THE VIABILITY OF STORMWATER PONDS IN THE ZEEKOE CATCHMENT AS WATER RESOURCES FOR CAPE TOWN, SOUTH AFRICA

Report

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by

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EXECUTIVE SUMMARY

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 drought expected to reoccur frequently in future, this study investigated the viability of stormwater ponds in the Zeekoe catchment as water resources – selecting the 89 km^2 Zeekoe catchment situated in the southern part of Cape Town as a case study. The study focused on available storage in the study area, i.e. stormwater ponds and vleis (with storage enhancement using real-time control (RTC)) and the groundwater aquifer (with managed aquifer recharge (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 the extended detention of water to provide the balancing storage required for stormwater harvesting (SWH). RTC was also essential for flood control through pre-emptive draining before storm events.
ii. The 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. An 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 water treatment plant (WTP) – was also investigated to determine the opportunity of supplying water to meet the potable water demand.
v. Undertaking an economic analysis 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, and small-scale and large-scale desalination (CCT, 2018).

METHOD

The study investigated two SWH options for the Zeekoevlei catchment: directly from surface storage or indirectly via groundwater aquifers. The assessment of SWH from surface water storage included modelling the hydrological process to estimate the quantity of the stormwater resource, identifying the appropriate volumetric capacity and constraints, determining the effectiveness of RTC to address the challenges of storage without increasing the risk of flooding, and establishing the impact of climate and land use change. In the case of SWH from groundwater storage, the MAR model was used to assess the likely surface-