 11th International Conference on Hydroinformatics.

Geber, A. (1981). A digital model of groundwater flow in the Cape Flats. Pretoria, South Africa.

Gerrity, D., Owens-Bennett, E., Venezia, T., Stanford, B. D., Plumlee, M. H., Debroux, J., & Trussell, R. S. (2014). Applicability of Ozone and Biological Activated Carbon for Potable Reuse. Ozone: Science and Engineering, 36(2), 123–137. doi:10.1080/01919512. 2013.866886. ISSN:15476545.

Gleick, P. H. (2000). A look at twenty-first century water resources development. Water International, 25(1), 127–138. doi:10.1080/02508060008686804. ISBN: 0250-8060. ISSN:02508060.

Goonrey, C. (2005). Use of stormwater as an alternative supply source. M.Eng Thesis. Victoria University. Melbourne, Australia.

Goonrey, C. M., Perera, B. J. C., Lechte, P., Maheepala, S., & Mitchell, V. . (2009). A technical decision-making framework: Stormwater as an alternative supply source. Urban Water

Journal, 6(6), 417–429. doi:10.1080/15730620903089787. ISBN: 1573-062X. ISSN:1573062X.

Grassi, M., Rizzo, L., & Farina, A. (2013). Endocrine disruptors compounds, pharmaceuticals and personal care products in urban wastewater: Implications for agricultural reuse and their removal by adsorption process. Environmental Science and Pollution Research, 20(6), 3616–3628. doi:10.1007/s11356-013-1636-7. ISBN: 0944-1344. ISSN:09441344.

GreenCape. (2017). Market Intelligence Report: Water. Cape Town.

Green, W.H & Ampt, G.A (1911). Studies on soil physics. Journal of Agricultural Science.

Grobicki, A. (2001). Integrated catchment management in an urban context. The Great and Little Lotus Rivers, Cape Town. Pretoria, South Africa.

Gucinski, H., Lackey, R. T., & Spence, B. C. (1992). Global Climate Change and Freshwater Ecosystems. Global Climate Change. New York.

Haddock, D. (2004). When are Environmental Amenities Policy Relevant? Natural Resources Journal.

Hamdan, S. M. (2009). A literature based study of stormwater harvesting as a new water resource. Water Science and Technology, 60(5), 1327–1339. doi:10.2166/wst.2009.396. ISSN:02731223.

Hansen, A., & Dale, V. (2001). Biodiversity in US forests under global climate change. Ecosystems. doi:10.1007/s10021-001-0001-8. ISBN: 1432-9840. ISSN:1432-9840.

Henzen, M. 1973. The reclamation, storage and abstraction of purified sewage effluents in the Cape Peninsula (in Afrikaans). D.Sc. Thesis, University of the Orange Free State, Bloemfontein

Hargreaves, G. H., & Allen, R. G. (2003). History and Evaluation of Hargreaves Evapotranspiration Equation. Journal of Irrigation and Drainage Engineering, 129(1), 53–63. doi:10.1061/(ASCE)0733-9437(2003)129:1(53). ISBN: 0733-9437. ISSN:0733-9437.

Haskins, C. (2012). Cape Town's sustainable approach to urban stormwater management. City of Cape Town.

Haskins, C. (2014). False Bay Ecology Park Water Quality Report. Cape Town, South Africa

Hathaway, J. M., Brown, R. A., Fu, J. S., & Hunt, W. F. (2014). Climate Change Impacts on Bioretention Performance in North, (September), 1–6.

Hathaway, J. M., Hunt, W. F., Wright, J. D., & Jadlocki, S. (2008). An evaluation of pathogen removal in stormwater best management practices in charlotte and wilmington, North Carolina, 7, 4472–4479. ISBN: 9781605605364 (ISBN).

Hatt, B. E., Deletic, A., & Fletcher, T. (2004). Integrated treatment and recycling of stormwater: A review of Australian practice. Journal of Environmental Management, 79(1), 102–113. doi:10.1016/j.jenvman.2005.06.003. ISBN: 0301-4797. ISSN:03014797.

Hatt, B., Deletic, A., & Fletcher, T. (2006). Integrated treatment and recycling of stormwater: a review of Australian practice. Journal of Environmental Management, 79(1), 102–13. doi:10.1016/j.jenvman.2005.06.003. ISSN:0301-4797.

Hay, R., Mcgibbon, D., Botha, F., Riemann, K., Town, C., Town, C., Town, C., & Town, C.

(2015). Cape Flats Aquifer and False Bay – Opportunities To Change. In 79th annual IMESA Conference, Cape Town, South Africa. 27th - 30th October 2015. Cape Town.

Heinrich, M. (2006). Residential Water End Use Literature Survey. Branz Ltd. Judgeford, New Zealand. ISSN:01133675.

Henzen, M. (1973). The reclamation, storage and abstraction of purified sewage effluents in the Cape Peninsula (in Afrikaans). University of the Orange Free State, Bloemfontein.

Herrmann, T., & Schmida, U. (2000). Rainwater utilisation in Germany : efficiency, dimensioning, hydraulic and environmental aspects. Urban Water, 1(4), 307–316. ISSN:14620758.

Hewitson, B., Tadross, M., & Jack, C. (2005). Scenarios from the University of Cape Town (WRC 1430/1/05). In Climate Change and Water Resources in Southern Africa: Studies on Scenarios, Impacts, Vulnerabilities and Adaptation (pp. 39–56). Water Research Commission (South Africa). Pretoria.

Hewitson, B. ., & Crane, R. . (2006). Consensus between GCM climate change projections with empirical downscaling: precipitation downscaling over South Africa. Int. J. Clim, (26), 1315–1337.

Hoban, A., Mills, K., Tanner, C., & Hamlyn-Harris, D. (2015). Climate Change Impacts On Stormwater Harvesting Yields: Evaluating the impacts of climate change on urban water systems through simulation of a range of climate change scenarios. Water.

Hoban, A., & Wong, T. (2006). "WSUD resilience to Climate Change." In 1st international Hydropolis Conference, Perth WA, October 2006.

Horton, R. . (1933). The role of infiltration in the hydrologic cycle. Trans. Am. GeoPhYs. Union, 14(1), 446–460.

Huang, Y., Hashim, Z., & Shaaban, A. (2009). Potential and effectiveness of rainwater harvesting systems in flash floods reduction in Kuala Lumpur City, Malaysia. In The 8th International Conference on Urban Drainage Modelling and The 2nd International Conference on Rainwater Harvesting and Management. Tokyo, Japan

Hunt, W. F., Smith, J. T., Jadlocki, S. J., Hathaway, J. M., & Eubanks, P. R. (2008). Pollutant Removal and Peak Flow Mitigation by a Bioretention Cell in Urban Charlotte, N.C. Journal of Environmental Engineering, 134(5), 403–408. doi:10.1061/(ASCE)0733-9372(2008)134:5(403). ISBN: 0733-9372. ISSN:0733-9372.

Hunter Water Corporation 2003 Estimating Guidelines for Water Supply

Hutchins, M. G., McGrane, S. J., Miller, J. D., Hagen-Zanker, A., Kjeldsen, T. R., Dadson, S. J., & Rowland, C. S. (2017). Integrated modeling in urban hydrology: reviewing the role of monitoring technology in overcoming the issue of 'big data' requirements. Wiley Interdisciplinary Reviews: Water, 4(1), e1177. doi:10.1002/wat2.1177. ISSN:20491948. Retrieved from http://doi.wiley.com/10.1002/wat2.1177

Ibrahim, S., Kiran, A., Rashid, M., Yamin, S., Zarlish, A., Munir, S., & Tehreem, R. (2015). The efficacy of chlorinated water used for irrigation purpose on plant initial growth, 4(4), 17–19.

Ilemobade, A., Adewumi, J. R., & Van Zyl, J. E. (2009). Framework for assessing the viability

of implementing dual water reticulation systems in South Africa. Water SA, 35(2), 216–227. ISBN: 0378-4738. ISSN:03784738.

Inamdar, P. M., Cook, S., Sharma, A. K., Corby, N., O'Connor, J., & Perera, B. J. C. (2013). A GIS based screening tool for locating and ranking of suitable stormwater harvesting sites in urban areas. Journal of Environmental Management, 128, 363–370. doi:10.1016/j.jenvman .2013.05.023. ISBN: 0301-4797. ISSN:03014797.

Innovyze. (2011). Info Works Collection Systems (CS) - Technical Review.

IPCC. (2014). A report of the Inter-Governmental Panel on Climate Change. Climate Change 2014: Synthesis Report. Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change. doi:10.1017/CBO 9781107415324. ISBN: 9789291691432. ISSN:1476-4687.

International Water Management Institute. (IWMI) (2000). November research update, Colombo, Sri Lanka, 5.

Jensen, M. E., Burman, R. D., and Allen, R. G. (1990). Evapotranspi- ration and irrigation water requirements. ASCE Manuals and Reports on Engineering Practice, No 70, 360.

Jacobs, H., & Haarhoff, J. (2004). Structure and data requirements of an end-use model for residential water demand and return flow. Water SA, 30(3), 293–304. doi:10.4314/wsa.v30i3.5077. ISBN: 0378-4738.

James, W. (2005). Rules for Responsible Modeling (4th ed.). Geulph, Ontario. ISBN: 0-9683681-5-8.

James, W., Rossman, L. E., & James, R. C. (2010). User guide to SWMM5 (13th Editi.). CHI, Guelph, Ontario, Canada. ISBN: 9780980885330.

Jimenez B and Asano T (2008) Water Reuse. An International Sur- vey of Current Practice, Issues and Needs. Technical report No. 20. International Water Association (IWA). ISBN 1843390892.

Jovanovic, N., Bugan, R. D. H., Tredoux, G., Israel, S., Bishop, R., & Marinus, V. (2017). Hydrogeological modelling of the atlantis aquifer for management support to the atlantis water supply scheme. Water SA, 43(1), 122–138. doi:10.4314/wsa.v43i1.15. ISSN:18167950.

Junying C, Jining C, Can W and Ping F (2004) Wastewater reuse potential analysis: implications for China water resources manage- ment. Water Res. 38 2746-2756.

Kalbacher, T., Delfs, J.-O., Shao, H., Wang, W., Walther, M., Samaniego, L., Schneider, C., Kumar, R., Musolff, A., Centler, F., Sun, F., Hildebrandt, A., Liedl, R., Borchardt, D., Krebs, P., and Kolditz, O. (2012). The IWAS-ToolBox: Software coupling for an integrated water resources management. Environmental Earth Sciences, 65(5), 1367–1380. doi:10.1007/s12665- 011-1270-y.ISSN:1866-6280.

Kim, J., & Furumai, H. (2012). Assessment of rainwater availability by building type and water use through GIS-based scenario analysis. Water Resources Management, 26(6), 1499–1511. doi:10.1007/s11269-011-9969-9. ISBN: 1126901199. ISSN:09204741.

Kim, M. H., Sung, C. Y., Li, M. H., & Chu, K. H. (2012). Bioretention for stormwater quality improvement in Texas: Removal effectiveness of Escherichia coli. Separation and

Purification Technology, 84, 120–124. doi:10.1016/j.seppur.2011.04.025. ISBN: 1383-5866. ISSN:13835866. Retrieved from http://dx.doi.org/10.1016/j.seppur.2011.04.025

Kimura, K., Amy. G., Drewes. J., Watanabe, Y. (2003). Adsorption of hydrophobic compounds onto NF/RO membranes: an artifact leading to overestimation of rejection. J Membr Sci 221:89–101

Klamerth, N., Rizzo L, Malato S, Maldonado MI, Agüera A, Fernández- Alba AR (2010) Degradation of fifteen emerging contaminants at µg/L initial concentrations by mild solar photo-Fenton in MWTP effluents. Water Res 44:545–554

Klamerth N, Malato S, Maldonado MI, Agüera A, Fernández-Alba A (2011) Modified photo-Fenton for degradation of emerging contaminats in municipal wastewater effluents. Catal Today 161:241–246

Konig, K. (2001). The rainwater technology handbook: rainharvesting in building. WILO- Brain. In: Roebuck, R. (2007). A whole life costing approach for rainwater harvesting - An investigation into the whole life cost developments in the UK. University of Bradford, United Kingdom. Dortmund, Germany. ISBN: 3000083685.

Lampe, L., Andrews, H., Barrett, M., Woods-Ballard, B., Kellagher, R., Martin, P., Jefferies, C., & Hollon, M. (2005). Performance and Whole Life Costs of Best Management Practices and Sustainable Urban Drainage Systems. Water Environment Research Foundation / IWA Publishing. Alexandria, United States of America / London, United Kingdom. ISBN: 1843397439

Lee, J. G., Selvakumar, A., Alvi, K., Riverson, J., Zhen, J. X., Shoemaker, L., & Lai, F. (2012). A watershed-scale design optimization model for stormwater best management practices. Environmental Modelling & Software, 37, 6–18. doi:10.1016/j.envsoft.2012.04.011. ISSN:13648152. Retrieved December 24, 2016 from http://linkinghub.elsevier.com/retrieve/pii/S1364815212001387

Liguori S, Rico-Ramirez MA, Schellart ANA, Saul AJ. (2012) Using probabilistic radar rainfall nowcasts and NWP forecasts for flow prediction in urban catchments. Atmos Res 2012;103:80–95.

Liaw, C., & Tsai, Y. (2004). Optimum Storage Volume Of Rooftop Rain Water Harvesting Systems For Domestic Use 1. Journal of the American Water Resources Association, 40(4), 901–912

Lim, M. H., Leong, Y. H., Tiew, K. N., & Seah, H. (2011). Urban stormwater harvesting: A valuable water resource of Singapore. Water Practice and Technology, 6(4). doi:10.2166/wpt.2011.067. ISBN: 1751-231X. ISSN:1751231X.

Lim, K. Y., Hamilton, A. J., & Jiang, S. C. (2015). Assessment of public health risk associated with viral contamination in harvested urban stormwater for domestic applications. Science of the Total Environment, 523, 95–108. doi:10.1016/j.scitotenv.2015.03.077. ISBN: 0048-9697. ISSN:18791026. Retrieved from http://dx.doi.org/10.1016/j.scitotenv.2015 .03.077

Mackenzie, L.D. (2010). Water and wastewater engineering. Retrieved from http://www.wrc. org.za/Knowledge Hub Documents/Research Reports/TT 278-06

Maheepala, S., Loonat, N., Mirza, F., & Coultas, E. (2011). Quantifying potable water savings of rainwater tanks at a city scale by considering the effect of spatial lumping. In Proceedings

of OzWater 2011 conference.

Mahinthakumar, G. (Kumar), & Sayeed, M. (2006). Reconstructing Groundwater Source Release Histories Using Hybrid Optimization Approaches. Environmental Forensics, 7(1), 45–54. doi:10.1080/15275920500506774. ISSN:1527-5922.

March, J.G., Gual, M., & Orozco F (2004) Experience on grey- water reuse for toilet flushing in a hotel. Desalination 164 241-247

Marsalek, J. E., Watt, E., & Zeman, Sieker, H. (Eds. . (2001). Advances in Urban Stormwater and Agricultural Runoff Source Controls, NATO Earth and Environmental Sciences Series,. Kluwer Academic Publishers. Boston, USA.

Marsden, J., & Pickering, P. (2006). Securing Australia's Urban Water Supplies: Opportunities and Impediments. Victoria.

MATLAB. (2010). version 7.10.0.499 Release 2010, computer software, The MathWorks, Inc., Natick, Massachusetts, United States.

Mauck, B. (2017). The capacity of the cape flats aquifer and its role in water sensitive urban design in Cape Town. University of Cape Town South Africa.

Mavundla, (2018). Infiltration Potential of Stormwater Ponds in the Zeekoe Catchment Area. Department of Civil Engineering University of Cape Town, South Africa

Mays, L. (2001). Water Resources Engineering. John Wiley & Sons. New York. doi:10.1002/ep. 670200306. ISBN: 0-471-29783-6.

McArdle, P., Gleesen, J., Hammond, T., Heslop, E., Holden, R., & Kuczera, G. (2011). Centralised urban stormwater harvesting for potable reuse. Water Science and Technology, 63(1), 16–24.

McMahon, T. A., Adeloye, A. J., & Zhou, S.-L. (2006). Understanding performance measures of reservoirs. Journal of Hydrology, 324(1-4), 359–382. ISBN: 0022-1694. ISSN:00221694.

Meldrum, J., Nettles- Anderson, S., Heath, G., Macknick, J. (2013). Life cycle water use for electricity generation: a review and harmonization of literature estimates, 10.1088/1748-9326/8/1/015031

Mesa-Jurado, M.A., Martin-Ortega, J., Ruto, E., Berbel, J. (2012). The economic value of guaranteed water supply for irrigation under scarcity conditions. Agric. Water Manage. 113, 10–18

Milano, M., Ruelland, D., Fernandez, S., Dezetter, A., Fabre, J., Servat, E. (2012). Facing climatic and anthropogenic changes in the Mediterranean basin: what will be the medium-term impact on water stress? C.R. Geosci. 344, 432–440.

Milly, P. C. D., J. Betancourt, M. Falkenmark,R.M. Hirsch, Z.W. Kundzewicz, D. P. Lettenmaier. & R. J. Stouffer. (2008). Stationarity is dead: Whither water management?, Science, 319,573–574, doi:10.1126/science.1151915

Mitchell, V.G. (2004). Aquacycle User Guide. CRC for Catchment Hydrology. Clayton, Victoria, Australia. Retrieved from www.toolkit.net.au.

Mitchell, G., & Diaper, C. (2005). UVQ: a tool for assessing the water and contaminant balance impacts of urban development scenarios. Water Science and Technology, 52(12), 91–8.

ISSN:0273-1223. Retrieved November 10, 2016 from http://www.ncbi.nlm.nih.gov/pubmed/16477975.

Mitchell, V.G. (2006). Applying integrated urban water management concepts: A review of Australian experience. Environmental Management, 37(5), 589–605. doi:10.1007/s00267 - 004-0252-1. ISBN: 0364-152X. ISSN:0364152X.

Mitchell, V.G., Deletic, A., Fletcher, T., Hatt, B., McCarthy, D., & Hatt, B. (2007). Achieving multiple benefits from stormwater harvesting. In 7th International Conference on Urban Drainage Modelling and the 4th International Conference on Water Sensitive Urban Design; Book of Proceedings. (Vol. 55, pp. 135–144). IWA.

Mitchell, V.G., McCarthy, D. T., Deletic, A., & Fletcher, T. (2007). Urban stormwater harvesting - sensitivity of a storage behaviour model. Environmental Modelling and Software, 23(6), 782–793. doi:10.1016/j.envsoft.2007.09.006. ISBN: 1364-8152. ISSN:13648152.

Mitchell, V.G., Siriwardene, N., Duncan, H., & Rahilly, M. (2008). Investigating the impact of temporal and spatial lumping on rainwater tank system modelling. In M. Lambert, T. Daniell, & M. Leonard (Eds.), Proceedings of Water Down Under 2008. (pp. 54–65). Engineers Australia. ISBN: 0858257351

Mohamed, M. & Al-Mualla, A. (2010). Water demand forecasting in Umm Al-Quwain (UAE) using the IWR-MAIN Specify Forecasting Model. Water Resources Management, 24, 4093-4120.

Moriasi, D. N., Arnold, J. G., Liew, M. W. Van, Bingner, R. L., Harmel, R. D., & Veith, T. L. (2007). Model Evaluation Guidelines for Systematic Quantification of Accuracy in WatershedMoriasi, D.N., Arnold, J.G., Liew, M.W. Van, Bingner, R.L., Harmel, R.D. & Veith, T.L. 2007. Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watersh. American Society of Agricultural and Biological Engineers, St. Joseph, Michigan, 50(3), 885–900. doi:10.13031/2013.23153)

Mukheibir, P. (2008). Water resources management strategies for adaptation to climate- induced impacts in South Africa. Water Resources Management, 22(9), 1259–1276. doi:10.1007/s11269-007-9224-6. ISBN: 0920-4741. ISSN:09204741.

Mwenge Kahinda, J., Sejamoholo, B., Taigbenu, A., Boroto, J., & Lillie, E. (2008). Water Resources Management in Rainwater Harvesting: An Integrated Systems Approach. Water Research Commission (South Africa). South Africa. ISBN: 9781770058163.

Mwenge Kahinda, J., Taigbenu, A. E., & Boroto, R. J. (2010). Domestic rainwater harvesting as an adaptation measure to climate change in South Africa. Physics and Chemistry of the Earth, 35(13-14), 742–751. doi:10.1016/j.pce.2010.07.004. ISBN: 1474-7065. ISSN:1474 7065.

Murray, R., Tredoux, G., Ravenscroft, P., & Botha, F. (2007). The artificial recharge concept, its application and potential. In Artificial Recharge Strategy. Department of Water Affairs and Forestry.

Neumann, L., Coultas, E., Moglia, M., & Mashford, J. (2011). Errors in yield and overflow estimation in rainwater tank cluster modeling. In 12th International Conference on Urban Drainage (p. 8). Porto Alegre, Brazil (10-15 September 2011).

Nakada N, Shinohara H, Murata A, Kiri K, Managaki S, Sato N, Takada H (2007) Removal of

selected pharmaceuticals and personal care products (PPCPs) and endocrine-disrupting chemicals (EDCs) during sand filtration and ozonation at a mu- nicipal sewage treatment plant. Water Res 41:4373–4382

Navarro AE, Cuizano NA, Lazo JC, Sun-kou MR, Llanos BP (2009) Comparative study ofthe removal ofphenolic compounds by biolog- ical and non-biological adsorbents. J Hazard Mater 164:1439–1446

Nghiem LD, Schäfer AI, Waite TD (2002) Adsorption of Estrone on NF and RO membranes in Water and Wastewater Treatment. Water Sci Technol 46(4–5):265–272

Nghiem L, Schaefer A, Elimelech M (2005) Nanofiltration ofhormone mimicking trace organic contaminants. Sep Sci Technol 40:2633– 2649

Nieuwoudt, W., Dockel, J., Mosaka, D. & Pott, A. (2008). Towards the Establishment of Water Market Institutions for Effective and Efficient Water Allocation in South Africa. Pretoria, South Africa. ISBN: 9781770057326.

NRMMC, EPHC, & NHMRC. (2008). Australian Guidelines for Water Recycling - managing health

Nolde E. (1999). Greywater reuse systems for toilet flushing in multi- storey buildings – over ten years experience in Berlin. Urban Water 1 275-284.

Ocampo-Martinez, C., Puig, V., 2010. Piece-wise linear functions-based model predictive control of large-scale sewage systems. Iet Control Theory A. 4, 1581– 1593.

Okun DA (1996) Distributing reclaimed water through dual systems. J. Am. Water Works Assoc. 89 52-64

Ortiz-Zamora, D., & Ortega-Guerrero, A. (2010). Evolution of long-term land subsidence near Mexico City: Review, field investigations, and predictive simulations. Water Resources Research, 46, 1–15. doi:10.1029/2008WR007398. ISBN: 0043-1397. ISSN:0043-1397.

Oweis, T., & Hachum, A. (2006). Water harvesting and supplemental irrigation for improved water productivity of dry farming systems in West Asia and North Africa. Agricultural Water Management, 80(1–3 SPEC. ISS.), 57–73. doi:10.1016/j.agwat.2005.07.004. ISBN: 0378-3774. ISSN:03783774.

Page, D., Vanderzalm, J., Barry, K., Levett, K., Kremer, S., Ayuso- Gabella, N., Dillon, P., Toze, S., Sidhu, J. & Shackleton, M. (2009). Operational residual risk assessment for the Salisbury stormwater ASTR project. Water for a Healthy Country Flagship Report, April 2009.

Page, D., Dillon, P., Vanderzalm, J., Bekele, E., Barry, K., Miotlinski, K. & Levett, K. (2010). Managed aquifer recharge case study risk assessments. CSIRO Water for a Healthy Country Flagship Report

Pabi, S., Amarnath, A., Goldstein, R., & Reekie, L. (2013). Electricity Use and Management in the Municipal Water Supply and Wastewater Industries. Water Research Foundation. Retrieved from http://www.epri.com/search/Pages/results.aspx?k=3002001433

Palla, A., Gnecco, I., & Lanza, L. G. (2011). Non-dimensional design parameters and performance assessment of rainwater harvesting systems. Journal of Hydrology, 401(1- 2), 65–76. doi:10.1016/j.jhydrol.2011.02.009. ISSN:00221694.

Parkinson, J. & Mark, O. (2005). Urban Stormwater Management in Developing Countries, IWA

Publishing, London

Pedrero, F., Kalavrouziotis, I., Alarcón, J. J., Koukoulakis, P., & Asano, T. (2010). Use of treated municipal wastewater in irrigated agriculture-Review of some practices in Spain and Greece. Agricultural Water Management, 97(9), 1233–1241. doi:10.1016/j.agwat. 2010.03.003. ISBN: 0378-3774. ISSN:03783774.

Peña-Guzmán, C. A., Melgarejo, J., Prats, D., Torres, A., & Martínez, S. (2017). Urban Water Cycle Simulation/Management Models: A Review. Water, 9(4), 285. doi:10.3390/ w9040285. ISBN: 2073-4441. ISSN:2073-4441.

Peng, J., Cao, Y., Rippy, M. A., Afrooz, A. R. M. N., & Grant, S. B. (2016). Indicator and pathogen removal by low impact development best management practices. Water (Switzerland), 8(12), 1–24. doi:10.3390/w8120600. ISSN:20734441.

Philp, M., Mcmahon, J., Heyenga, S., Marinoni, O., Jenkins, G., Maheepala, S., & Greenway, M. (2008). Review of Stormwater Harvesting Practices: Urban Water Security Research Alliance Technical Report No . 9. Adelaide. ISSN:1836-5566.

Pitman, W. V. (2011). Overview of water resource assessment in South Africa: Current state and future challenges. Water SA, 37(5), 659–664. doi:10.4314/wsa.v37i5.3. ISBN: 0378-4738. ISSN:18167950.

Po, M, Kaercher J.D. & Nancarrow. B.E. (2003). Literature Review of Factors Influencing Public Perceptions of Water Reuse. Technical Report 54/03. CSIRO Land and Water.

Po, M., Nancarrow, B.E., Leviston, Z., Porter, N.B, Syme, G.J & Kaercher, J.D. (2005). Predicting Community Behaviour in Relation to Wastewater Reuse: What Drives Decisions to Accept or Reject? Water for a Healthy Country National Research Flagship. CSIRO Land and Water. Perth, Australia

Prathapar, S.A., (2000). Water shortages in the 21st century. In: Cadman, H. (Ed.), The Food and Environment Tightrope. Australian Centre for International Agricultural Research. ACT, Canberra, pp. 125–133.

Quayle, T. (2012). Recycling wastewater to bolster ground water supply: the story of Atlantis, South Africa. Cape Town, South Africa.

Radjenovic, J., Petrovic, M., Venturac, F., Barcelo, D., (2009). Rejection ofpharmaceuticals in nanofiltration and reverse osmosis membrane drinking water treatment, Water Research 42 (2008) 3601–3610.

Rajaganapathy, V., Xavier, F., Sreekumar, D., Mandal, P.K. (2011). Heavy metal contamination in soil, water and fodder and their presence in livestock and products: a review. J. Environ. Sci. Technol. 4, 234–249.

Rangari, V. A., Patel, A. K., & Umamahesh, N. V. (2000). Review of Urban stormwater Models. Environmental Modeling and Software, 16, 37.

Raskin, P., Gleick, P., Kirshen, P., Pontius, G., Strzepek, K. (1997). Water Futures: Assessment of Long-range Patterns and Prospects. Stockholm Environment Institute, Stockholm, Sweden.

Reichle, H., Tanre, D., Boucher, O., Nakajima, T., Orlando, J. J., Tyndall, G. S., & Dentener, F. (2003). References 5. Science, 302(1), 1719–1724.

Richards, L. A. (1931). Capillary conduction of liquids through porous mediums. Physics, 1, 318–333.

Rijsberman, F.R., ed. (2000). World Water Scenarios Analyses. London: Earthscan.

Rijsberman, F.R., Molden, D.J., 2001. Balancing Water Uses: Water for Food and Water for Nature. Thematic Background Papers. International Conference on Freshwater, Bonn, 3–7 December 2001, pp. 43–56.

Rijsberman, F. R. (2006). Water scarcity: Fact or fiction? Agricultural Water Management, 80(1–3), 5–22. doi:10.1016/j.agwat.2005.07.001. ISSN:03783774. Retrieved June 11, 2016 from http://linkinghub.elsevier.com/retrieve/pii/S0378377405002854

Rinaudo, J. (2015). Long - term water demand forecasting. Understanding and Managing Urban Water in Transition, p 239-268, 2015, ISBN: 9789401798013.

Rizzo, M., and Dougherty, D. E. (1996). "Design optimization for mul- tiple management period groundwater remediation." Water Resour. Res.,32 (8), 2549–2561.

Roberts, P. (2005). Yarra Valley Water 2004 Residential End Use Measurement Study. Yarra Valley Water. Melbourne, Australia. Retrieved November 13, 2016, from http://www.manuelectronics.com.au/pdfs/YarraValleyWater2004REUMS.pdf

Rohrer, A. (2014). Cape Town's Ponds. University of Cape Town. Department of Civil Engineering University of Cape Town, South Africa

Roebuck, R. (2007). A whole life costing approach for rainwater harvesting - An investigation into the whole life cost developments in the UK. University of Bradford, United Kingdom.

Rohrer, A. (2014). Cape Town Ponds. Department of Civil Engineering University of Cape Town, South Africa

Rohrer, A. (2017). The viability of using the stormwater ponds on the Diep River in the Constantia Valley for stormwater harvesting. Department of Civil Engineering University of Cape Town, South Africa

Rohrer, A. R., & Armitage, N. P. (2017). Improving the viability of stormwater harvesting through rudimentary real time control. Water (Switzerland), 9(6). doi:10.3390/w9060371. ISSN:20734441.

Rossman, A. L. (2010). Stormwater Management Model. Stormwater Management Model Version 5.

Roy, S. B., Chen, L., Girvetz, E. H., Maurer, E. P., Mills, W. B., & Grieb, T. M. (2012). Projecting water withdrawal and supply for future decades in the U.S. under climate change scenarios. Environmental Science and Technology, 46(5), 2545–2556. doi:10.1021/es2030 774. ISBN: 1520-5851 (Electronic)n0013-936X (Linking). ISSN:0013936X.

Rozos, E., Makropoulos, C., & Butler, D. (2010). Design Robustness of Local Water-Recycling Schemes. Journal of Water Resources Planning and Management, (October), 531–538. doi:10.1061/ASCEWR.1943-5452.0000067.

RSA. (2011a). National Climate Change Response: White Paper. Government Printer. Pretoria, South Africa. Retrieved from http://www.climateresponse.co.za/.

RSA. (2011b). National Development Plan: Our Future - make it work. National Planning Commission. Pretoria, South Africa. ISBN: 9780621411805.

Savva, A. P., & Frenken, K. (2002). Crop Water Requirements and Irrigation Scheduling. Food and Agriculture Organization, 132. doi:10.1007/978-1-4419-6335-2. ISBN: 0-7974-2316

Scarlett, K., Collins, D., Tesoriero, L., Jewell, L., van Ogtrop, F., & Daniel, R. (2015). Efficacy of chlorine, chlorine dioxide and ultraviolet radiation as disinfectants against plant pathogens in irrigation water. European Journal of Plant Pathology, 145(1), 27–38. doi:10.1007/s10658-015-0811-8. ISSN:15738469.

Schulze, R. (2005). Setting the scene : the current hydroclimatic "landscape" in southern Africa. In R. Schulze (Ed.), Climate change and water resources in southern Africa. Studies on Scenarios, Impacts, Vulnerabilities and Adaptation (pp. 81–94). Water Research Commission of South Africa. ISBN: 1770053654.

Schulze, R., Lumsden, T., Horan, M., Warburton, M., & Maharaj, M. (2005). An assessment of impacts of climate change on agrohydrological responses over Southern Africa. In R. Schulze (Ed.), Climate Change and Water Resources in Southern Africa: Studies on Scenarios, Impacts, Vulnerabilities and Adaptation (pp. 141–189). Water Research Commission (South Africa). Pretoria, South Africa.

Schutze, M., Campisano, A., Colas, H., Schilling, W., & Vanrolleghem, P. A. (2004). Real time control of urban wastewater systems - Where do we stand today? Journal of Hydrology, 299(3–4), 335–348. doi:10.1016/j.jhydrol.2004.08.010. ISBN: 0022-1694. ISSN:00221 694.

Seo, Y., Park, S., & Kim, Y.-O. (2015). Potential Benefits from Sharing Rainwater Storage Depending on Characteristics in Demand. Water, 7(3), 1013–1029. doi:10.3390/w7031013. ISBN: 8253810288. ISSN:2073-4441.

Seward, P. (2009). The Cape Flats Aquifer: A neglected resource needing integrated water resource management. In Proceedings of the 2009 Biennial Groundwater Conference "Pushing the Limits", At Somerset West, South Africa. Cape Town, South Africa.

Seymour, R. M. (2005). Capturing Rainwater to Replace Irrigation Water for Landscapes: Rain Harvesting and Rain Gardens. Landscape, [np]. ISBN: 0935835091.

Shiklomanov, I.A. (1998). Archive of World Water Resources and World Water Use. Global Water Data Files. State Hy- drological Institute, St. Petersburg, Russia, CD-ROM. November 9-10.

Sidhu, J.P.S., Hodgers, L., Ahmed, W., Chong, M.N., Toze, S., 2012. Prevalence of human pathogens and indicators in stormwater runoff in Brisbane, Australia. Water Res. 46, 6652–6660.

Smith, J. A. (2010). How much water is enough? Domestic metered water consumption and free basic water volumes: The case of Eastwood, Pietermaritzburg. Water SA, 36(5), 595–606. doi:10.4314/wsa.v36i5.61993. ISSN:03784738.

Smith, M. (1992). CROPWAT, a computer program for irrigation plan- ing and management, FAO Irrigation and Drainage Paper 46.,

South African National Roads Agency Limited. (2013). Drainage Manual, 6th Edition. Pretoria. ISBN: 1868443280. Retrieved from www.nra.co.za

Stahre, P. (2006). Sustainability in Urban Storm Drainage. Svenskt Vatten. Stockholm. ISBN: 9185159204.

StatsSA. (2011). Statistical release (Revised) P0301.4 Census 2011. Statistics South Africa. Pretoria, South Africa.

Steduto, P., Faurès, J., Hoogeveen, J., Winpenny, J., & Burke, J. (2012). Coping with water scarcity: An action framework for agriculture and food security. ISBN: 978-92-5-107304-9. ISSN:1020-1203.

Stephenson D and Turner K (1996) Water demand patterns in Gauteng. IMIESA 21 (1) 11-16.

Swartz, C., Thompson, P., Maduray, P., Offringa, G., & Mwiinga, G. (2013). WATCOST - Manual for a Costing Model for Drinking Water Supply Systems Report. Pretoria, South Africa. ISBN: 9781431204014.

Tambosi, JL., de Sena, RF., Favier, M., Gebhardt, W., José, HJ., Schröder, HF., Moreira R (2010) Removal of pharmaceutical compounds in membrane biore- actors (MBR) applying submerged membranes. Desalination 261:148–156

Thorndahl S., Poulsen T. S., Bøvith T., Borup M., Ahm M., Nielsen J. E., Grum M., Rasmussen M. R., Gill R. and Mikkelsen P. S. (2013). Comparison of short-term rainfall forecasts for modelbased flow prediction in urban drainage systems. Water Science and Technology, 68(2), 472-478.

Tortajada, C., Joshi, Y., & Biswas, A. (2013). The Singapore Water Story: Sustainable Development in an Urban City State. Routledge 2 Park Square, Milton Park, Abingdon, Oxon, OX14 4RN.

Toze, S. (2006). Reuse of effluent water - Benefits and risks. Agricultural Water Management, 80(1–3 SPEC. ISS.), 147–159. doi:10.1016/j.agwat.2005.07.010. ISBN: 0378-3774. ISSN:03783774.

Tredoux, G., Ross, W., & Gerber, A. (1980). The potential of the Cape Flats aquifer for the storage and abstraction of reclaimed effluent (South Africa). In Proceedings of the International Symposium on Artificial Groundwater Recharge, 14–18 May 1979, Germany. Z. Dt. Geol. Ges. 131 23–43.

Tredoux, G., Van Der Merwe, B., and Peters, I. (2009). Artificial recharge of the Windhoek aquifer, Namibia: Water quality considerations. Boletín Geológico Y Minero, 120(2), 269–278. ISSN:0366-0176.

Tredoux, G., & Cain, J. (2010). The Atlantis Water Resource Management Scheme: 30 years of Artificial Groundwater Recharge. Department of Water Affairs, 76. doi:PRSA 000/00/11609/10 - Activity 17.

Trowsdale, S. A., & Simcock, R. (2011). Urban stormwater treatment using bioretention. Journal of Hydrology, 397(3–4), 167–174. doi:10.1016/j.jhydrol.2010.11.023. ISBN: 00 22-1694. ISSN:00221694.

Turner, N. C. (2006). Preface. Agricultural Water Management, 80(1–3), 1–3. doi:10.1016/j. agwat.2005.07.002. ISSN:03783774. Retrieved from http://linkinghub. elsevier.com/ retrieve/pii/S0378377405002842

Turpie, J., Winkler, H., & Midgley, G. (2002). Economic Impacts of Climate Change in South Africa : A Preliminary Analysis of Unmitigated Damage Costs. Southern Waters & Energy & Development Research Centre

Twort, A., Ratnayaka, D., & Brandt, M. (2000). Water Supply. (B. Heinemann, Ed.) (5th ed.). London, UK.

United Nations. (1999). The world at six billion. United Nations. Retrieved February 8, 2016, from http://www.un.org/esa/population/publications/sixbillion/sixbilpart1

USEPA. (1995). Post-Construction Stormwater Management in New Development and Redevelopment.

USEPA. (2004). Guidelines for Water Reuse. United States Environmental Protection Agency (Vol. 26). Washington, D.C. ISBN: EPA 625/R-92/004. Retrieved from http://www.epa.gov/nrmrl/pubs/625r04108/625r04108.pdf

USEPA. (2005). Guidelines for carcinogen risk assessment. Risk Assessment Forum, United States Environmental Protection Agency, Washington, DC. EPA/630/P-03/001F.

USEPA. (2006). Real Time Control of Urban Drainage Networks. Washington, D.C. ISBN: EPA/600/R-06/120. Retrieved from http://nepis.epa.gov/Adobe/PDF/P1008A1S.pdf

USEPA. (2016). Estimating Monetized Benefits of Groundwater Recharge from Stormwater Retention Practices. Washington D.C.

Vallabhaneni, S., & Speer, E. (2011). Real-Time Control to reduce combined sewer overflows. Water World, 27(2), 1

van Daal, P., Gruber, G., Langeveld, J., Muschalla, D., & Clemens, F. (2017). Performance evaluation of real time control in urban wastewater systems in practice: Review and perspective. Environmental Modelling and Software, 95, 90–101. doi:10.1016/j.envsoft.2017.06.015. ISSN:13648152.

van Vuuren, S., & Van Beek, J.C. (1997). Re-evaluation of the Existing Urban and Industrial Water Management Guidelines Based on Measured Water Abstraction Phase 1: Pretoria Voorsienings- gebied. WRC Report No. 705/1/97, Water Research Commission, Pretoria, South Africa.

van Vuuren, S., & van Dijk, M. (2006). Life Cycle Costing Analyses for Pipeline Design and Supporting Software. Pretoria, South Africa. ISBN: 1770054499. Retrieved from http://www.wrc.org.za/Knowledge Hub Documents/Research Reports/TT 278-06.pdf

Van Vuuren, D. P., Edmonds, J., Kainuma, M., Riahi, K., Thomson, A., Hibbard, K., Hurtt, G. C., Kram, T., Krey, V., Lamarque, J. F., Masui, T., Meinshausen, M., Nakicenovic, N., Smith, S. J., & Rose, S. K. (2011). The representative concentration pathways: An overview. Climatic Change, 109(1-2), 5–31. doi:10.1007/s10584-011-0148-z. ISBN: 0165-0009. ISSN:01650009.

Van Zyl, H., Ilemobade, A., van Zyl, J., & Africa, S. (2008). An improved area-based guideline for domestic water demand estimation in South Africa. WaterSA, 34(3), 381–392. ISSN:03784738.

Van Zyl, F., Manus, N., & Pensulo, C. (2008). Water Services: Infrastructure Asset Management for municipal managers and management. Municipal Indaba 2008

van Zyl, J. (2014). Introduction to Operation and Maintenance of Water Distribution Systems. Water Research Commision. ISBN: 9781431205561.

Vanderalm, J., Levett, K., Page, D., Dillon, P., Toze, S., Miotlinski, K., Sidhu, J., Barry, K.,

Alexander, K., Hyde, K., & Regel, R. (2010). Assessing the Risks of Recycling Urban Stormwater for Potable Supply Via an Aquifer. National Conference of the Stormwater Industry Association, 1–9.

Vandoolaeghe, M. A. C. (1989). The Cape Flats groundwater development pilot abstraction scheme. Cape Town, South Africa

Veefkind, M. (2002). Life Cycle Costing. [online]. Available: http://www.io.tudelft.nl/research /tpa/menno/ls/ch3.htm [2005, December 11].

Vezzaro, L., & Grum, M. (2014). A generalised Dynamic Overflow Risk Assessment (DORA) for Real Time Control of urban drainage systems. Journal of Hydrology, 515, 292–303. doi:10.1016/j.jhydrol.2014.05.019. ISBN: 0022-1694. ISSN:00221694.

Weiss, P. T., LeFevre, G., & Gulliver, J. S. (2008). Contamination of Soil and Groundwater Due to Stormwater Infiltration Practices. Minnesota Pollution Control Agency. Minnesota. ISBN: 515. 115341

Westerhoff, P., & James, J. (2003) Nitrate Removal in Zero-valent Iron Packed Columns, Water Res. 2003, 37, 1818–1830

WHO. (2008). Guidelines for Drinking-Water Quality 3rd edi- tion, Vol.1, Recommendations. World Health Organisation, Geneva.

Willems, P. (2009). A time series tool to support the multi-criteria performance evaluation of rainfall-runoff models. Environmental Modelling and Software, 24(3), 311–321. doi:10.1016/j.envsoft.2008.09.005. ISBN: 1364-8152. ISSN:13648152.

Willems, P., Olsson, J., & Arnbjerg-Nielsen, K. (2012). Impacts of Climate Change on Rainfall Extremes and Urban Drainage Systems. IWA Publishing. ISBN: 1780401256.

Wilson, Z., & Pfaff, B. (2008). Religious, philosophical and environmentalist perspectives on potable wastewater reuse in Durban, South Africa. Desalination, Elsevier B.V. doi:10.10.1016/j.desal.2007.07.022.

Wong, T. (2007). Water Sensitive Urban Design – the Journey thus far. Australian Journal of Water Resources. Vol 110. pp 213–222.

Wong, T. H. (2011). Sustainable urban water management and water sensitive cities. ISBN: 9781921853067.

Wong, T., Allen, R., Beringer, J., Brown, R., Chaudhri, V., Deletic, A., Fletcher, T., Gernajak, W., Hodyl, L., Jakob, C., Reeder, M., Tapper, N., & Waslh, C. (2012). Stormwater Management in a Water Sensitive City. The Centre for Water Sensitive Cities Monash Sustainability Institute. Melbourne, Australia. ISBN: 9781921912009.

Woods-Ballard, B., Ashley, R., Wilson, S., Udale-, H., Kellagher, R., Bray, B., Shaffer, P., Baylis, A., Wallingford, H. R., Park, H., Ox, O., & Kingdom, U. (2014). Designing Sustainable Drainage Systems — UK Guidance. In 13th International Conference on Urban Drainage, Sarawak, Malaysia, 7-12 September 2014 (pp. 1–9).

Woods-Ballard, B., Wilson, S., Udale-Clarke, H., Illman, S., Scott, T., Ashley, R and Kellagher, R (2015) The SuDS Manual, C753, CIRIA, London, UK. ISBN 978-0-86017-760-9

Wu, Z., McKay, J., & Keremane, G. (2012). Issues affecting community attitudes and intended behaviours in stormwater reuse: A case study of Salisbury, South Australia. Water

(Switzerland), 4(4), 835–847. doi:10.3390/w4040835. ISBN: 6188302099. ISSN:207-34441.

Wurbs, R.A., 1994. Computer Models for Water Resources Planning and Management. Institute for Water Resources, US Army Corps of Engineers, Alexandria, Virginia. Report 94-NDS-

WWAP. (2018). The United Nations World Water Development Report 2018: Nature-based Solutions for Water:

XP Solutions. (2014). XPSWMM Hydrodynamic Modeling. Retrieved February 20, 2016, from http://xpsolutions.com/Software/XPSWMM/

Xu, H., Rahilly, M., & Maheepala, S. (2010). Assessing the impact of spatial lumping on rainwater tank performance using daily modelling. In Submitted to the 9th International Conference on Hydro-informatics. Chinese Academy of Sciences conference, May 9-11, 2011, Adelaide, Australia.

Yener J, Kopac T, Dogu G, Dogu T (2008) Dynamic analysis of sorption of Methylene Blue dye on granular and powdered acti- vated carbon. Chem Eng J 144:400–406

Yim S-K, Ahn W-Y, Kim G-T, Koh G-W, Cho J, Kim S-H (2007) Pilot-scale evaluation of an integrated membrane system for do- mestic wastewater reuse on islands. Desalination 208:113–124

Youngblood, S., Vogel, J., Brown, G., Storm, D., McLemore, A., & Kandel, S. (2017). Field studies of microbial removal from stormwater by bioretention cells with fly-ash amendment. Water (Switzerland), 9(7), 1–12. doi:10.3390/w9070526. ISBN: 2073-4441. ISSN:20734441.

Yu, P.S., Yang, T.C. & Chen, S.J. (2008). Comparison of uncertainty analysis methods for a distributed rainfall–runoff model. Journal of Hydrology, Vol. 244, No. 1–2, pp. 43-59.

Yuan, T., Fengmin, L., & Puhai, L. (2003). Economic analysis of rainwater harvesting and irrigation methods, with an example from China. Agricultural Water Management, 60(3), 217–226. doi:10.1016/S0378-3774(02)00171-3. ISSN:03783774.

Ziervogel, G., & Smit, W. (2009). Learning to swim : Strengthening flooding governance in the City of Cape Town. In 2009 Amsterdam Conference on the Human Dimensions of Global Environmental Change 'Earth System Governance: People, Places and the Planet', 2-4 December 2009, Amsterdam (pp. 1–16). Amsterdam.

Zimmerman, B.D., Korajkic, A., Brinkman, N.E., Grimm, A.C., Ashbolt, N.J., Garland, J.L. A spike cocktail approach to improve microbial performance monitoring for water reuse. Water Environ. Res. 2016, 88, 824–837.

Zoppou, C. (2001). Review of urban storm water models. Environmental Modelling and Software, 16(3), 195–231. doi:10.1016/S1364-8152(00)00084-0. ISBN: 1364-8152. ISSN:13648152.

Appendices

Appendix 1: Conference Presentations

Some of the work in the report have been presented at international conferences as listed below

Okedi, J., & Armitage, N.P. (2018). Benefits of Real-Time Control for Catchment Scale Stormwater Harvesting in Cape Town, South Africa. 11th IWA/IAHR International Urban Drainage Modelling Conference in Palermo, Italy, 23-26 Sept 2018.

Okedi, J., Armitage, N. P., Carden, K.J., & Mahinthakumar, K (2017). An investigation into the potential for catchment scale stormwater harvesting in Cape Town, South Africa. 14th International Conference on Urban Drainage, September 10 - 15, 2017 | Prague, Czech Republic

Okedi, J., Armitage, N. P., & Carden, K.J. (2017). Stormwater as a water resource – a case of Zeekoe catchment in Cape Town, South Africa. 6th World Sustainability Forum 2017: Cape Town, South Africa, 27 - 28 January 2017.

Fisher-Jeffes, L. N., **Armitage, N. P**., Carden, K.J., Winter, K. & **Okedi, J.** (2016). Addressing Water Scarcity in South Africa through the Use of LID. The EWRI International Low Impact Development 2016 Conference in Portland, Maine, USA, 20-23 September 2016.

Okedi, J., Alastair, R., & **Armitage, N.P**. (2015). Towards improving the performance of Cape Town's stormwater ponds using a SuDS approach. 10th IWA/IAHR International Urban Drainage Modelling Conference in Quebec City, Canada, 29-31 August 2015.

Appendix 2: Ethics in Research Clearance

EBE Faculty: Assessment of Ethics in Research Projects

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). Students must include a copy of the completed form with the thesis when it is submitted for examination.

Name of Principal Researcher/Student: ***JOHN OKEDI*** Department: ***CIVIL ENGINEERING***

If a Student: Degree: **PhD *CIVIL ENGINEERING*** Supervisor: **PROFESSOR NEIL ARMITAGE**

If a Research Contract indicate source of funding/sponsorship: **CARNEGIE**

Research Project Title: **APPLICATION OF SUDS TECHNIQUES FOR PERFORMANCE IMPROVEMENT OF STORMWATER PONDS 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.

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:	Okedi John	13th April 2015

This application is approved by:

Supervisor (if applicable):	Professor Neil Armitage	13th April 2015
HOD (or delegated nominee): Final authority for all assessments with NO to all questions and for all undergraduate research.	Professor Neil Armitage	13th April 2015
Chair : Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the above questions.	G. Sithole	14/05/2015

Appendix 3: Data disclosure statement

South African Weather Service

ISO 9001 Certified Organisation

Disclosure Statement

4. GRABOUW STEENBRAS IV (0005760 3) - all daily rain and temperature (max, mean, min) from 1996
5. GRABOUW (0006039 7) - all daily rain and temperature (max, mean, min) from 1902
6. ELGIN EXP FARM (0006038 5) - all daily rain and temperature (max, mean, min) from 2004
7. NUWEBERG (0006065 1) - all daily rain and temperature (max, mean, min) from 1927
8. DISAVLEI (0006031 2) - all daily rain and temperature (max, mean, min) from 1972
9. JONKERSNEK (0022030 3) - all daily rain and temperature (max, mean, min) from 1974
10. VIRGIN PEAKS (0022029 2) - all daily rain and temperature (max, mean, min) from 1945
11. SWARTBOSKLOOF 2D (0021809 7) - all daily rain and temperature (max, mean, min) from 1936
12. JONKERSHOEK (0021778 8) - all daily rain and temperature (max, mean, min) from 1935
13. STELLENBOSCH (0021656A6) - all daily rain and temperature (max, mean, min) from 1978
14. STRAND (0005609 8) - all daily rain and temperature (max, mean, min) from 1996
15. HELDERBERG NATURE RESERVE (0005634 0) - all daily rain and temperature (max, mean, min) from 1991
16. HELDERBERG KOLLEGE (0005603 7) - all daily rain and temperature (max, mean, min) from 1987
17. MITCHELLS PLAIN WOLFGAT (0005154 4) - all daily rain and temperature (max, mean, min) from 2005
18. RONDEVLEI (0004874) - all daily rain and temperature (max, mean, min) from 1952
19. CAPE TOWN WO (0021178A3) - all daily temperature (max, mean, min) from 1992
20. ALTYDGEDACHT (0021230 3) – all daily temperature (max, mean, min) from 1923

I hereby accept that:

- SAWS will be acknowledged in the resulting thesis/project or when published, for the data it provided.
- SAWS will be provided with a copy of the final results in printed or electronic format.
- The data received shall not be provided to any third party.

Signature of the User: Date: 18th August 2015

(Please sign the document and do not type your name in as this is a legal document and requires a signature.

Private Document

Document Template Reference: CLS-Disclosure-001.6
Record Reference: CLS-CI-DS
Page **3** of **3**

Appendix 4 – Historical data aggregated to monthly values

Appendix 4a – Historic rainfall data aggregated to monthly values – Airport Station

	Mean monthly precipitation 1996 - 2015 (mm/month) – Airport Station												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1996	2	35	27	33	55	131	85	107	100	69	34	28	707
1997	10	2	2	41	74	106	15	82	8	25	48	12	423
1998	10	0	14	35	120	53	100	46	31	15	47	45	517
1999	1	1	0	57	35	83	41	105	98	1	16	3	442
2000	16	0	13	14	62	93	46	46	67	7	7	6	376
2001	8	5	3	39	81	62	208	97	47	26	13	6	595
2002	61	15	9	28	72	76	98	66	26	33	22	16	522
2003	2	8	48	12	37	25	33	100	64	19	6	21	376
2004	6	0	9	63	4	91	65	170	25	99	3	9	544
2005	25	2	9	95	78	90	65	90	30	14	20	1	517
2006	0	13	5	30	122	34	71	56	20	37	38	10	436
2007	1	27	19	66	96	123	152	102	18	19	41	19	681
2008	7	14	5	15	51	63	182	80	138	12	53	8	629
2009	1	4	1	24	64	108	88	52	60	32	86	4	525
2010	3	8	6	12	95	70	40	32	24	31	28	18	369
2011	6	3	6	28	60	85	25	53	24	12	28	19	348
2012	3	6	23	44	40	78	92	82	55	34	8	1	466
2013	13	37	14	36	54	115	44	168	68	16	85	5	655
2014	23	2	44	24	61	109	106	91	28	5	21	3	517
2015	14	3	2	3	24	107	71	31	21	3	28	16	321

Appendix 4c – Historic mean monthly maximum temperature values – Airport Station

	Mean monthly maximum temperature (°C) – Airport Station												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Maximum
1996	27.4	26.9	24.7	24.6	21.3	18.5	16.7	17.2	17.3	19.9	20.7	24.2	27.4
1997	26.4	25.9	25.1	22.4	21.5	16.6	18.8	17.5	22.3	24.1	23.2	25.4	26.4
1998	26.1	28.9	25.5	23.8	20.2	18.1	17.3	19	19.3	22.2	23.2	26.1	28.9
1999	26.9	27.6	27.9	24.4	20.8	20.3	18.1	19.4	18.3	23.6	24.4	28.8	28.8
2000	28.4	27.5	26	24.1	21.2	20.3	17.9	19	18.7	22.5	24.7	24.4	28.4
2001	26.3	28	25.6	22.6	20.6	18.1	17.9	17	18.9	21.8	24.6	25.9	28
2002	25.4	28.3	26.9	23.5	19.6	16.5	16.8	18.8	21.3	21.2	22.7	26.6	28.3
2003	26.5	27.3	25.7	24.3	21.2	19.5	18	16.8	18.9	23.1	24.7	24.4	27.3
2004	27.6	27.3	24.5	22.8	21.3	19.2	18.7	17.7	20.7	21.4	24.6	26.5	27.6
2005	27.3	27.2	26.3	23.3	19.2	17.2	19.7	15.9	19.5	21.8	24.6	25	27.3
2006	27.7	27.7	25.4	22.8	19.9	20.1	16.9	17.7	20.9	22.4	24.6	25	27.7
2007	28.2	26.4	26.5	24	21.1	17.9	17.6	17.8	19.8	23.3	22.2	26.3	28.2
2008	26.5	26.6	26.6	24.1	21.4	17.7	16.7	18.3	18.1	22	23.2	25.5	26.6
2009	26.2	28.1	26.9	23.9	20.3	18.6	19.7	18.7	19.1	23	24.1	24.9	28.1
2010	26.7	27.5	26.8	23	19.8	18.6	18.2	19.3	20	21.8	23.6	26.9	27.5
2011	27.8	28.6	26.8	23.4	20.3	17.7	19.1	19	19.2	21.8	22.3	24.3	28.6
2012	28.3	26.7	26.1	23	19.5	17.9	17.3	16.4	19	21.1	23.7	27.4	28.3
2013	26.5	26.4	26.2	23.2	21.1	17.6	18.2	17.5	17.3	21.1	23.7	27	27
2014	27	28.2	24	25.5	20.2	18	17.3	19.2	20.2	25	24.3	25.8	28.2
2015	27.5	26.3	26.9	23.9	21.1	17.1	16.5	18.4	21	23.2	24.5	27	27.5

Appendix 4d – Historic mean monthly minimum temperature values – Airport Station

	Mean monthly minimum temperature (°C) – Airport Station												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Minimum
1996	16.3	15.4	13.1	12	9.1	7.9	6	6.8	9.3	10.7	11.9	14.8	6
1997	15.8	16.1	14.1	11.3	9.2	7.6	6.9	9.4	9.8	11.4	13	14.7	6.9
1998	15.8	17.5	14.8	12.8	10	8.9	6.9	7.2	8.1	10.3	12.8	15.6	6.9
1999	16.3	16.3	15.1	12.3	11.1	8.5	7.8	9	8	12.9	13.8	17.5	7.8
2000	15.8	16.9	15.7	11.8	10.3	9.4	8.1	9.8	9	10.4	13.9	15.3	8.1
2001	15.5	16	13.8	12.4	10.5	8.6	8.3	9.3	10.2	12.9	14.8	15.8	8.3
2002	15.6	16	15	12.2	9.8	7.8	7.2	7.1	10.4	10.6	11.3	16.3	7.1
2003	16.5	16.7	16	13.6	10.3	6.2	6.8	6.7	9.5	11.5	13.4	14.5	6.2
2004	17	16.8	13.2	12.3	10.7	8.2	6.1	8.6	9.3	11.7	15.1	16.3	6.1
2005	17	16.9	15	12.1	10.4	7	7.8	7.4	9.5	9.7	13.4	14.6	7
2006	16.9	16.5	14.1	12	9.3	8.1	8.7	7.9	10.3	11.3	13.9	15.4	7.9
2007	17.6	16.2	14.2	12.8	9.7	8.1	6.9	8.2	9.2	12.1	12.8	15.7	6.9
2008	16.8	16.9	14.8	12.2	12.9	9.8	7.5	7.7	7.6	11	14	16	7.5
2009	16.4	17	15.7	13.2	10.6	9.5	7.7	8.7	9.8	12.5	14.1	15.2	7.7
2010	17.2	17.1	15.9	12	10.3	7.5	6.1	7.5	9.5	11.1	13.2	16.3	6.1
2011	16.4	17.9	15.7	11.7	10.8	8.3	7	6.4	9	10.9	12.5	15	6.4
2012	17.9	16.7	16.1	12.5	8.9	8.1	7.5	7.4	8.5	11.9	13.3	17.2	7.4
2013	17.1	17.2	16.2	10.7	10.1	8	8.5	8	8.9	12.3	14.4	16.6	8
2014	17.9	18.1	14.9	13	10.5	7.8	7.2	9.7	9.9	12.1	14.1	16.2	7.2
2015	17.2	15.2	15.5	12.1	10.9	7.3	7.3	9.2	10.1	12.5	13.5	16.7	7.3

Appendix 4e – Historic mean monthly percentage humidity values – Airport Station

	Mean monthly humidity (%) – Airport Station											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1996	66.5	68.2	68.8	70.3	75.5	75.9	73	74.6	75.4	75	71	70.4
1997	67.3	64.3	72.8	77.5	77.3	79.4	73.7	80.8	73.4	64.8	65.9	68.8
1998	65.9	69.1	71.6	74.7	80.2	81.5	79.7	74.9	73.5	70.5	70.9	68.4
1999	68.8	64.6	69.8	74.8	78.9	76.1	77.9	75.2	77.1	67.9	66.3	65.9
2000	62.1	67.3	69.8	71.5	76.5	76.5	76.9	79.7	73.4	65.1	67.6	64.7
2001	61.8	62.8	69.7	72.5	70.8	74.5	71.8	74.7	73.8	73.2	68.1	66.6
2002	68	65.9	74.1	77.8	79	80.3	80.8	75.5	77.5	69.5	66.4	69.1
2003	66.5	70.3	74.7	77.5	80.3	76.5	74.8	77.7	75.1	69.4	64.7	65.2
2004	66.6	69.4	67.2	75.4	81.9	80.4	80	84.1	75	72.8	68.5	65.3
2005	65.5	70.1	70.6	79.2	82.4	85.9	80.7	83.7	79.8	71.2	67.9	62.6
2006	67.3	73	67	77.6	78.8	78.3	84.6	78.5	74.5	72	68.4	65
2007	66.9	71.3	70.5	75.9	81.9	81.2	79.2	77.4	73.7	67.4	69.5	69.5
2008	70.4	74.7	70.8	74.5	81.5	82.2	84.1	80.3	75.5	71.9	71.8	70.2
2009	69.7	66.1	72.3	77.7	83.8	81.9	74.9	77.4	75.4	71.5	69.8	69.1
2010	70.4	71.8	73.4	74.5	77.5	76.5	75.6	79.2	74.1	70.5	69	66.4
2011	69.6	74.6	75.5	73.1	80.4	84.6	75.9	73.8	75.6	70.2	67.5	68
2012	69.5	67.8	72.9	74	77.5	72.8	73.5	74.1	69	65.2	64.5	65.8
2013	62.5	66.4	66.4	69.6	73.6	69.7	71.4	67	67.4	68.4	67.5	64
2014	69.3	66.7	71	66.9	76.2	73.2	78.1	75.5	71.3	66.6	66.5	63.5
2015	66.6	63.5	68.1	70	76.7	84	81.5	82	69.4	69.2	62.5	64.8

Appendix 5: Data from climate change prediction models

Appendix 5a: Approval to use data from climate change prediction models

UNIVERSITY OF CAPE TOWN
IYUNIVESITHI YASEKAPA • UNIVERSITEIT VAN KAAPSTAD

Dept. Environmental and Geographical Science Phone: +27 21 650-2784,
Fax: +27 21 650-5773 Private Bag, Rondebosch, 7701, South Africa
email: barnard@csag.uct.ac.za

CSAG Data Provision Contract: No-cost data provision

This document outlines the terms of usage for data products provided by CSAG to external collaborators on a non-commercial basis, and for research purposes only.

1. **Category of service:** This contract is for the supply of data and related support information and materials (together termed CSAG products) for the intended application and uses in research, where the primary outputs are peer-reviewed academic research papers, public technical reports and/or student theses.

2. **Data licensing:** Products supplied by CSAG are considered to be licensed for use in the activities stipulated below only. CSAG retain all valid pre-existing intellectual property rights associated with products supplied. The products are not to be further re-distributed or disseminated to third parties, either freely or at any charge, without an express written agreement with CSAG. Existing public products supplied via CSAG are exempt.

3. **Quality and limitations:** The products developed by CSAG are supplied "as is", with no guarantees as to error and quality. CSAG will take all reasonable measures in developing and producing the products to ensure the best quality, but the products should at all times be considered research output that is subject to correction and change.

4. **Responsibilities of use:** It is the responsibility of the user to familiarise themselves with the product's information limitations, their appropriate application and usefulness, and the relevant caveats as to data uncertainty and noise. With explicit reference to climate change products (for past or future climates), the user is understood to be fully aware of the technical and methodological issues which may constrain spatial and temporal accuracy.

5. **Updates and enhancements:** The user is understood to recognize that CSAG supplies a versioned product. Through CSAG's activities, all products will periodically evolve and be further enhanced. Contradictions between older and newer versions may occur, while some differences are a near certainty, and the user is expected to be fully aware of this and is responsible for the appropriate cautionary application of the products.

6. **Acknowledgements:** CSAG should be acknowledged in all publications which are fully or partially based on the products from CSAG, whether electronic or hardcopy, with the appropriate inclusion of one or more of CSAG's logo, name, relevant funding agencies supporting CSAG (as may be specified), and references to relevant CSAG academic papers and reports. Where members of CSAG have made an identifiable contribution to reports and publications, these should include appropriate co-authorship.

Services provided under this Contract:

Downscaled Dataset:

Statistically downscaled station level daily temperature and precipitation from 11 CMIP-5 GCM simulations for the current and future projected climate.

The CMIP-5 GCMS provide a continuous 140-year period of data (1960-2109), under the RCP4.5 and RCP8.5 emission.

Downscaled Methodology:

The downscaled projections are produced using a statistical downscaling technique called Self-Organizing Map Downscaling (SOMD) developed at the Climate Systems Analysis Group (CSAG).

Reference:

Hewitson, B.C, and Crane, R.G., 2006. Consensus between GCM climate change projections with empirical downscaling: precipitation downscaling over South Africa. Int. J. Clim., 26, 1315-1337

Global Climate Models:

Coupled Model Intercomparison Project Phase 5 (CMIP5)

We acknowledge the World Climate Research Programme's Working Group on Coupled Modelling, which is responsible for CMIP, and we thank the climate modelling for producing and making available their model output. For CMIP the U.S. Department of Energy's Program for Climate Model Diagnosis and Intercomparison provides coordinating support and led the development of software infrastructure in partnership with the Global Organization for Earth System Science Portal.

Observed Dataset:

Daily observed records of rainfall, maximum and minimum temperature provided by the client (see Client Data Provision Contract for further details on terms and conditions of use).

For USER	For CSAG
Name: OKEDI JOHN	Name: LISA VAN AARDENNE
Institution: UNIVERSITY OF CAPE TOWN	
Date: 11th April 201	Date: 11th April 2016
Signature:	Signature:

Appendix 5b: List of climate change prediction models - the source of future rainfall and temperature data

1	bcc-csm1-1-rcp45	Beijing Climate Centre, China Meteorological Administration
2	bcc-csm1-1-rcp85	
3	BNU-ESM-rcp45	College of Global Change and Earth System Science, Beijing Normal University
4	BNU-ESM-rcp85	
5	CanESM2-rcp45	Canadian Centre for Climate Modelling and Analysis
6	CanESM2-rcp85	
7	CMCC-CESM-rcp85	Centro Euro-Mediterraneo Cambiamenti Climatic, Lecce LE, Italy
8	CNRM-CM5-rcp85	
9	CNRM-CM5-rcp85	
10	HadGEM2-CC-rcp45	National Institute of Meteorological Research/Korea Meteorological Administration
11	HadGEM2-CC-rcp85	
12	inmcm4-rcp45	Institute for Numerical Mathematics, Russian Academy of Sciences
13	inmcm4-rcp85	
14	IPSL-CM5A-MR-rcp45	Institut Pierre-Simon Laplace Atmosphere, France
15	IPSL-CM5A-MR-rcp85	
16	IPSL-CM5B-LR-rcp45	
17	IPSL-CM5B-LR-rcp85	
18	MIROC5-rcp45	Atmosphere and Ocean Research Institute (University of Tokyo), National Institute for Environmental Studies, and Japan Agency for Marine-Earth Science and Technology
19	MIROC5-rcp85	
20	MIROC-ESM-CHEM-rcp45	Japan Agency for Marine-Earth Science and Technology, Atmosphere and Ocean Research Institute (The University of Tokyo), and National Institute for Environmental Studies
21	MIROC-ESM-CHEM-rcp85	
22	MIROC-ESM-rcp45	
23	MIROC-ESM-rcp85	
24	MPI-ESM-LR-rcp85	Max Planck Institute for Meteorology, Hamburg, Germany
25	MRI-CGCM3-rcp45	Meteorological Research Institute, Japan
26	MRI-CGCM3-rcp85	

Appendix 5c – Future rainfall data from climate change prediction models aggregated to monthly values – Rondevlei Station

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Rondevlei Station							
	bcc-csm rcp45	bcc-csm rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCCCESM-rcp85	CNRMCM 5-rcp45	CNRMCM 5-rcp85
Jan	9	8	8	12	14	8	13	8	14
Feb	10	8	8	9	13	12	7	8	7
Mar	34	26	28	23	56	41	13	26	15
Apr	104	83	92	55	86	77	39	83	36
May	99	92	101	111	114	136	96	92	87
Jun	76	84	95	86	103	112	113	84	96
Jul	86	80	103	103	107	85	114	80	126
Aug	94	81	54	75	89	108	99	81	110
Sep	36	38	34	37	55	39	70	38	84
Oct	29	31	31	22	23	22	42	31	54
Nov	18	22	35	16	17	14	24	22	25
Dec	31	17	15	20	18	12	16	17	19
Mean	627	570	605	569	695	666	647	570	670
Max	719	698	758	683	825	764	842	698	852
Min	469	456	500	406	507	516	475	456	544

Appendix 5d – Future rainfall data from climate change prediction models aggregated to monthly values – Rondevlei Station (continued)

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Rondevlei Station							
	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45
Jan	13	10	11	6	7	8	11	9	10
Feb	15	9	7	8	19	7	12	8	12
Mar	20	16	45	20	40	21	28	25	45
Apr	34	40	39	57	91	63	106	90	77
May	88	73	70	81	128	110	85	97	96
Jun	79	101	66	73	92	85	72	85	113
Jul	120	137	64	71	86	95	88	83	103
Aug	107	111	55	59	83	76	95	89	102
Sep	63	81	26	24	38	24	54	50	36
Oct	59	81	17	21	48	29	47	20	16
Nov	42	33	9	13	19	11	38	28	11
Dec	26	18	11	11	10	10	20	15	9
Mean	667	710	419	444	661	538	657	599	630
Max	808	921	510	562	720	694	834	768	866
Min	594	550	227	346	562	375	470	391	414

Appendix 5e – Future rainfall data from climate change prediction models aggregated to monthly values – Rondevlei Station (continued)

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Rondevlei Station							
	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	10	16	9	18	14	6	9	12
Feb	18	13	15	13	9	5	8	8
Mar	34	22	11	22	16	12	14	18
Apr	77	38	24	46	30	39	29	26
May	94	89	77	101	71	99	87	75
Jun	103	97	75	92	78	92	94	90
Jul	127	117	99	103	117	106	133	103
Aug	89	105	95	105	65	108	110	119
Sep	33	78	54	62	63	73	76	73
Oct	12	48	36	45	54	38	55	53
Nov	5	48	35	38	37	27	27	29
Dec	8	20	22	31	17	16	19	10
Mean	610	691	551	676	570	622	661	617
Max	748	850	639	1025	776	764	892	687
Min	490	571	395	454	444	403	525	444

Appendix 5f – Future rainfall data from climate change prediction models aggregated to monthly values – Airport Station

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Airport Station							
	bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESM-rcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85
Jan	8	7	8	8	9	5	9	8	9
Feb	10	8	6	10	10	7	5	7	5
Mar	25	19	20	25	32	27	10	11	11
Apr	62	61	61	62	60	49	29	32	25
May	75	65	75	75	82	97	65	53	65
Jun	52	62	74	52	80	80	80	55	61
Jul	64	57	73	64	83	68	74	74	86
Aug	65	59	44	65	70	79	72	87	80
Sep	26	30	25	26	43	27	59	62	63
Oct	17	18	24	17	16	16	30	34	38
Nov	12	17	31	12	13	13	22	18	18
Dec	17	8	10	17	10	9	12	12	13
Mean	434	410	450	434	507	478	465	453	475
Max	535	507.37	557	535	586	568	590	565	594
Min	283	299	369	283	380	398	420	350	382

Appendix 5g – Future rainfall data from climate change prediction models aggregated to monthly values – Airport Station (continued)

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Airport Station							
	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45
Jan	13	8	8	8	5	4	9	5	6
Feb	12	8	6	6	13	5	10	7	10
Mar	14	10	32	15	22	19	16	14	26
Apr	27	37	26	38	60	42	69	58	56
May	49	55	48	56	85	74	64	68	68
Jun	46	65	45	55	68	59	59	58	81
Jul	82	89	48	50	60	74	64	64	79
Aug	76	81	42	42	63	56	71	70	58
Sep	48	57	21	19	29	20	37	37	30
Oct	39	56	11	14	30	18	36	16	9
Nov	29	24	8	9	19	9	26	24	10
Dec	22	14	8	6	7	8	14	15	8
Mean	457	504	304	319	462	386	476	436	442
Max	553	676	378	390	553	538	573	536	616
Min	396	375	175	226	337	243	372	298	310

Appendix 5h – Future rainfall data from climate change prediction models aggregated to monthly values – Airport Station (continued)

	Climate change prediction models - mean monthly precipitation 2090 - 2100 (mm/month) including mean annual precipitation with maximum and minimum values – Airport Station							
	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	7	12	6	12	9	7	9	10
Feb	10	9	9	10	6	6	7	7
Mar	22	13	8	17	10	8	10	12
Apr	33	29	19	30	20	28	15	16
May	58	63	51	58	44	61	55	48
Jun	60	68	56	64	53	67	63	72
Jul	80	86	77	73	81	79	85	73
Aug	54	77	66	68	47	84	83	81
Sep	19	59	40	50	39	49	58	51
Oct	10	39	27	29	35	27	39	40
Nov	7	40	26	29	28	18	21	24
Dec	6	16	15	24	12	10	16	8
Mean	366	509	399	466	383	441	462	443
Max	493	562	516	644	460	538	595	548
Min	334	421	274	354	297	328	356	337

Appendix 5i – Future temperature data from climate change prediction models aggregated to monthly values – Airport Station

	Climate change prediction models - mean monthly maximum temperature 2090 - 2100 (ºC) – Airport Station							
	bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESM-rcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85
Jan	28.5	29.8	28.6	30.8	28.9	30.5	29.0	28.5	29.8
Feb	28.5	29.8	29.0	31.2	29.0	30.4	30.3	29.3	30.7
Mar	26.4	28.0	26.7	28.9	25.9	27.4	30.4	29.2	29.9
Apr	22.3	23.8	23.2	25.8	22.3	23.9	28.0	26.3	28.7
May	20.9	22.2	21.8	23.4	20.9	22.6	24.7	22.8	23.9
Jun	20.7	21.5	21.3	23.2	20.7	22.1	22.5	20.3	21.8
Jul	21.2	22.3	21.7	23.7	21.5	22.7	22.8	20.6	21.8
Aug	21.6	23.2	22.5	24.1	22.4	23.5	23.4	21.2	21.7
Sep	22.9	24.4	23.2	25.4	23.4	25.2	23.7	21.6	22.2
Oct	25.2	27.3	24.4	27.6	26.0	28.0	25.1	23.2	24.7
Nov	27.3	28.7	26.3	29.5	28.8	29.9	27.4	25.8	26.8
Dec	27.4	29.0	27.6	29.9	28.5	30.2	28.5	27.4	28.5
Mean	24.4	25.8	24.7	27.0	24.9	26.4	26.3	24.7	25.9
Max	28.5	29.8	29.0	31.2	29.0	30.5	30.4	29.3	30.7
Min	20.7	21.5	21.3	23.2	20.7	22.1	22.5	20.3	21.7

Appendix 5j – Future temperature data from climate change prediction models – Airport Station

	Climate change prediction models - mean monthly maximum temperature 2090 - 2100 (ºC) – Airport Station							
	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45
Jan	26.4	29.4	28.9	30.7	30.2	32.8	27.0	29.4	29.2
Feb	27.6	30.4	28.5	29.8	30.1	32.7	28.1	29.6	28.1
Mar	27.9	30.4	25.1	26.7	26.6	30.1	26.4	27.7	24.7
Apr	26.6	28.8	22.6	24.1	23.3	26.5	22.0	23.7	22.4
May	24.2	25.3	21.4	22.6	22.0	24.4	20.6	21.6	20.5
Jun	21.9	23.6	20.5	21.6	21.4	24.0	20.5	21.1	20.1
Jul	21.6	22.7	21.4	22.6	22.2	24.2	21.1	21.9	20.7
Aug	21.8	22.9	22.8	23.2	22.7	25.1	21.4	23.0	21.2
Sep	22.0	23.1	24.5	25.2	23.9	26.9	22.1	23.9	24.2
Oct	23.3	24.5	26.2	27.3	26.0	29.5	23.5	25.5	26.2
Nov	25.0	28.0	28.6	29.3	28.4	31.7	25.3	27.0	28.1
Dec	26.0	28.9	28.8	30.1	29.4	32.4	26.3	28.1	28.8
Mean	24.5	26.5	24.9	26.1	25.5	28.4	23.7	25.2	24.5
Max	27.9	30.4	28.9	30.7	30.2	32.8	28.1	29.6	29.2
Min	21.6	22.7	20.5	21.6	21.4	24.0	20.5	21.1	20.1

Appendix 5k – Future temperature data from climate change prediction models – Airport Station (continued)

	Climate change prediction models - mean monthly maximum temperature 2090 - 2100 (ºC) – Airport Station							
	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	30.1	27.3	29.6	27.1	29.1	30.1	27.0	28.7
Feb	29.7	28.4	29.9	27.6	29.9	30.5	28.3	29.8
Mar	27.5	28.3	30.3	27.8	29.9	30.3	28.4	29.1
Apr	24.2	26.1	29.3	25.9	28.5	27.3	26.5	27.5
May	21.9	22.9	25.3	22.9	25.6	23.8	22.7	24.1
Jun	20.9	21.7	23.3	21.4	23.7	22.1	20.6	21.7
Jul	21.5	21.0	23.1	21.6	23.2	22.4	20.6	21.8
Aug	22.7	21.0	23.1	21.5	23.5	22.4	20.9	22.2
Sep	24.6	21.3	23.3	22.0	23.9	23.8	21.4	23.4
Oct	27.8	22.7	25.0	23.2	25.2	25.5	22.4	23.6
Nov	29.5	25.2	28.0	25.0	27.1	27.6	24.5	26.1
Dec	29.7	26.6	28.8	26.0	28.8	29.0	25.5	27.6
Mean	25.8	24.4	26.6	24.3	26.5	26.2	24.1	25.5
Max	30.1	28.4	30.3	27.8	29.9	30.5	28.4	29.8
Min	20.9	21.0	23.1	21.4	23.2	22.1	20.6	21.7

Appendix 5l – Future rainfall data from climate change prediction models – Airport Station (continued)

	Climate change prediction models - mean monthly minimum temperature 2090 - 2100 (ºC) – Airport Station							
	bcc-csm1-1-rcp45	bcc-csm1-1-rcp85	BNU-ESM-rcp45	BNU-ESM-rcp85	CanESM2-rcp45	CanESM2-rcp85	CMCC-CESM-rcp85	CNRM-CM5-rcp45	CNRM-CM5-rcp85
Jan	17.3	18.7	17.9	20.0	17.5	18.8	18.5	17.2	18.4
Feb	17.2	18.5	17.8	19.9	17.5	18.6	19.2	17.6	18.9
Mar	15.1	16.7	15.6	18.0	14.8	16.4	18.8	17.3	18.2
Apr	11.3	12.9	12.6	14.7	12.1	13.5	17.4	15.4	17.6
May	10.6	11.5	11.2	13.0	11.1	12.3	14.5	11.9	13.6
Jun	10.2	11.1	11.2	12.6	10.6	11.9	12.7	10.4	11.4
Jul	10.8	11.5	11.3	13.1	11.2	12.3	12.5	10.6	11.7
Aug	10.9	12.1	11.4	13.2	11.5	13.0	12.5	10.8	11.8
Sep	11.6	12.9	11.8	13.9	12.5	14.1	13.0	11.0	12.1
Oct	14.3	16.2	13.9	16.6	15.0	16.9	14.1	12.4	13.8
Nov	16.9	18.2	16.4	19.1	17.4	18.6	16.3	14.9	16.3
Dec	17.1	18.6	17.5	19.8	17.6	19.0	17.9	16.5	17.7
Mean	13.6	14.9	14.1	16.2	14.1	15.5	15.6	13.8	15.1
Max	17.3	18.7	17.9	20.0	17.6	19.0	19.2	17.6	18.9
Min	10.2	11.1	11.2	12.6	10.6	11.9	12.5	10.4	11.4

Appendix 5m – Future rainfall data from climate change prediction models – Airport Station (continued)

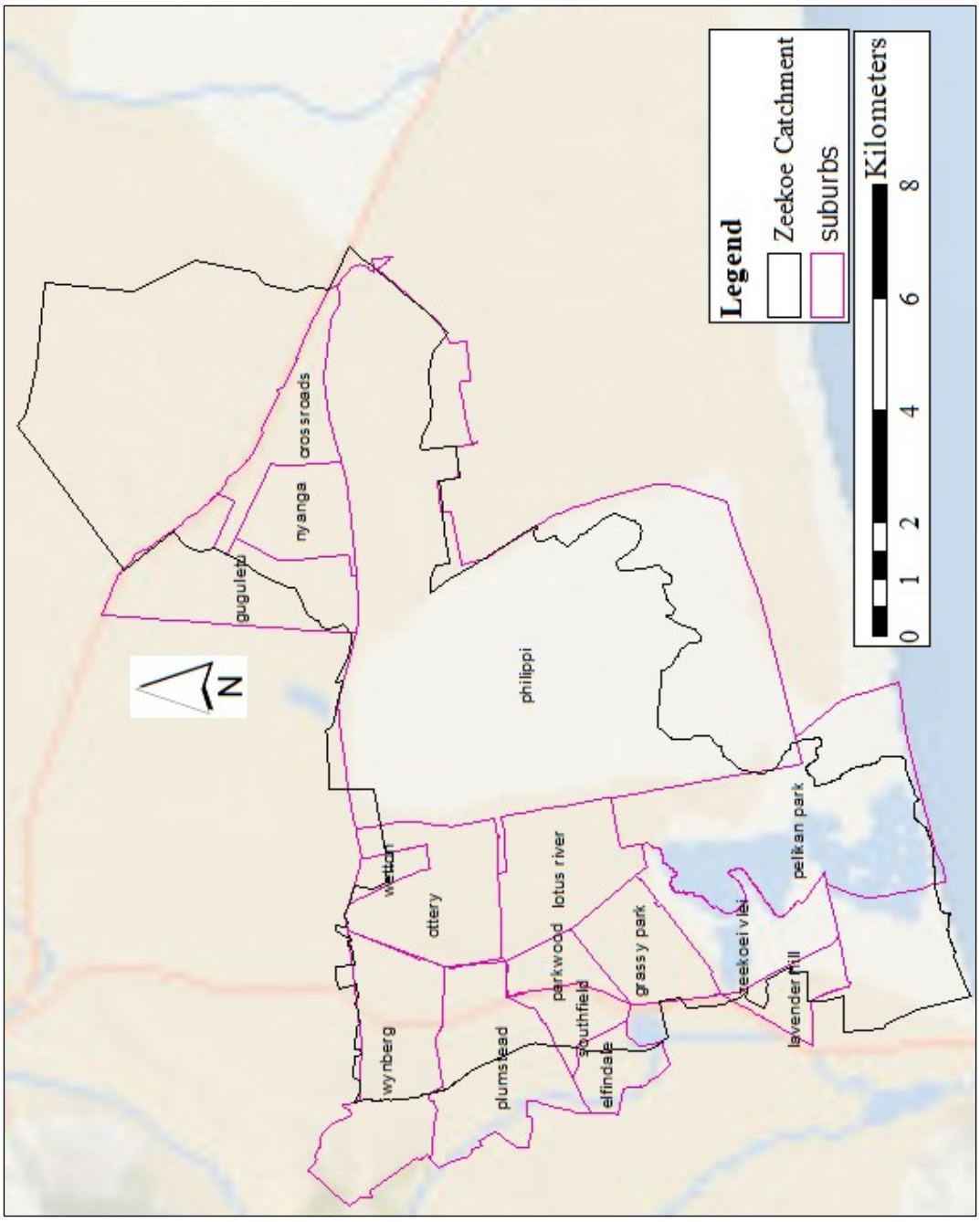
	Climate change prediction models - mean monthly minimum temperature 2090 - 2100 (°C) – Airport Station							
	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45
Jan	15.9	18.4	17.3	18.6	18.4	20.6	16.4	18.2	17.2
Feb	16.6	18.7	16.9	18.1	18.3	20.6	16.7	18.1	16.5
Mar	16.3	18.5	14.1	15.3	15.9	18.7	14.9	16.5	13.9
Apr	15.4	17.5	10.8	12.2	12.8	15.5	11.7	13.3	11.5
May	13.0	14.3	10.2	10.8	11.7	13.5	10.4	11.5	10.1
Jun	11.0	12.3	10.1	10.7	11.2	13.0	10.0	10.9	9.8
Jul	10.9	12.2	10.2	11.5	11.7	13.5	10.4	11.3	10.5
Aug	10.9	11.8	11.2	12.4	12.2	14.1	10.8	12.2	10.9
Sep	11.4	12.3	13.5	14.3	12.7	16.0	11.5	13.0	12.9
Oct	12.5	13.8	15.2	16.5	15.2	18.4	13.2	14.9	15.1
Nov	14.6	16.4	17.1	18.2	17.6	20.4	15.2	16.7	16.8
Dec	15.2	17.8	17.4	18.7	18.4	20.7	16.3	17.9	17.2
Mean	13.6	15.3	13.7	14.8	14.7	17.1	13.1	14.5	13.5
Max	16.6	18.7	17.4	18.7	18.4	20.7	16.7	18.2	17.2
Min	10.9	11.8	10.1	10.7	11.2	13.0	10.0	10.9	9.8

Appendix 5n – Historic mean monthly minimum temperature values – Airport Station (continued)

	Climate change prediction models - mean monthly minimum temperature 2090 - 2100 (ºC) – Airport Station							
	MIROC5-rcp85	MIROC-ESM-CHEM-rcp45	MIROC-ESM-CHEM-rcp85	MIROC-ESM-rcp45	MIROC-ESM-rcp85	MPI-ESM-LR-rcp85	MRI-CGCM3-rcp45	MRI-CGCM3-rcp85
Jan	18.0	17.3	19.4	17.0	19.0	18.8	16.4	17.7
Feb	17.9	17.9	19.6	17.6	19.4	18.6	17.0	18.1
Mar	15.9	17.5	19.4	17.2	19.1	18.2	16.5	17.7
Apr	13.2	15.4	18.0	15.1	17.6	16.2	15.3	16.3
May	11.0	12.3	14.5	12.4	14.5	13.2	12.0	13.5
Jun	10.9	11.3	12.6	11.1	12.8	11.7	10.4	11.8
Jul	11.2	11.2	12.6	11.0	12.8	11.7	10.4	11.5
Aug	11.9	11.1	12.5	11.4	12.6	11.8	10.5	11.7
Sep	13.3	11.3	12.6	11.3	13.0	12.8	10.7	12.6
Oct	16.5	11.6	14.0	12.3	14.4	14.4	11.8	13.0
Nov	17.8	14.4	17.0	14.4	16.6	16.7	13.6	15.1
Dec	18.2	16.5	18.7	16.2	18.3	18.1	15.3	17.0
Mean	14.7	14.0	15.9	13.9	15.8	15.2	13.3	14.7
Max	18.2	17.9	19.6	17.6	19.4	18.8	17.0	18.1
Min	10.9	11.1	12.5	11.0	12.6	11.7	10.4	11.5

Appendix 6: Demographics and water use in the study area

Appendix 6a – Suburbs in the study area



Appendix 6b – Population and gender per suburb in the study area

	Male	Female	Total
Crossroads	13,209	14,202	27,411
Elfindale	1,215	1,359	2,577
Grassy Park SP	9,126	10,089	19,212
Gugulethu SP	28,791	31,851	60,642
Lavender Hill	15,753	16,842	32,598
Lotus River	18,390	19,752	38,145
Nyanga	12,825	13,455	26,280
Ottery	3,855	4,149	7,998
Parkwood	5,703	6,168	11,871
Pelican Park	6,285	6,273	12,552
Plumstead	10,950	12,837	23,787
Southfield	3,483	3,621	7,104
Wetton	1,587	1,710	3,300
Wynberg	6,993	7,713	14,703
Zeekoevlei	210	207	420
Philippi	31,413	31,485	62,898

Appendix 6c – City of Cape Town approval to use water consumption data

w.2|oen|2799

CITY OF CAPE TOWN
ISIXEKO SASEKAPA
STAD KAAPSTAD

WATER AND SANITATION

Nina Viljoen
Research and Development Officer: Integrated Management Systems

T: +27 21 444 3398
E: nina.viljoen@capetown.gov.za

RMEMO:07

Research Study, Data or Interview Permission Request

Date:	05 November 2015
For Approval By:	Dr Gisela Kaiser, Executive Director: Utility Services
Subject:	Research Study Permission – PhD Student; UCT
Purpose:	Permission for Mr John Okedi to receive data for research thesis study titled: ***"Viability of Urban Stormwater Ponds as Water Resources in Cape Town – Case of the Zeekoevlei Catchment".***
Research Request Received on:	27 October 2015

Seeking permission for research study as follows:

(a) Permission to obtain and use Water and Sanitation, water consumption data for various suburbs around the Zeekoevlei catchment of the Cape Flats, Southern district.

Background

The ultimate aim of this project is to determine the viability of stormwater ponds to supplement the City of Cape Town's water needs whilst providing additional benefits such as water quality improvement, increased amenity and biodiversity preservation. With regard to water supply, envisaged storage options include open surface water storage – potentially enhanced through the use of 'Real Time Control' (RTC, i.e. the use of real-time weather reports to manage the water levels in the ponds so that they are as empty as possible before a storm event and as full as possible afterwards); and Managed Aquifer Recharge (MAR) where appropriate. Currently, detailed modelling of the Zeekoevlei catchment in the 'Cape Flats' is being modelled in the Water Evaluation and Planning (WEAP) environment (Sieber & Prukey, 2011). WEAP model is being used to estimate runoff, storage and infiltration at a daily time step – from which to allocate available water supply to demand whilst simultaneously predicting pollution generation and in-stream water quality.

The research will require 5 years of water consumption data (2010 – 2015) for the area in Cape Town around Zeekoevlei catchment of the Cape Flats. The zones include Parkwood, Crossroads, Montevideo, Boquinar, Industrial Area, King David Country Club, Guguletu, Manenberg, Primrose Park, Mountview, Newfields, Nyanga, Sand Industria, Pinati, Hanover Park, Weltevreden Valley, Elfindale, Zeekoeivlei, Philippi, St James, Lansdowne, Bishopscourt, Kenilworth, Wynberg, Pelikan Park, Strandfontein, Steenberg, Muizenberg, Vrygrond, Seawinds, Heathfield, Retreat, Grassy Park, Lavender Hill, Wetton, Ottery, Lotusriver, Plumstead, Diepriver, Southfield, Montana Extension, Cape Farms.

Questions as requested as part of the process for approval of research requests:

Question 1
Please provide a formal research proposal highlighting the research topic, hypothesis (if applicable), research methodology and intended sample group. The impact on the time participants would need to complete the research is also needed and must be clearly stated. Also, how would the researcher envisage accessing the participants? What would you require from the City for the research project- i.e. interviews, data etc.?

Answer: Topic: Viability of Urban Stormwater Ponds as Water Resources in Cape Town – Case of the Zeekoevlei Catchment. Please refer to attached project proposal for further information.

Question 2
What are the set deliverables of the research project?

Answer: 1) The identification of areas where stormwater ponds can be adapted to function as water resources; 2) identification of the potential benefits and costs of stormwater harvesting from stormwater ponds in these areas;

3

MARIO CARELSE Recommended ☒/Not Recommended ☐ 14/12/2015
Date
IMS SPECIALIST: WATER AND SANITATION

COMMENTS:

JACO DE BRUYN Recommended ☒/Not Recommended ☐ 14/12/2015
Date
HEAD: INTEGRATED PLANNING,
STRATEGY AND INFORMATION MANAGEMENT
(DATA CUSTODIAN)

COMMENTS:

DR GISELA KAISER Approved ☒/Not Approved ☐ Date
EXECUTIVE DIRECTOR: UTILITY SERVICES

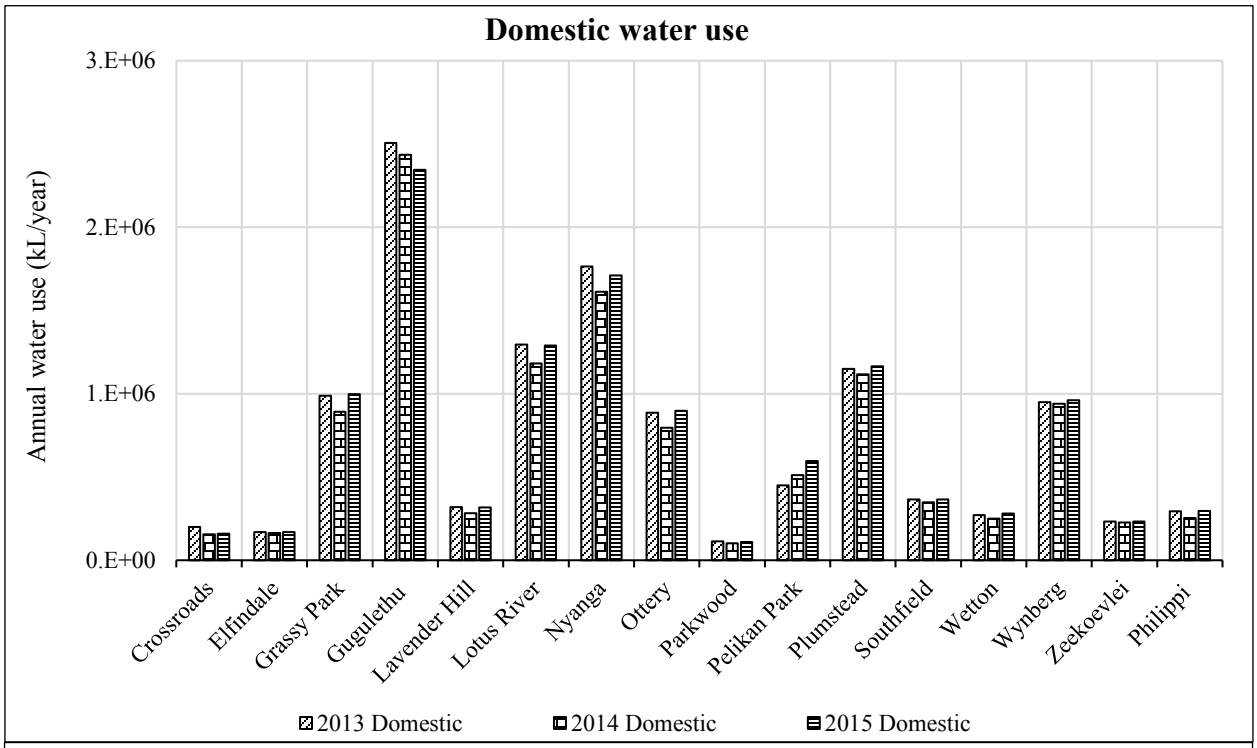
COMMENTS: Subject to conditions as listed

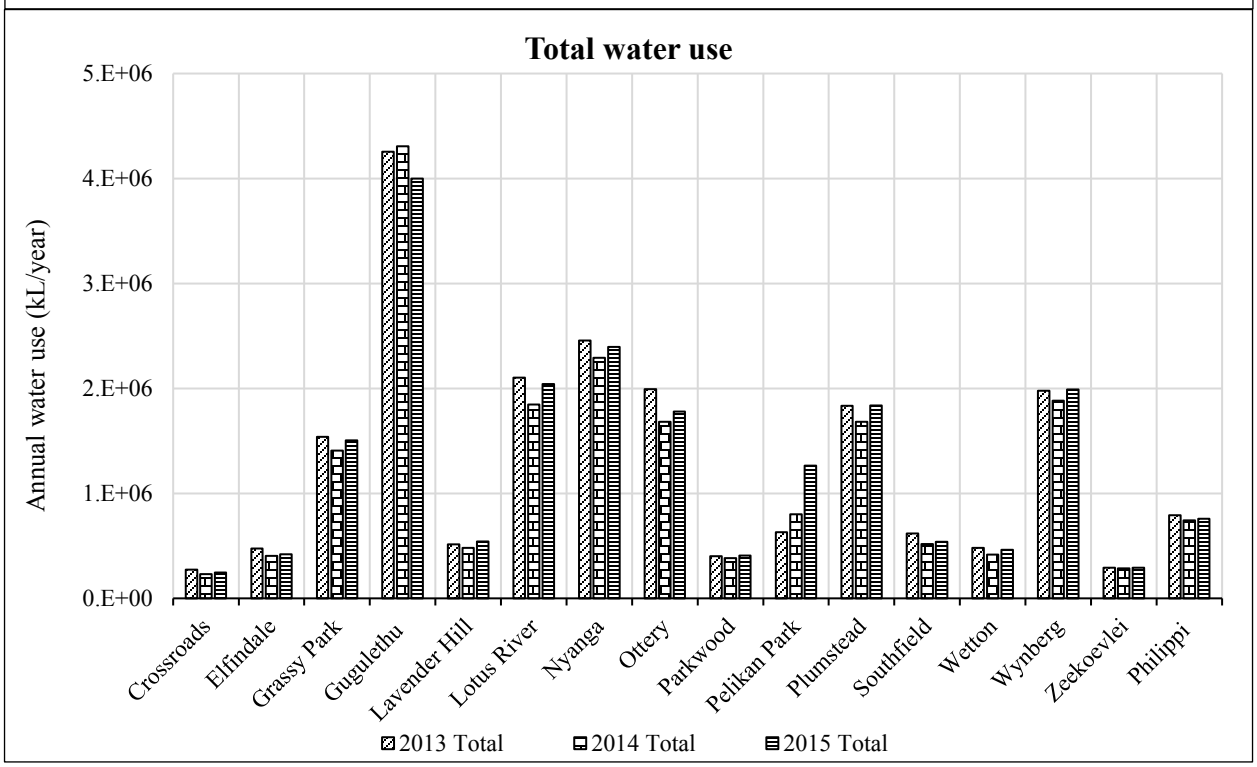
OFFICE OF THE EXECUTIVE DIRECTOR
UTILITY SERVICES
RECEIVED
28 DEC 2015
SIGNATURE:

CIVIC CENTRE IZIKO LEENKONZO ZOLUNTU BURGERSENTRUM
12 HERTZOG BOULEVARD CAPE TOWN 8001 PO BOX 298 CAPE TOWN 8000
www.capetown.gov.za

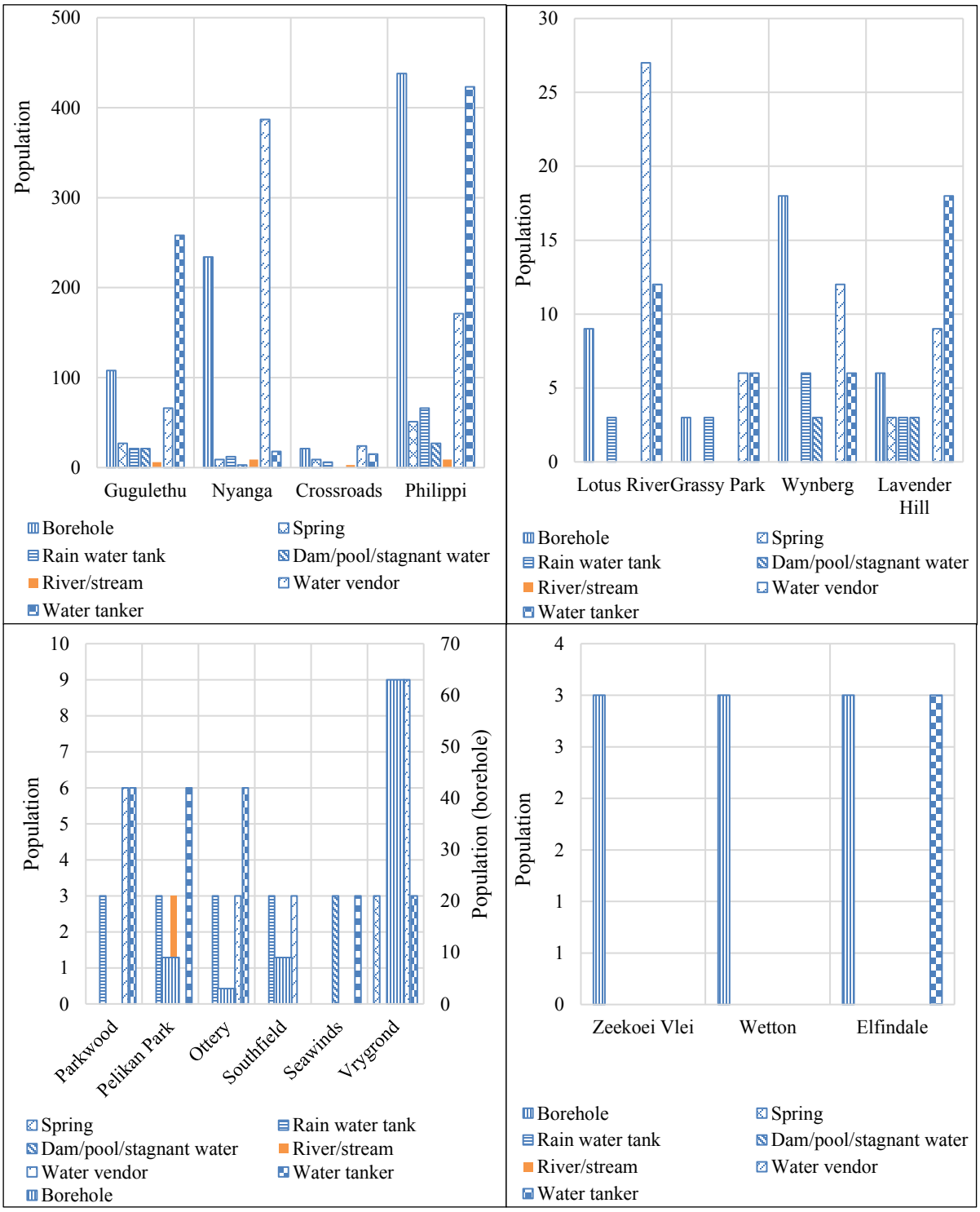
Making progress possible. Together.

Appendix 6d – Annual domestic and total water use per suburb from reticulation system

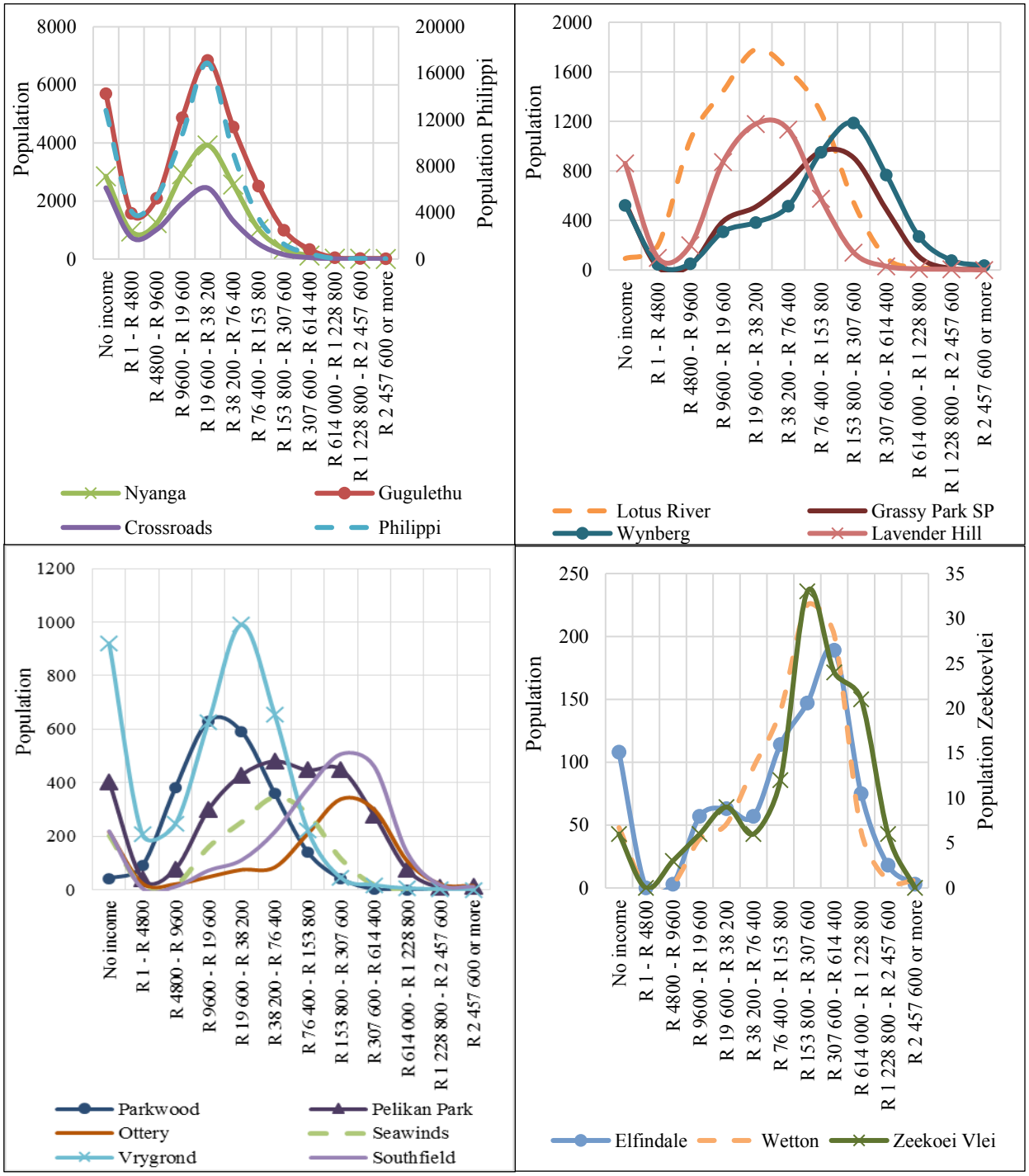




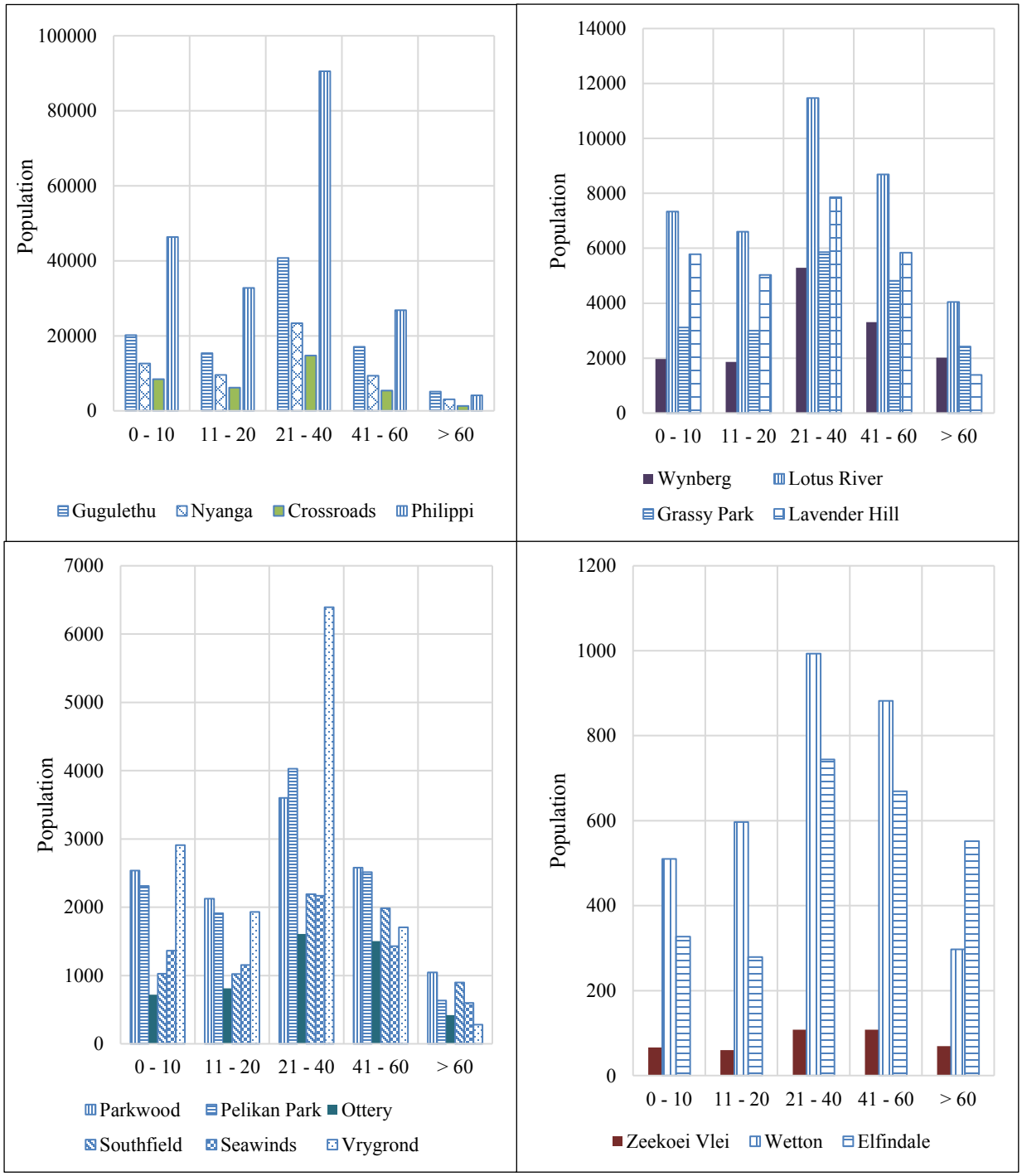
Appendix 6e – Household alternative water source per suburb (StatsSA, 2011)



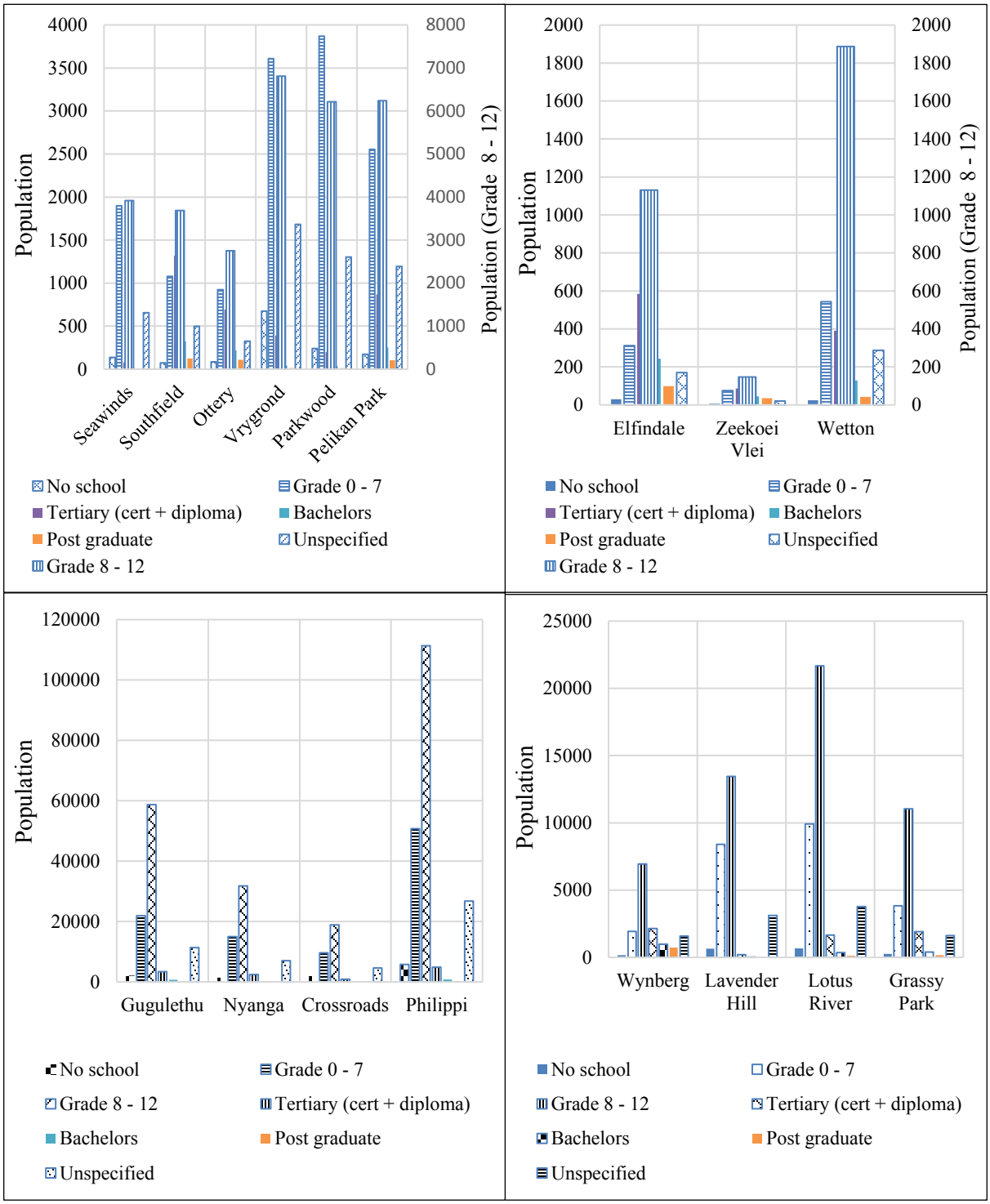
Appendix 6f – Household income groups per suburb in the study area (StatsSA, 2011)



Appendix 6g – Household age groups per suburb in the study area (StatsSA, 2011)



Appendix 6h – Level of Education per suburb (StatsSA, 2011)



Appendix 7: Innovations in Nature Based Stormwater management

Available literature shows that developing countries such as South Africa will be most affected by water scarcity due to rapidly growing domestic, commercial and industrial demands (Rijsberman, 2006). Another challenge is water availability which maybe abundant but in a form not easily usable by human, *e.g.* sea water and frozen water at the poles (IWMI, 2000). Seawater consists of 97% of total global water, 2.25% is trapped in glaciers and ice, leaving only 0.75% as freshwater in groundwater aquifers, rivers and lakes (Turner, 2006). Poor people in developing countries that can only access water in usable forms will be most affected by acute scarcity as water supplied would often be inadequate to meet their needs (Rijsberman, 2006).

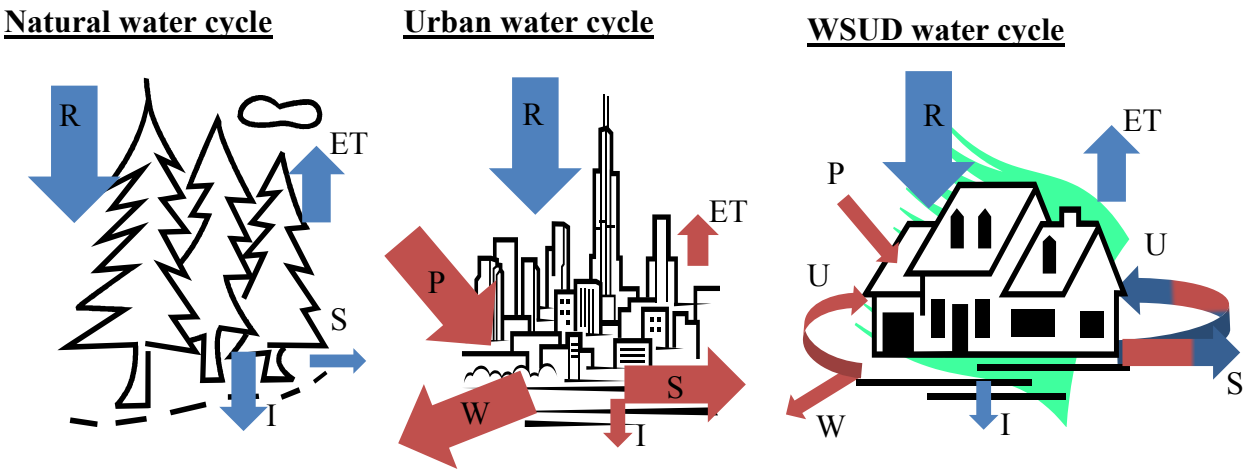
In the field of urban hydrology, several concepts aimed at preserving the environment and providing opportunity for stormwater reuse have emerged including *inter alia* Water Sensitive Urban Design (WSUD) in Australia, Low Impact Development (LID) in the United States and Sustainable Drainage Systems (SuDS) in the United Kingdom (Fletcher *et al*., 2014). The environmentally sensitive approaches that have evolved over the last three decades since 1990 including *inter alia* WSUD, LID and SuDS are linked to a philosophy where holistic water cycle management approach aims to minimise net outflow of stormwater from a given catchment (Fletcher *et al.*, 2014). The principles common to these environmentally sensitive approaches are summarised in Table 7a.

Table 7a Conventional to environmentally sensitive approaches (Fletcher *et al.,* 2014)

Conventional	**Environmentally sensitive approaches**
▪ End of catchment solution (reactive)	▪ Source and regional control solution (proactive)
▪ Flood management (Problem-solving)	▪ Water resource management (opportunity utilisation)
▪ Protection of human life and property	▪ Protection of human and ecosystem life, property and habitat
▪ Pipe and convey	▪ Mimic natural hydrology
▪ Single-use (flood management)	▪ Multifunctional (water quantity and quality management, amenity and biodiversity preservation)
▪ Solely owned and managed by local government/city department	▪ Public-private partnership (community participation and co-ownership)

In South Africa, the application of these concepts have been the subject of research by the Urban Water Management research unit at the University of Cape Town over recent years and has culminated in the publication of guidelines to assist in the design and management of SuDS in

South Africa (Armitage *et al.,* 2013), as well as a framework and guidelines for the implementation of WSUD (Armitage *et al.*, 2014). The general working principles of the environmentally sensitive approaches compared to other water balances is shown in Figure 7b.



R – Rain; ET – Evapotranspiration; I – Infiltration; S – Runoff; P – potable water; W – Wastewater; U – Recycle

Figure 7b WSUD Concept (Hoban & Wong, 2006)

The seasonal availability of stormwater in regions such as Cape Town, balancing storage is required to enhance the reliability of water supply from the option. There are various storage options for stormwater harvesting systems discussed in literature including closed storage (*e.g.* underground tanks and closed pipe networks), open storage (*e.g.* stormwater ponds and open pipe networks) and groundwater storage (through managed aquifer recharge). The determination and selection of a suitable storage option for a stormwater harvesting system would be case-specific and depends on climate, system yield, land availability, topography, geology, demand and end-uses. Further, the selection of the storage option would consider the scale of stormwater harvesting system and the intended application of the harvested water (Fisher-Jeffes, 2015). According to DECNSW (2006), the design of the storage option should consider how the water will be collected, stored, treated and distributed to end users. Mitchell *et al.* (2007) determined that the design of the storage option for a stormwater harvesting system should consider maximising volumetric reliability while minimising storage size and associated costs. The storage component of the stormwater management infrastructure was a critical element in this study as optimising storage in stormwater ponds and groundwater through managed aquifer recharge was the focus of the research. An overview of available storage options in literature is briefly discussed in this section. In closed storage systems such as underground tanks and closed pipe networks, stormwater is temporarily stored in sealed units where direct precipitation and

evaporation will not increase or decrease the stored volume (Fisher-Jeffes, 2015). Underground tanks that collect and temporary store rainwater that runs off roofs or properties (Hatt *et al.*, 2006; Begum *et al.*, 2008) and permeable paving (Armitage *et al.*, 2013) are some examples of closed storage. However, due to limited storage capacity, closed storage systems are limited to small-scale or property level stormwater harvesting and rarely applied in a catchment-scale system where significant uptake would be required for impact to be noticed (Hatt *et al.*, 2006).

If well designed, open storage systems such as stormwater ponds *i.e.* detention ponds, retention ponds and constructed can provide at least four types of benefits, *viz*.: the management of water quantity; the improvement of water quality; the provision of amenity; and the preservation of biodiversity (Armitage *et al.*, 2013). The management of water quantity can be further broken down into the reduction of flood peak flows and volumes, and the potential for stormwater to be a significant water resource in its right (Armitage *et al.*, 2013). Adaptation of stormwater ponds to function as a water resource was the focus of the study and the details are provided in this report. Other examples of open storage systems include open water bodies such as wetlands, dams, lakes (various shallow lakes referred to as vleis in the study area), rivers, streams and creeks (Goonrey, 2005). The use of natural open water bodies such as wetlands and lakes for stormwater harvesting would require an environmental impact assessment to determine the extent of the negative impact to other activities like recreation and ecology especially from a water quality perspective (Armitage *et al.*, 2013). Open storage systems are attractive to a range of flora and fauna that need to be protected from the poor water quality associated with stormwater (DECNSW, 2006; Armitage *et al.*, 2013).

Stormwater ponds refer to the regional control stormwater management infrastructure as described in the South African guidelines for SuDS (Armitage *et al.,* 2013). These stormwater ponds include detention ponds, retention ponds and constructed wetlands. Detention ponds are dry basins that temporarily hold stormwater for short periods of time to attenuate peak flows from storm events to mitigate flood risk downstream of the ponds (Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). Detention basins are typical in conventional stormwater management due to available storage capacity (no permanent pool of water) as the focus flood control (Armitage et al., 2013). Detention ponds are typically a vast expanse of depressions on land. Some detention pond designs including concrete linings and sports fields that could also be used as recreational facilities and car parks in residential and non-residential areas in dry periods when there is no flood. They can also be adapted to contribute towards aesthetic value and affluence of the area (Woods-Ballard *et al.,* 2015). Unfortunately, detention ponds may not be able to provide a water quality improvement benefit and the stormwater residence time is often minimal (Armitage *et al.,* 2013). A modified version of detention ponds, *i.e.* with extended stormwater detention period may provide water quality improvement, however, the level of improvement would still be limited (Armitage *et al.,* 2013). Retention ponds and constructed wetlands would provide a much

better benefit with regard to water quality improvement as both allow greater emphasis on water treatment (Armitage *et al.,* 2013).

Retention ponds hold a permanent pool of water providing some level of stormwater quality improvement in addition to peak flow attenuation from storm events to mitigate flood risk downstream of the ponds (Debo & Reese, 2003; Mays 2001; Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). Water quality improvement function in retention ponds is typically characterised by processes such as sedimentation, filtration, infiltration and biological uptake processes to remove pollutants from stormwater runoff (Stahre, 2006; Armitage *et al.,* 2013). Retention ponds are not common as they provide limited flood control measure, an essential requirement in conventional stormwater management. Retention ponds require regular maintenance to avoid public health risks from pollution build-up, the potential risk of people drowning, mosquitos breeding and reeds covering the entire pond (Armitage *et al.,* 2013). Well maintained retention ponds can also offer additional benefits such as ambience and affluence to an area providing a sense of serenity and good living (Haddock, 2004). There is evidence that a well-maintained pond system can provide an economic benefit by increasing the selling price of nearby properties by 10% to 25% (US EPA, 1995; Dinovo, 1995). Another advantage of retention ponds is that the permanent pond may be utilised as a source of water for various non-potable purposes (Armitage *et al.,* 2013). Conversely, a poorly maintained retention pond would be characterised with litter and solid waste, potential breeding ground for mosquitos and can result in a health hazard for nearby communities. Since retention ponds typically require a permanent pool of water, they cannot be used in arid regions with high evaporation rates and limited rainfall (Armitage *et al.,* 2013).

A constructed wetland is typically characterised with marshy shallow water with partially or completely covered in aquatic vegetation and provides more stormwater quality improvement than peak flow attenuations from storm events to mitigate flood risk downstream of the ponds (Woods-Ballard *et al.*, 2015; Armitage *et al.*, 2013). Constructed wetlands also provide a vibrant habitat for fish, birds and other wildlife, potentially offering a sanctuary for rare and endangered species (Armitage *et al.*, 2013). Although constructed wetlands offer much lower flood control measures than detention and retention ponds, the opportunity to improve ecosystem health, aesthetic appeal that mimics natural systems make them attractive to property owners (Armitage *et al.*, 2013). As expected, constructed wetlands are not common as they provide limited flood control measure, a key requirement in conventional stormwater management. The water quality improvement function in the constructed wetland is typically characterised by processes such as sedimentation, fine particle filtration and biological nutrient and removal of some pathogen (Field & Sullivan, 2003; Parkinson & Mark, 2005; Armitage *et al.,* 2013).

Appendix 8: Stormwater harvesting from surface water storage

Appendix 8a: Surface water restricted to the Zeekoe Catchment

In this approach referred to as the 'decentralised system', the harvested stormwater would be restricted to locations in the Zeekoe Catchment. The harvested stormwater would be treated to potable or non-potable water standards, distributed via existing municipal system or dual reticulation *i.e.* 'third pipe' which is colour coded and secured with locks to minimise health risks and used for selected applications such as toilet flushing, irrigation of residential gardens, open parks, and urban agriculture. The abstraction was simulated from the two most downstream vleis (Zeekoevlei and Rondevlei) and distribution in the study area was modelled in EPANET2 integrated with PCSWMM with a network as shown in the Figure 8a and the main transmission pipeline design with fitting in Table 8b.

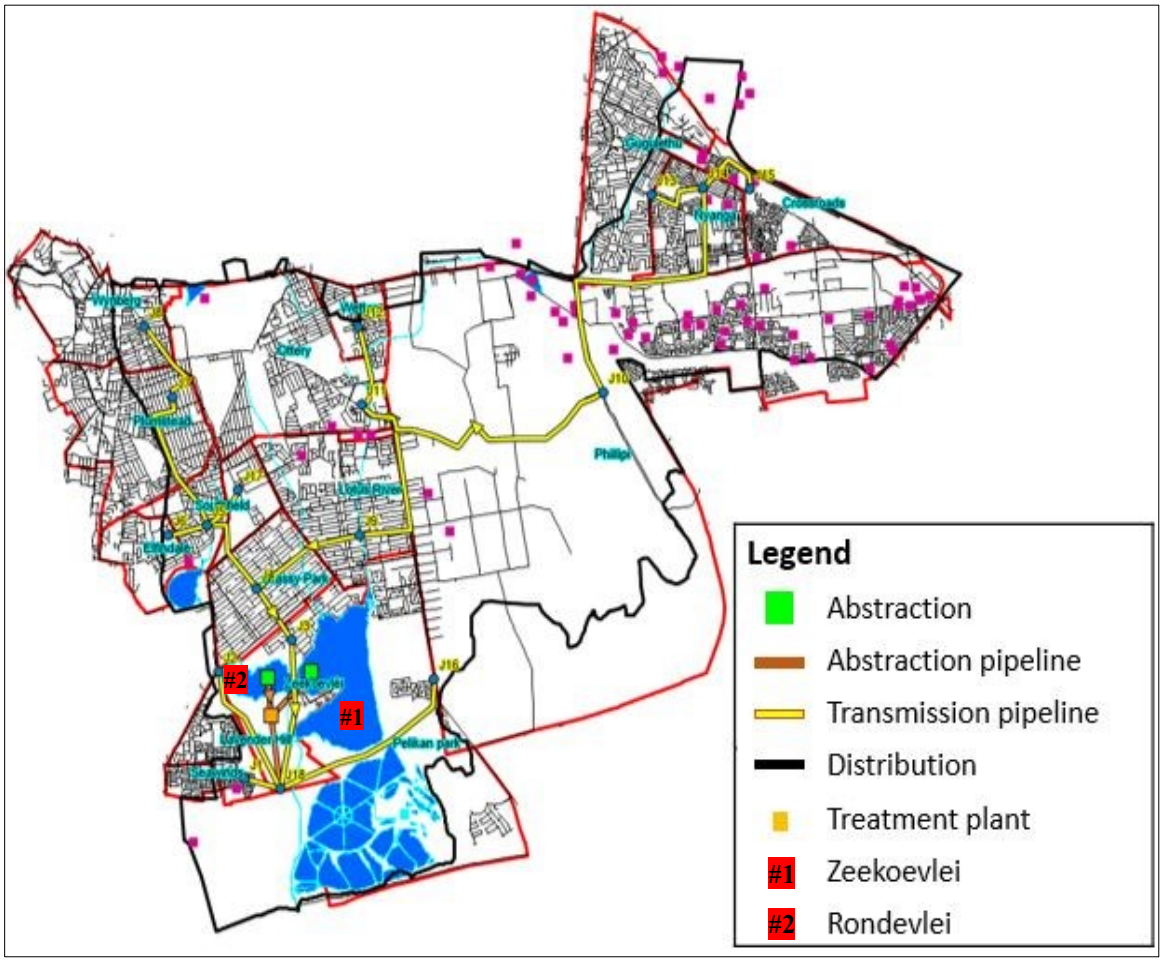


Figure 8a Stormwater harvesting and distribution network

Appendix 8b: Transmission from source to reservoir

Parameter	Values
Maximum Day Demand (m^3/day)	43,200
Total Pumping Main Output	43,200
Hours of Pumping (hr)	24.0
Delivery (m^3/hr)	1,800
Delivery (m^3/s)	0.5
Pumping Main	
Water Level at WTP (m AMSL)	40.0
Water Level at source (m AMSL)	7.0
Static Lift (m)	33.0
Cwh	120.0
Pipe Details	DN 600 DI PN 16
Pipe Diameter ND (mm)	600
Pipe Diameter ND (m)	0.60
Flow in Pipe (m^3/s)	0.50
Velocity (m/s)	1.77
Chainage at WTP (m)	16+550
Chainage at source (m)	0+000
Length of Pipe (m)	16,550
Friction Loss (m)	83.03
Fittings losses - 10% (m)	8.30
Total Head (m)	124
Capacity of each pump	
Head (m)	**124**
Flow (m^3/hr)	**0.50**

Appendix 8c: Transmission from source to water treatment plant

The other option assessed was abstraction from the two vleis *i.e.* Zeekoevlei and Rondevlei labelled #1 and #2 respectively in Figure 8c, pre-treated at a new proposed WTP and conveyed to an existing WTP as shown in Figure 8c and the main transmission pipeline design is given in Table 8d.

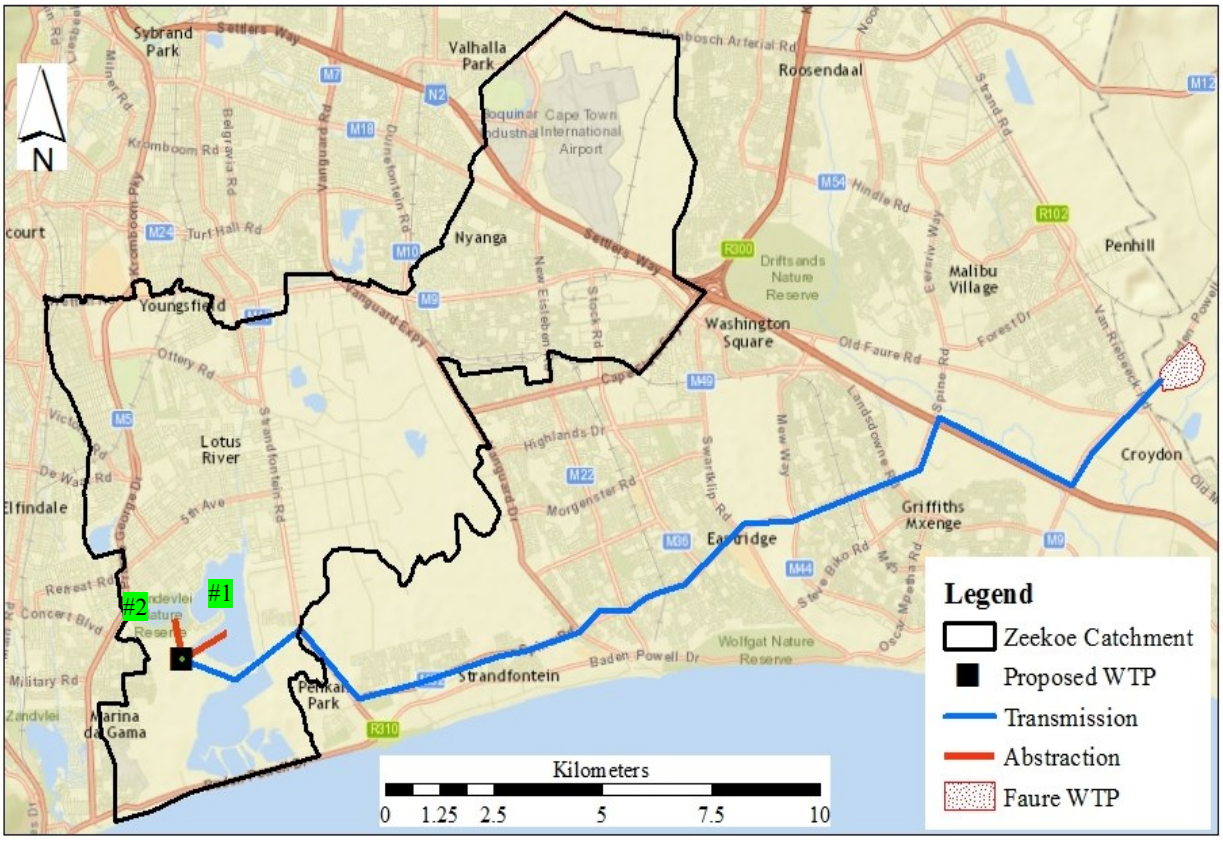


Figure 8c Centralised system with abstraction and conveyance

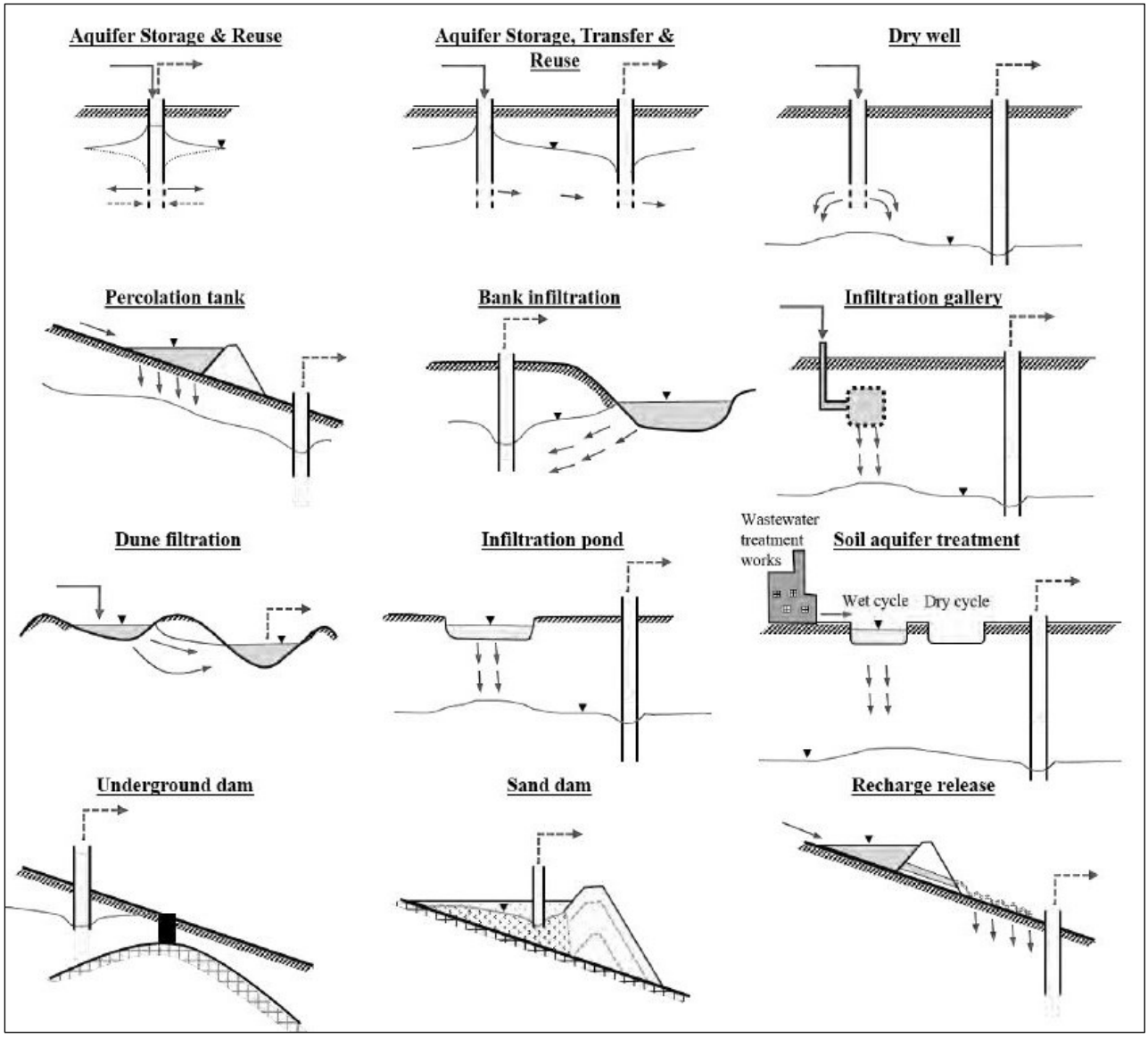
Appendix 8d: Transmission from source to existing water treatment plant

Parameter	Values
Maximum Day Demand (m^3/day)	43,200
Total Pumping Main Output	43,200
Hours of Pumping (hr)	24
Delivery (m^3/hr)	1,800
Delivery (m^3/s)	0.50
Pumping Main	
Water Level at WTP (m AMSL)	68.0
Water Level at source (m AMSL)	7.0
Static Lift (m)	61.0
Cwh	120
Pipe Details	**DN600 DI PN 25**
Pipe Diameter ND (mm)	600.00
Pipe Diameter ND (m)	0.60
Flow in Pipe (m^3/s)	0.50
Velocity (m/s)	1.77
Chainage at WTP (m)	30+760.0
Chainage at source (m)	0+000.0
Length of Pipe (m)	**30,760.0**
Friction Loss (m)	154.3
Fittings losses - 10% (m)	15.4
Total Head (m)	**231**
Capacity of each pump	
Head (m)	231
Flow (m^3/hr)	0.50

Appendix 9: Stormwater harvesting from groundwater storage

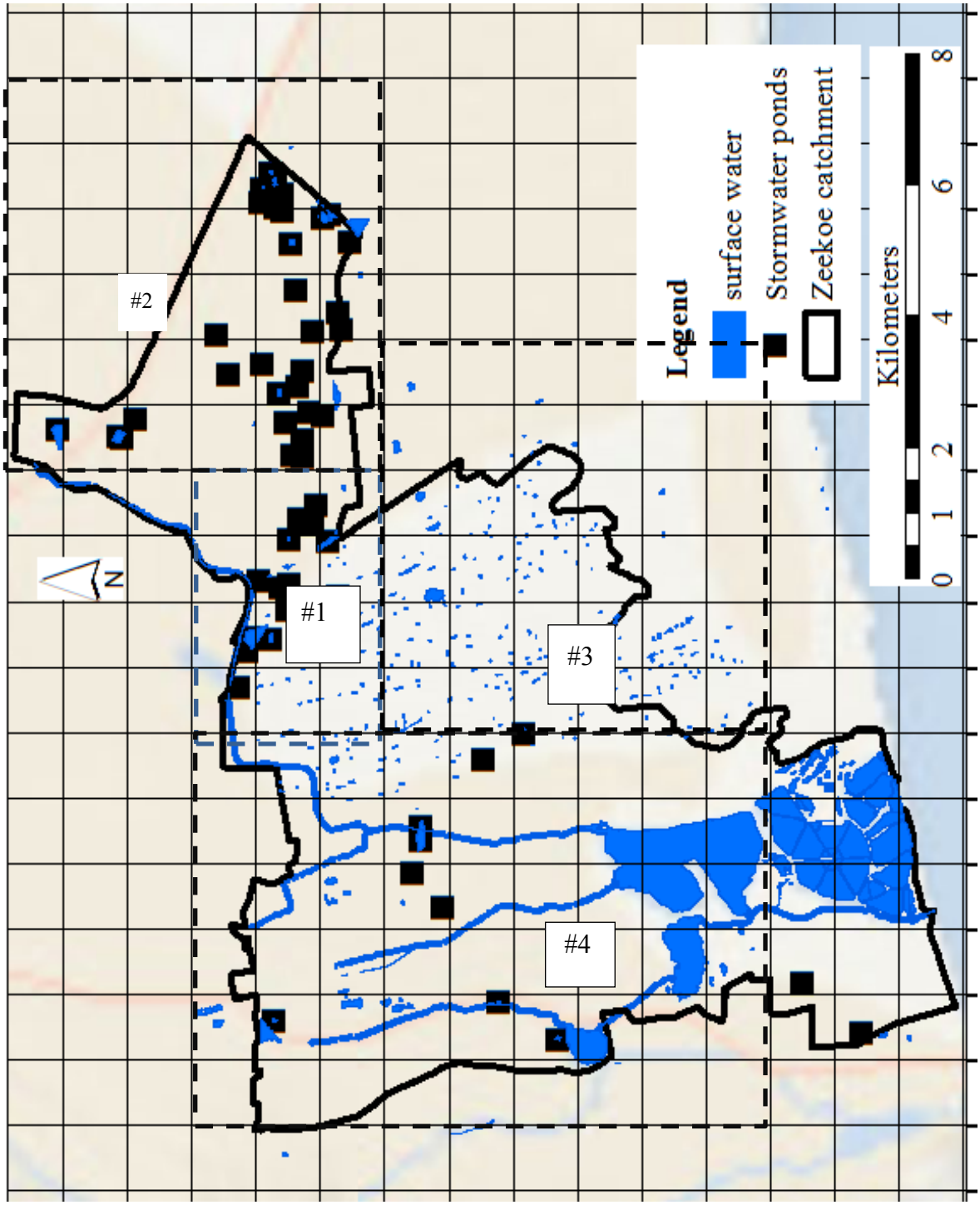
Appendix 9a: Overview of stormwater harvesting from groundwater storage

Managed Aquifer Recharge and Recovery (MAR&R) is a process where surface water *e.g.* stormwater or wastewater is temporarily stored in excavated depressions in the earth surface with specific configuration to allow recharging groundwater aquifers for future use or for environmental benefits (Dillon *et al.,* 2009; Wu *et al.,* 2012; Fisher-Jeffes, 2015).



Recharge of the groundwater aquifer can be accomplished through direct injection of surface water to underground dams through various MAR&R approaches as shown (Dillon *et al.,* 2009; Wu *et al.,* 2012; Fisher-Jeffes, 2015). The main aim of the transfer of surface water to groundwater aquifers is to make use of large storage capacity and to benefit from the limited loss from evaporation (Philp *et al.,* 2008). The various treatment process associated with MAR *i.e.* extended retention in the depressions and filtration in the groundwater aquifer provides some level of stormwater quality improvement (Dillon *et al.,* 2009; Wu *et al.,* 2012). Further, the process of stormwater harvesting results in a reduction of the runoff component in the hydrological cycle water balance (*i.e.* infiltration component is increased), thus providing additional peak flow attenuation from storm events to mitigate flood risk (Fisher-Jeffes, 2015). Although MAR&R can provide significant water quality improvement and water quantity management (both flood control and water supply), implementation usually depends on land availability, topography (generally flat) and geology (suitable aquifer and porous sandy soils) (Wu *et al.,* 2012; Fisher-Jeffes, 2015).

Appendix 9b – A discretised section of Zeekoe Catchment for groundwater modelling in MATLab (*After* CCT 2015)



Appendix 9c – MATLab code for modelling groundwater flow

```
% solves groundwater flow equation in two dimensions with an optional heterogeneous
hydraulic conductivity
% generates flow field for solute transport solver gwtrans2d
function gwflow2d
tic;
% size of domain (Lx x Ly)
Lx=4000; Ly=3000;
% number of grid points (nx x ny)
nx=201; ny=201;
% hydraulic conductivity mean (mu) and variance (sigma)
sigma=0.5; mu=1.667;
% Generate lognormally distributed
% Heterogeneous Hydraulic conductivity K
% K=mu*exp(sigma*randn(nx,ny));
K=mu*ones(nx,ny); % homogeneous K
% Transmittivity in x and y directions Tx, Ty
thickness=30; % thickness of aquifer
Tx=K*thickness;
Ty=Tx;
% Boundary conditions on the 4 boundaries
% internal dirichlet bc's specified in 'assign_nodel_dirichlet'
bctype=zeros(4,1);
bcval=zeros(4,1);
% specify boundary conditions
% left
bctype(1)=1;
bcval(1)=24;
% bottom
bctype(2)=1;
bcval(2)=23;
% right
bctype(3)=1;
bcval(3)=29;
% top
bctype(4)=1;
bcval(4)=29;
% specify well conditions
flux_flag = 1; % set to 1 if flux needs to be saved for transport
% nw=3; number of wells
% q=zeros(nw,1); % pumping rate at wells (+ve) injection
% xw=zeros(nw,1); % well x coordinates
% yw=zeros(nw,1); % well y coordinates
% q(1) = -1000; xw(1) = 500; yw(1) = 400;
% q(2) = 1000; xw(2) = 550; yw(2) = 650;
% q(3) = -300; xw(3) = 850; yw(3) = 350;
```

```
% you can also use a function to assign wells
[q, xw, yw, nw] = assign_wells;
dx=Lx/(nx-1);
dy=Ly/(ny-1);
n = nx*ny;
% generate matrix and solve for hydraulic head
head=gwflow2d_solve(n,nx,ny,Lx,Ly,Tx,Ty,dx,dy,q,xw,yw,nw,bctype,bcval);
% plot
gwhead_plot(nx,ny,Lx,Ly,dx,dy,head);
save('hydraulics.mat','head','Tx','thickness');
save('well_cords.mat','xw','yw');
save('domain.mat','Lx','Ly','nx','ny');
if (flux_flag == 1 || nw > 0)
   %     flux = node_flux(n,nx,ny,head,Tx,Ty,dx,dy,bctype,bcval);
   for k=1:nw
      iw(k)=round(xw(k)/dx+1);
      jw(k)=round(yw(k)/dy+1);
   end
   save('well_info.mat','nw','iw','jw','q','thickness');
end
toc;
return
```

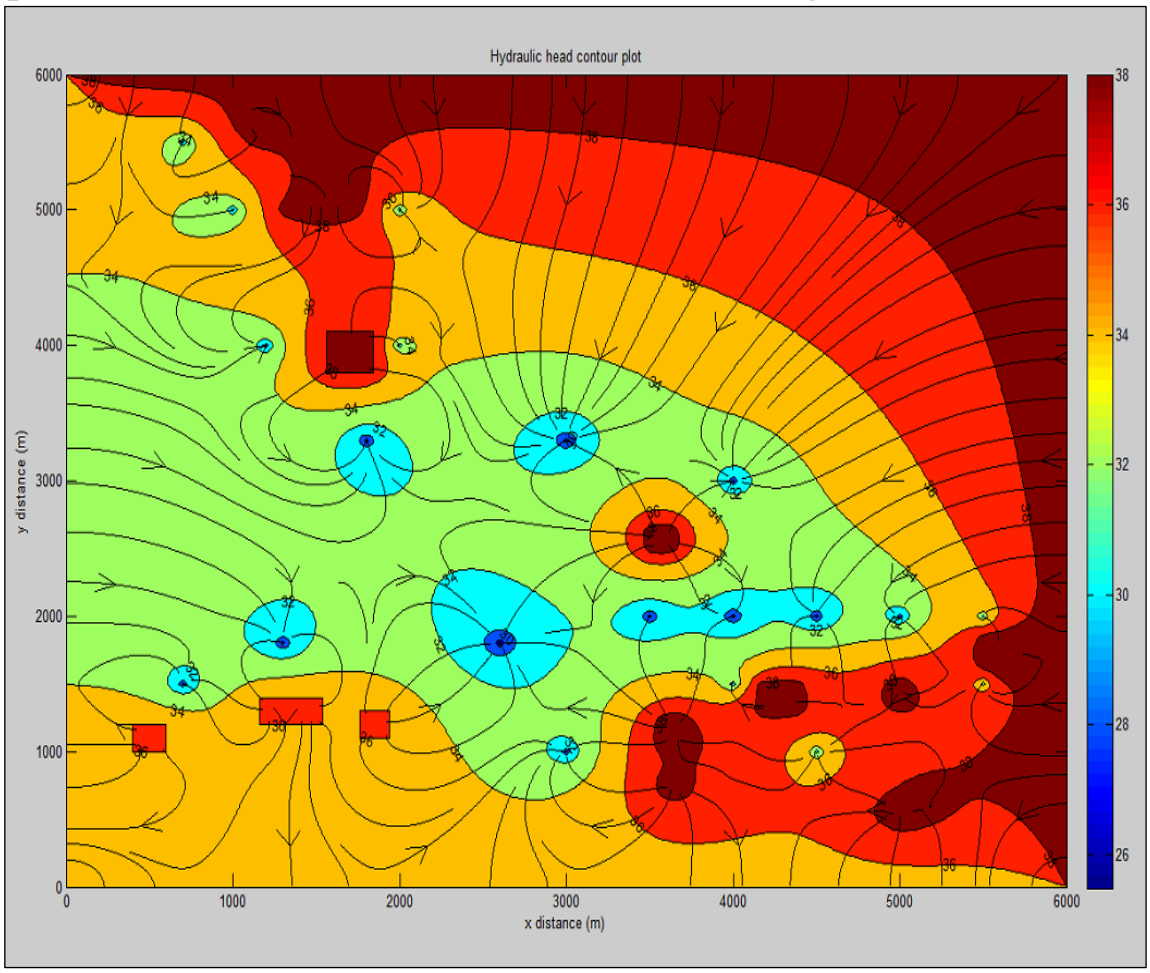
Appendix 9c – MATLab code for modelling pollution transport and generation of breakthrough curves

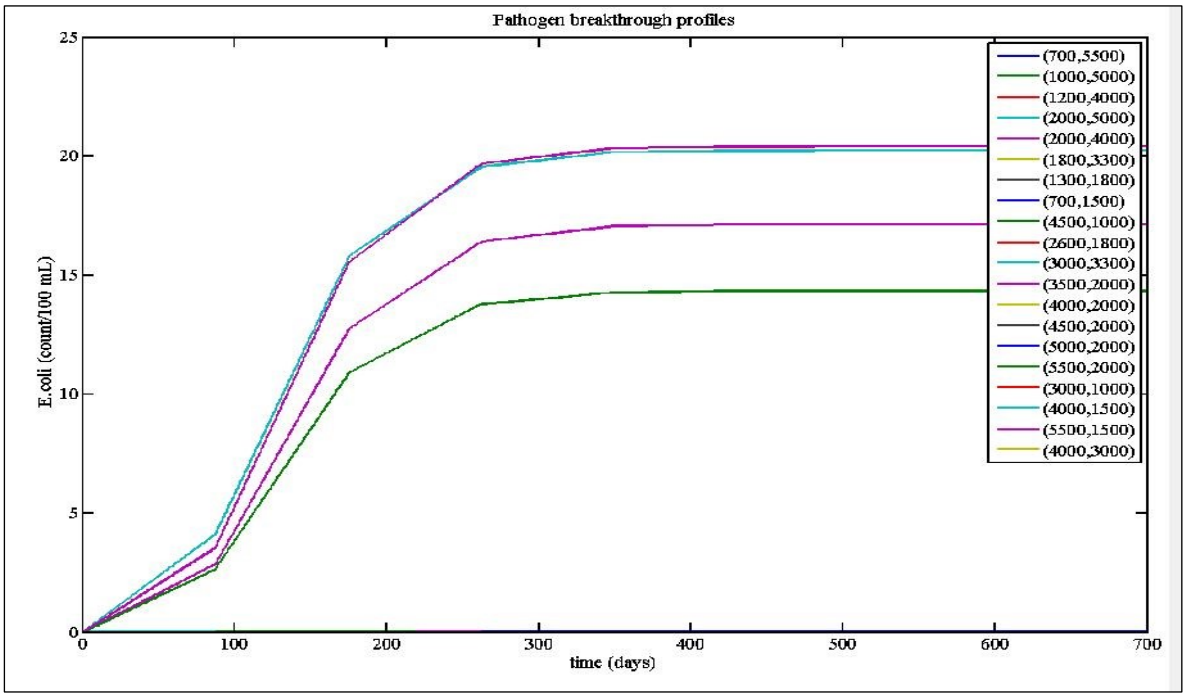
```
% solves the 2d dimensional solute transport problem
% uses the flow field generated by gwflow2d
function gwtrans2d
tic;
% domain size
Lx=1000; Ly=1000;
% grid resolution (number of grid points in x and y)
nx=51; ny=51;
% time parameters
T=1000; % total time duration (days)
nt=10000; % number of time steps
R=1; % retardation factor
lambda=0.01; % decay rate
% dispersion parameters (defualt values)
% alphaL = longitudinal dispersivity (m)
% alphaT = transverse dispersivity (m)
% Dm = molecular dispersivity (m2/d)
alphaL=40; alphaT=10; Dm=0.01;
% porosity
porosity = 0.36; % default value
% solution returned at time steps specified by isol
isol = (0:100:nt);
% set concentration to zero at all boundaries
% ok if boundaries are further away from all wells and sources
bctype(1:4)=1; bcval(1:4)=0;
% bctype(1)=2; bctype(3)=2;
dx=Lx/(nx-1);
dy=Ly/(ny-1);
dt=T/nt;
% calculate velocity from head written by flow code
[vx,vy]=velocity(dx,dy,nx,ny, porosity);
% solve for concentration and output solution at isol time steps
[csol] = gwtrans2d_solve(Lx, Ly, bctype, bcval, T, nx, ny, nt, isol, vx, vy, ...
   alphaL, alphaT, Dm, R, lambda);
% plot solution contour at final time step
gwconc_plot(nx,ny,Lx,Ly,dx,dy,csol,T);
% plot solution profile at [xp,yp]
load well_cords xw yw;
xp=xw(1); yp=yw(1);
tsol=isol*dt;
gwconc_profile_plot(csol,tsol,dx,dy,nx,ny,xp,yp);
% write entire output concentrations in excel
% xlswrite('conc.xlsx',[tsol',csol']);
```

```
toc;
return
%%%%%%%
function [nodal_dirich]=assign_nodal_dirichlet(dx,dy,nodal_dirich)
% specify nodal dirichlet conditions by hardcoding
% this will overwrite all other boundary conditions
% below is an example based on ABC site
% single rectangular patch
x1=450; x2=550; y1=450; y2=550; hvalue=25;
ix1=1+round(x1/dx);
ix2=1+round(x2/dx);
iy1=1+round(y1/dy);
iy2=1+round(y2/dy);
for i=ix1:ix2
   for j=iy1:iy2
      nodal_dirich.type(i,j)=1;
      nodal_dirich.val(i,j)=hvalue;
   end
end
return

% assign wells symmetrically around a centroid
function [q, xw, yw, nw] = assign_wells
% center of circle (x0,y0) radius (R)
% number of wells (nw)
% total extraction rate q0 (m3/d)
x0 = 500; y0 = 500; R = 200; nw=8; q0=3200;
% calculate angle between wells
dtheta = 2*pi/nw;
xw=zeros(nw,1);
yw=zeros(nw,1);
q = zeros(nw,1);
theta=0;
xw(1)=x0;
yw(1)=y0;
for i = 1:nw
   xw(i) = x0+R*cos(theta);
   yw(i) = y0+R*sin(theta);
   q(i) = -q0/nw; % extraction wells
   theta=theta+dtheta;
end
% plot(xw,yw,'ro');
% xlim([0 10000]);
% ylim([0 10000]);
Return

% plot concentration contour at the end of simulation
```

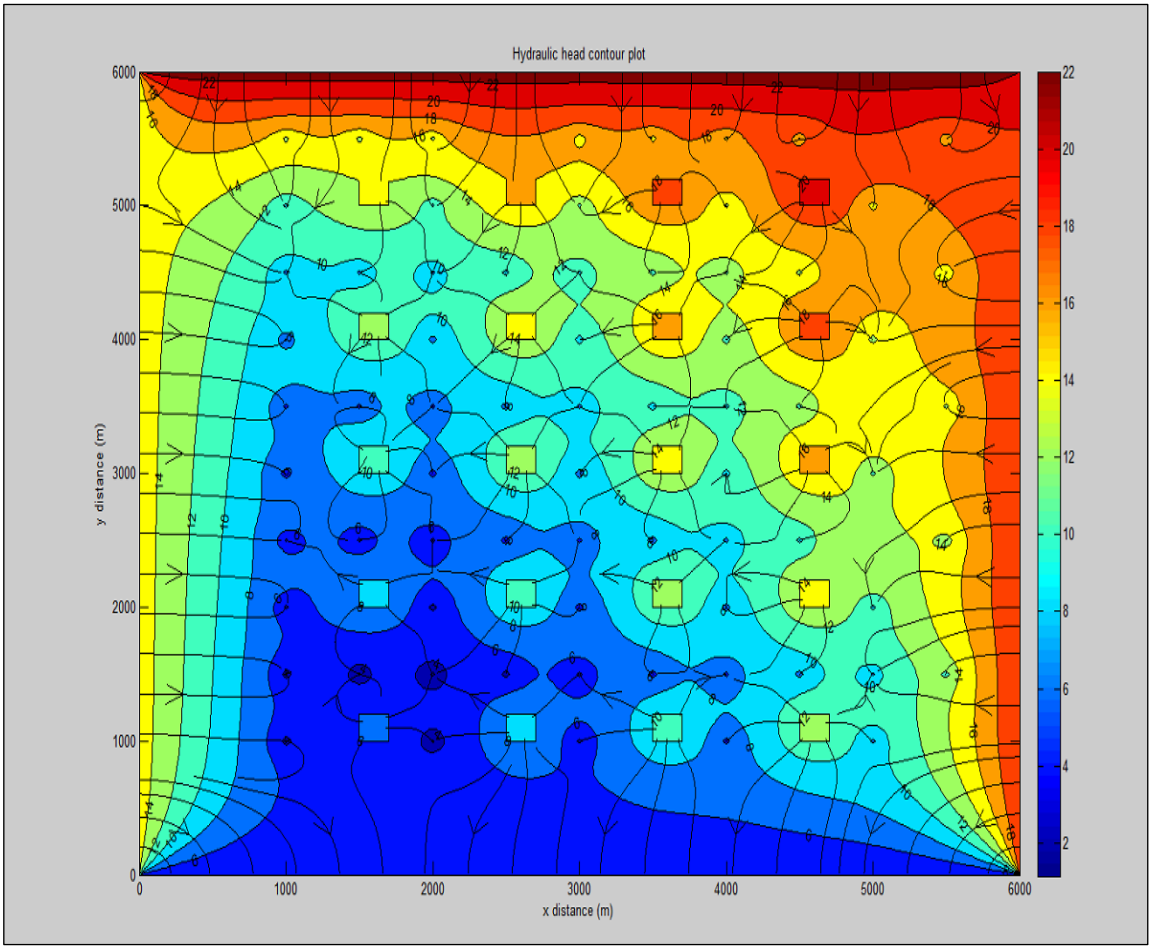
```
function gwconc_plot(nx,ny,Lx,Ly,dx,dy,csol,T)
x=0:dx:Lx; y=0:dy:Ly;
[X,Y]=meshgrid(x,y);
% plot
ij=0;
x=zeros(nx,ny);
y=zeros(nx,ny);
z=zeros(nx,ny);
for i=1:nx
   for j=1:ny
      ij=ij+1;
      z(i,j)=csol(ij,end);
   end
end
[C, h] = contourf(X,Y,z');
clabel(C,h);
colorbar;
xlabel('x distance (m)');
ylabel('y distance (m)');
title(['Concentration (mg/L) at ', num2str(T),' days']);
hold off;
return

function gwconc_profile_plot(csol,tsol,dx,dy,nx,ny,xp,yp)
ixp=1+round(xp/dx);
iyp=1+round(yp/dy);
cp = csol(ind(ny,ixp,iyp),:);
plot(tsol,cp,'-r');
xlabel('time (days)');
ylabel('concentration (mg/L)');
title(['Concentration profile at (',num2str(xp),',',num2str(yp),') ']);
end
```

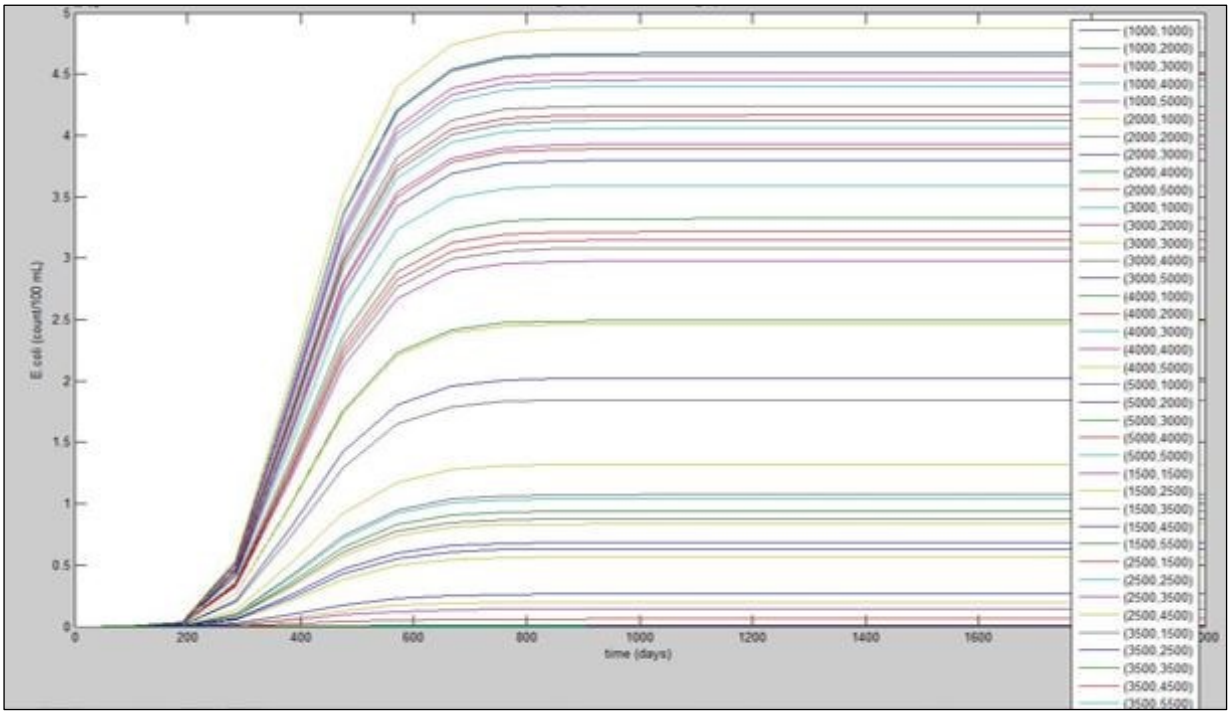
Appendix 9c – Location of boreholes and breakthrough curves for Section #2



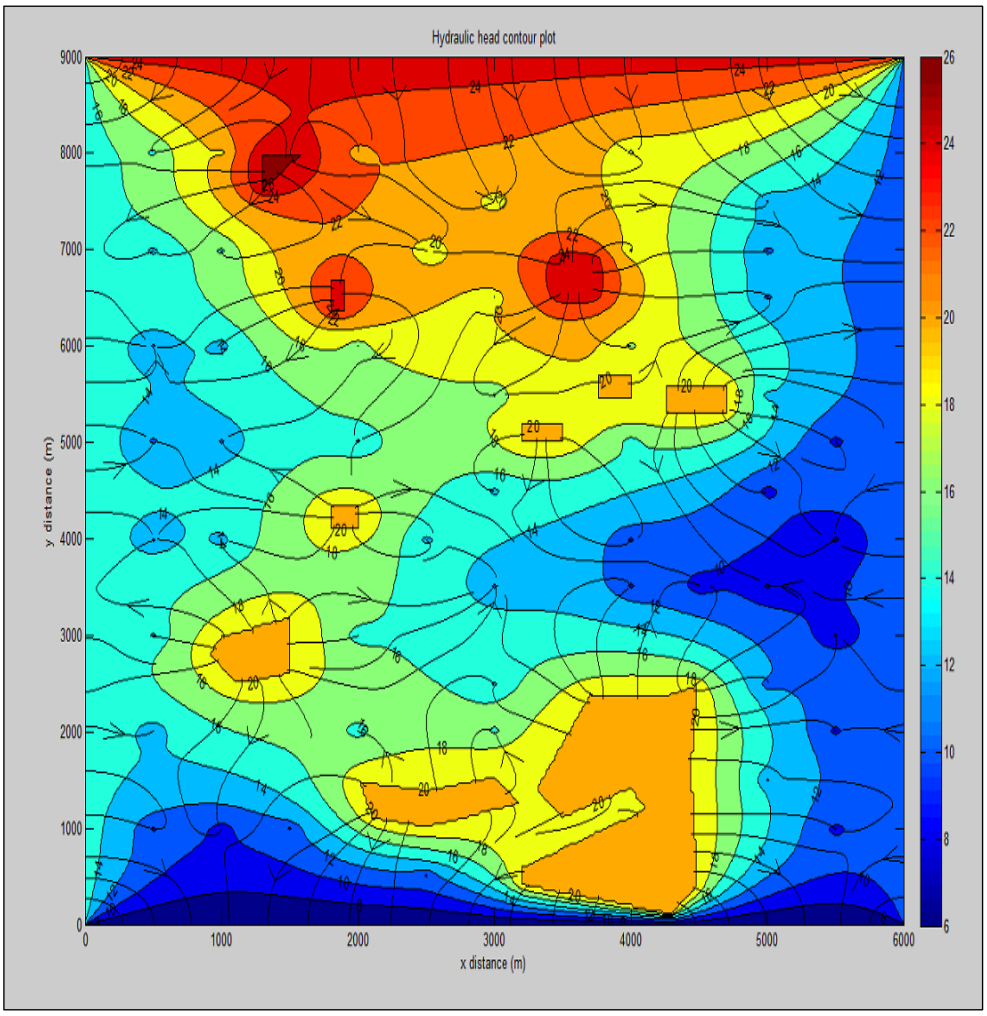


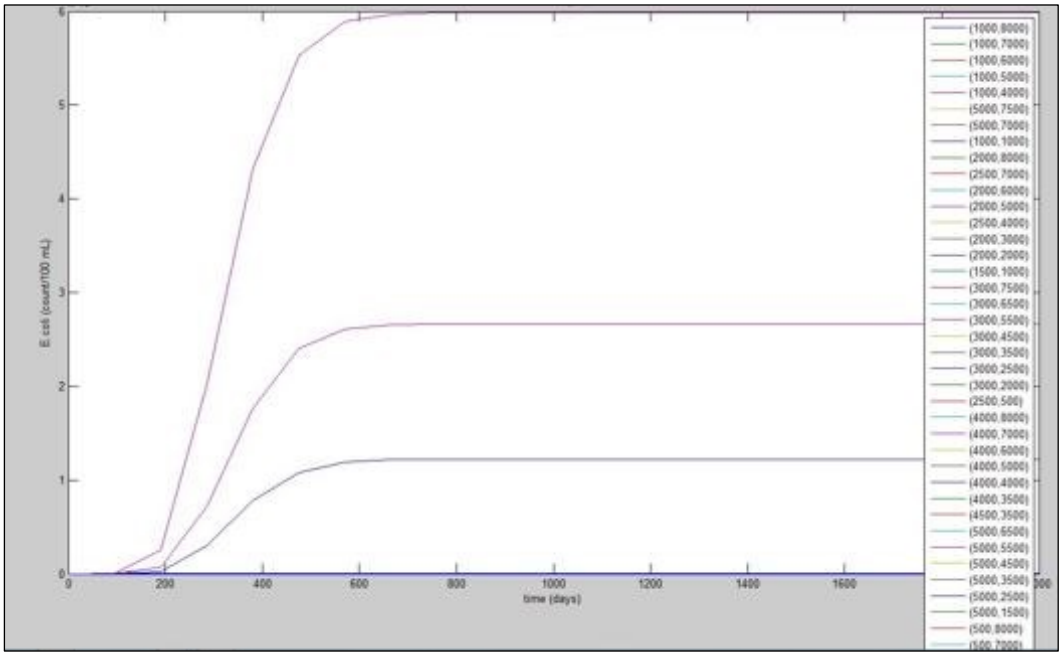
Appendix 9d – Location of boreholes and breakthrough curves for Section #3





Appendix 9e – Location of boreholes and breakthrough curves for Section #4

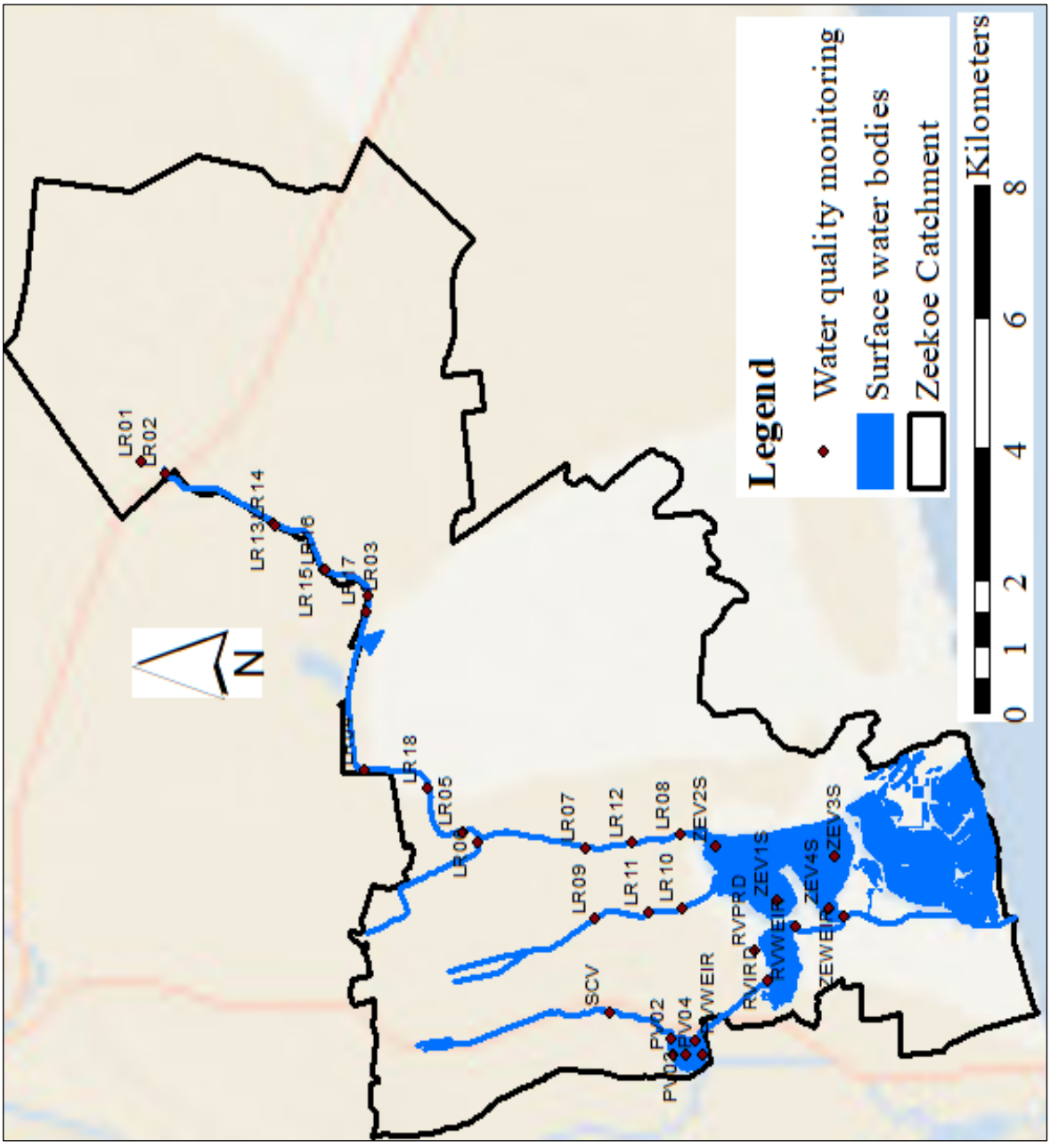




Appendix 10 – Water quality monitoring and parameters measured in the study area

Water quality monitoring is undertaken in the study area for various parameters on a monthly basis at several locations.

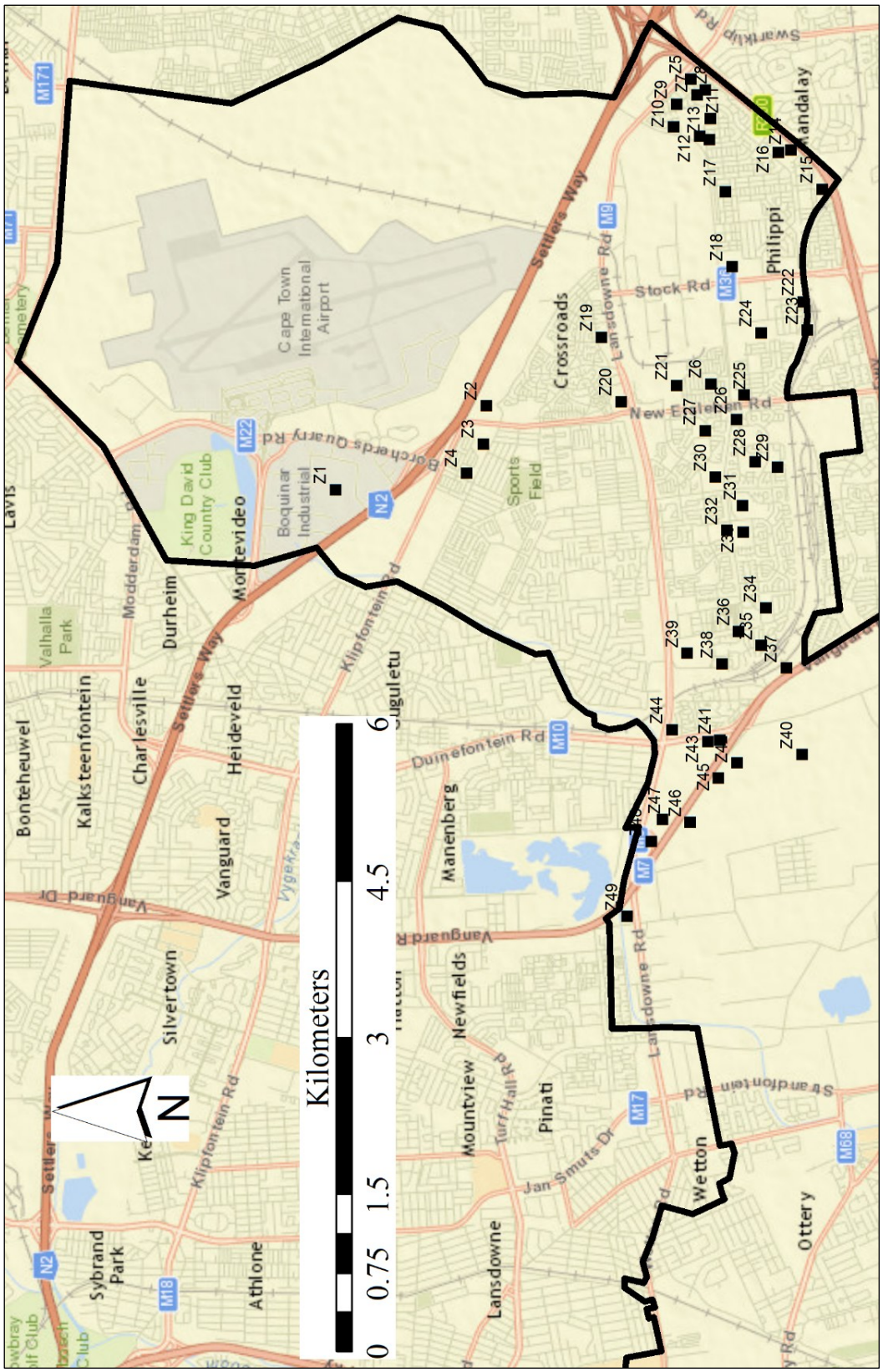
Appendix 10a – Water quality monitoring stations in the study area

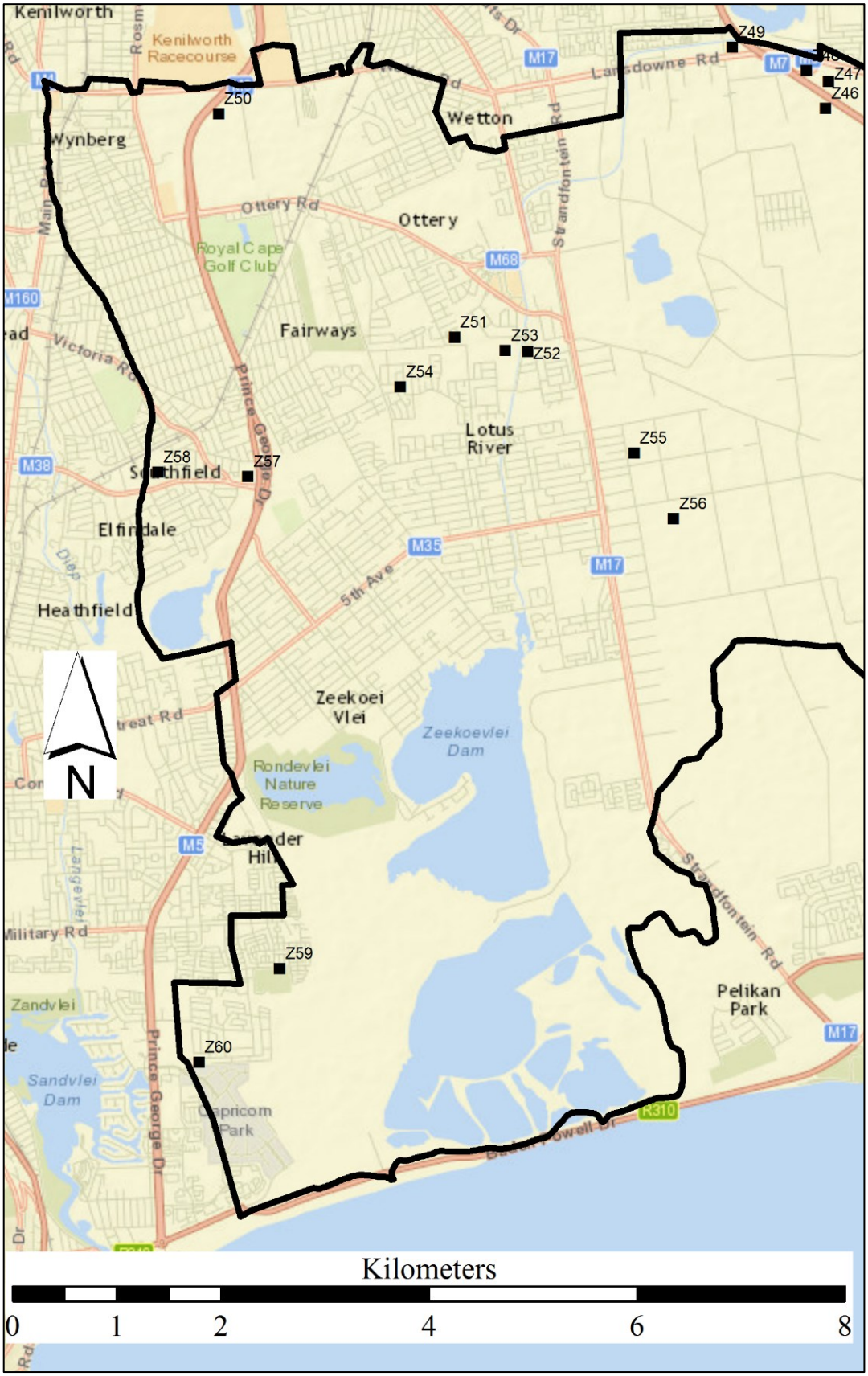


Appendix 10b – Monitored water quality parameters

	Monitoring	Description	Code
Big Lotus	Chemical / Bacteriological	Lotus River on Airport Approach Road opposite Borcherd's Quarry final effluent ponds	LR01
		Lotus River on Settler's Way (N2) about 500m from Airport Approach Road	LR02
	Bacteriological	NY3A u/s stormwater outlet	LR13
		NY3A d/s stormwater outlet	LR14
		NY3 u/s stormwater outlet	LR15
		NY3 d/s stormwater outlet	LR16
		Lansdowne Road opposite Sherwood Park	LR17
	Chemical / Bacteriological	Lotus River at corner Duinefontein and Lansdowne Roads	LR03
		Lotus River at Lansdowne Road	LR04
	Bacteriological	Lotus River at Springfield Rd Turfhill Estate	LR18
	Chemical / Bacteriological	Lotus River at Plantation Road (near Hillstar Traffic Department)	LR05
		Lotus River at New Ottery Road (near Ottery Hypermarket)	LR06
		Lotus River at Klip Road	LR07
		Lotus River at Fifth Avenue - Grassy Park	LR12
		Lotus River at Fisherman's Walk bridge (just u/s of vlei)	LR08
	Chemical / Bacteriological/ Algalogical	Opposite inlet of Big Lotus River	ZEV2S
		Home Bay in front of Zeekoevlei Yacht Club	ZEV1S
		In front of Cape Peninsula Aquatic Club	ZEV3S
		SW corner approx. 200m from the weir	ZEV4S
Little Lotus	Chemical / Bacteriological	Little Lotus River at Klip Road (near Montagues Gift Road)	LR09
		Little Lotus River at Fifth Avenue Grassy Park	LR11
		Little Lotus River at Eighth Avenue	LR10
Princessvlei	Chemical / Bacteriological/ Algalogical	Princessvlei - vlei inlet	PV01
		Princessvlei - centre	PV03
		Princessvlei - north	PV02
		Princessvlei - south	PV04
		Princessvlei near outlet weir	PVWEIR
	Chemical / Bacteriological	Southfield Canal at Victoria Road	SCV
Rondevlei	Chemical / Bacteriological	Italian Rd canal leading to Rondevlei	RVIRD
	Chemical / Bacteriological/Algalogical	Rondevlei Weir	RVWEIR
	Chemical / Bacteriological	Perth Rd canal leading to Rondevlei	RVPRD
Zeekoevlei	Chemical / Bacteriological/Algalogical	Vlei sample at Zeekoevlei - inlet	ZEV2S
		Vlei sample at Zeekoevlei - centre	ZEV1S
		Vlei sample at Zeekoevlei weir - outlet	ZEWEIR

Appendix 11 – Location of stormwater ponds in the study area




Kenilworth
Kenilworth Racecourse
Wynberg
Wetton
Ottery
Ottery Rd
Royal Cape Golf Club
Fairways
Lotus River
Elfindale
Heathfield
Zeekoei Vlei
Zeekoevlei Dam
Rondevlei Nature Reserve
Pelikan Park
Zandvlei
Sandvlei Dam
Capricorn Park
Lansdowne Rd
Strandfontein Rd
Prince George Dr
Victoria Rd
5th Ave
Z46
Z47
Z49
Z50
Z51
Z52
Z53
Z54
Z55
Z56
Z57
Z58
Z59
Z60
M17
M68
M35
M38
M5
M7
M160
R310
N
Kilometers
0 1 2 4 6 8

Appendix 11c – Location of stormwater ponds in the study area

ID	Pond Type	Road	Suburb	Erf No.	Latitude	Longitude
Z1	Retention	Mobile Street	Boquinar Industrial	00-112706	-33° 58' 28.42"	18° 35' 5.06"
Z2	Detention	Owen Drive	Crossroads	27-2849-1	-33° 58' 59.26"	18° 35' 0.84"
Z3	Detention	Owen Drive	Crossroads	27-2849-1	-33° 59' 1.41"	18° 34' 59.78"
Z4	Detention	Ntlangano Crescent	Crossroads	27-14240	-33° 59' 8.17"	18° 35' 11.2"
Z5	Detention	Nyamakazi Street	Philippi	55-5568	-34° 0' 16.62"	18° 37' 37.69"
Z6	Detention	Nyamakazi Street	Philippi	55-5568	-34° 0' 16.62"	18° 37' 37.69"
Z7	Detention	Sangoma Street	Philippi	55-5550	-34° 0' 18.53"	18° 37' 31.88"
Z8	Detention	Sangoma Street	Philippi	55-5550	-34° 0' 18.57"	18° 37' 31.69"
Z9	Detention	Sangoma Street	Philippi	55-5442	-34° 0' 12.23"	18° 37' 28.36"
Z10	Detention	Indwe Street	Philippi	55-131216	-34° 0' 11.3"	18° 37' 20.02"
Z11	Detention	Ngqwangi Drive	Philippi	55-8092	-34° 0' 22.18"	18° 37' 24.63"
Z12	Detention	Metlane Close	Philippi	Ca597-15	-34° 0' 19.18"	18° 37' 16.57"
Z13	Detention	Gamtriya Road	Philippi	55-5623	-34° 0' 22.08"	18° 37' 15.27"
Z14	Detention	R300	Philippi	55-5620	-34° 0' 46.1"	18° 37' 12.31"
Z15	Detention	Mvundla Crescent	Philippi	55-5630	-34° 0' 56.28"	18° 36' 56.57"
Z16	Detention	Feljisi Road	Philippi	55-5620	-34° 0' 43.04"	18° 37' 10.37"
Z17	Detention	Ngwamza Walk	Philippi	55-5616	-34° 0' 26.94"	18° 36' 55.69"
Z18	Detention	Sheffield Road	Philippi	55-3377	-34° 0' 28.84"	18° 36' 27.77"
Z19	Detention	Gwayi Street	Crossroads	39-1	-33° 59' 49.23"	18° 36' 1.63"
Z20	Detention	New Eisleben Road	Crossroads	39-50	-33° 59' 55.06"	18° 35' 37.58"

Appendix 11c – Location of stormwater ponds in the study area

ID	Pond Type	Road	Suburb	Erf No.	Latitude	Longitude
Z21	Detention	Cwangco Crescent	Philippi	55-12719	-34° 0' 11.97"	18° 35' 43.41"
Z22	Detention	Stock Road	Philippi	Ca693-9	-34° 0' 50.46"	18° 36' 14.58"
Z23	Detention	Acacia Street	Philippi	Ca693-9	-34° 0' 51.58"	18° 36' 4.03"
Z24	Detention	Informal Road	Philippi	55-5267	-34° 0' 37.62"	18° 36' 3.06"
Z25	Detention	New Eisleben Road	Philippi	55-5624	-34° 0' 32.4"	18° 35' 39.85"
Z26	Detention	Sagwityi Street	Philippi	55-1997	-34° 0' 30.02"	18° 35' 30.77"
Z27	Detention	Sagoloda Street	Philippi	55-664	-34° 0' 20.52"	18° 35' 26.49"
Z28	Detention	Sagwityi Street	Philippi	55-1552	-34° 0' 35.67"	18° 35' 14.9"
Z29	Detention	Nowanga Street	Philippi	55-1854	-34° 0' 42.4"	18° 35' 12.95"
Z30	Detention	Sikhwenene Street	Philippi	55-956	-34° 0' 23.59"	18° 35' 9.28"
Z31	Detention	Mbomvane Street	Philippi	55-2424	-34° 0' 31.73"	18° 34' 58.53"
Z32	Detention	Sheffield Road	Philippi	55-3366	-34° 0' 26.95"	18° 34' 49.47"
Z33	Detention	Msingizane Street	Philippi	55-2309	-34° 0' 31.88"	18° 34' 48.61"
Z34	Retention	Mdubi Street	Philippi	55-4208	-34° 0' 38.62"	18° 34' 20.35"
Z35	Detention	Tamani Road	Philippi	55-4158	-34° 0' 36.88"	18° 34' 8.55"
Z36	Detention	Sheffield Road	Philippi	55-3157	-34° 0' 30.48"	18° 34' 11.71"
Z37	Detention	Dora Tamana	Philippi	40-3305	-34° 0' 44.85"	18° 33' 58"
Z38	Detention	Govan Mbeki Road	Philippi	Ca604-28	-34° 0' 25.36"	18° 33' 59.55"
Z39	Detention	Duinefontein Road	Philippi	Ca609-6	-34° 0' 25.15"	18° 33' 32.46"
Z40	Retention	Weltevreden Road	Philippi	Ca609-9	-34° 0' 49.51"	18° 33' 25.7"

Appendix 11c – Location of stormwater ponds in the study area

ID	Pond Type	Road	Suburb	Erf No.	Latitude	Longitude
Z41	Retention	Duinefontein Road	Philippi	Ca609-6	-34° 0' 24.63"	18° 33' 31.17"
Z42	Detention	Old Lansdowne Road	Philippi	Ca609-11	-34° 0' 29.72"	18° 33' 22.7"
Z43	Detention	Duinefontein Road	Philippi	Ca609-6	-34° 0' 20.89"	18° 33' 30.52"
Z44	Detention	Lansdowne Road	Philippi	Ca609-4	-34° 0' 10.04"	18° 33' 35.11"
Z45	Detention	Old Lansdowne Road	Philippi	Ca609-12	-34° 0' 24.14"	18° 33' 17.06"
Z46	Retention	Old Lansdowne Road	Philippi	Ca609-86	-34° 0' 15.49"	18° 33' 0.52"
Z47	Retention	Govan Mbeki Road	Philippi	Ca609-84	-34° 0' 7.1"	18° 33' 1.67"
Z48	Detention	Vanguard Drive	Philippi	00-40308-1	-34° 0' 3.65"	18° 32' 53.44"
Z49	Detention	Lansdowne Road	Philippi	00-159596	-33° 59' 59.47"	18° 32' 31.76"
Z50	Retention	Kromboom Parkway	Ottery	00-90477	-34° 0' 16.25"	18° 29' 13.08"
Z51	Detention	Plumbago Close	Ottery	14-4326	-34° 1' 26.41"	18° 30' 41.06"
Z52	Retention	Eric Way	Philippi	14-3373	-34° 1' 30.99"	18° 31' 8.4"
Z53	Retention	Clifford Street	Philippi	14-3371	-34° 1' 30.56"	18° 31' 0.12"
Z54	Detention	Cynthia Road	Lotusriver	30-3250	-34° 1' 41.73"	18° 30' 20.71"
Z55	Detention	Schaap	Philippi	28-177	-34° 2' 2.74"	18° 31' 48.2"
Z56	Detention	Vlei Road	Philippi	28-237	-34° 2' 23.36"	18° 32' 2.88"
Z57	Detention	Lourier Street	Southfield	00-75574	-34° 2' 9.5"	18° 29' 23.39"
Z58	Wetland	Briana Crescent	Southfield	00-79581	-34° 2' 39.58"	18° 29' 1.3"
Z59	Detention	Soutpansberg Road	Seawinds	0-137477-2	-34° 4' 43.32"	18° 29' 34.42"
Z60	Detention	Drury Road	Vrygrond	97-148	-34° 5' 12.35"	18° 29' 3.97"

Appendix 11c – Location of stormwater ponds in the study area

ID	Pond Type	Road	Suburb	Erf No.	Latitude	Longitude
Z61	Detention	Madeira Drive	Muizenberg	00-160998	-34° 5' 36.56"	18° 29' 0.69"

Appendix 12 – Other identified benefits of stormwater harvesting

Water quality improvement

MAR&R provides water quality improvement benefits. The study area contains several informal settlements (slums, shanty towns), that generate wastewater and litter discharges into the drainage channels particularly in the upper reaches of the catchment. The CCT monthly grab samples of stormwater quality showed that the drainage system in the study area is highly impacted by pollution. In various studies (Hunt *et al.*, 2008; Fletcher *et al.*, 2014; Hathaway *et al.*, 2014), bio-retention cells have shown potential for considerable stormwater quality improvement. The selection of bio-retention cells as a potential infiltration device was aimed at benefiting from the water quality improvement. Further water quality improvement will result from movement through the sandy aquifer associated with the study area as discussed in Chapter 5. A preliminary assessment suggested that a residence time of about a year should provide die-off of pathogens water to values less than 10 *E.coli* counts/100 mL pond. Other contaminants that are likely to be substantially reduced are nutrients and heavy metals.

Amenity provision

Stormwater management infrastructure has the potential to provide amenity values to the local community including *inter alia* environmental improvement; recreation and aesthetics; education and awareness (Armitage *et al.*, 2013). There is evidence that well-maintained stormwater infrastructure can provide an economic benefit by increasing the selling price of nearby properties by 10% to 25% (USEPA, 1995; Dinovo, 1995). According to De Wit *et al.*, (2009), Cape Town has some of the most sought-after properties in the world largely due to amenity values directly linked to natural assets that provide aesthetic and recreational values including easy access to world-class beaches and green open spaces. Conversely, poorly maintained stormwater infrastructure characterised by litter and solid waste, are a potential breeding ground for mosquitos and can result in a health hazard for nearby communities. A poorly maintained stormwater pond would present low amenity, unpleasant ambience and subsequent reduction in property value. Evaluation of amenity value, however, requires specialised skills that were not available to this study.

The assessment of amenity benefits was thus based on available literature with values reasonably linked to the infrastructure in the study area. Some studies (*e.g.* De Wit *et al.*, 2009; Fisher-Jeffes, 2015) provided some values associated with natural and built environments in Cape Town including recreational opportunities in parks, nature reserves, beaches and other open spaces. In De Wit *et al.* (2009), the monetory value for local green open spaces was estimated to be in the range of ZAR 270 – ZAR 326 million per annum based on fees to various sites including Table Mountain National Park (TMNP) and Kirstenbosch. Fisher-Jeffes, (2015) suggested a value of ZAR 20.40/yr.m2 as an estimate for green spaces directly linked to stormwater management infrastructures such as parks, wetlands and open spaces based on De Wit *et al.*

(2009). With stormwater ponds in the study area covering a total land area of 0.8 km^2, the total benefit value from well-managed ponds could well be as much as ZAR16.2 million/yr – noting, however, that much of the residents are poor, which might mean that these benefits will be hard to realise.

Biodiversity preservation

Cape Town is located in the Cape Floral Region (CFR) and is considered an urban biodiversity hotspot (CCT, 2008). It has about 45% of southern Africa's plant species in only 4% of the total land area (Cowling & Hilton, 1992). The endemic vegetation includes *inter alia* closed scrub fynbos (Campbell 1986; Cowling & Hilton 1992; Cowling *et al*. 1997), hygrophilous mountain fynbos (Bovee *et al.*, 1998), broad *sclerophyllous* closed scrub (Kruger, 1978) and tall herb-lands (CCT, 2011). However, almost all ecosystems in Cape Town have been modified by human activities (Macdonald, 1989) leading to species habitat loss. The change in land use associated with urbanisation has caused permanent alterations of the ecosystem (Hobbs *et al.*, 2006) and has been determined to be the most important threat to biodiversity (Rebelo *et al.*, 2011). Studies have also shown that discharge of stormwater into natural systems in Cape Town have contributed to the negative impact on riverine plant ecosystem and species habitat loss (CCT, 2005; Brown & Magoba, 2009; Haskins, 2012). The riverine plant ecosystem in Cape Town has been altered to the extent that it is difficult to reconstruct historical communities (Brown 1998; Prins *et al.* 2004). For example, Zeekoevlei (one of the shallow lakes where abstraction of surface water is proposed in this study area), has been severely polluted and requires about R60 – R70 million in dredging and rehabilitation costs (De Wit *et al.*, 2009).

As for amenity, the evaluation of biodiversity value required specialised skills that were not available to this study. With the proposed infrastructure such as bio-retention cells where stormwater ponds are managed as an ecosystem, the study area could contribute to a valuable environmental resource over and above groundwater replenishment. Other environmental benefits would include *inter alia* energy conservation and mitigation of urban heat island effect, carbon sinking and air quality improvement, efficient land use planning and robust urban development (Dharmaratna & Gangadharan, 2011). In De Wit *et al.* (2009), it was estimated that for every ZAR1 spent on the environment by CCT in 2008/9, approximately ZAR8.30 – ZAR13.50 of ecosystems goods and services was generated. The benefits accrued from investment in biodiversity was 1.2 – 2 times more than the value added with investment in the economy estimated at ZAR7.30 per ZAR1. With an estimated cost of ZAR 56 million to install some the 60 bio-retention cells covering about 80 hectares, the total ecosystem goods and services generated for CCT might be as high as ZAR 465 – 756 million.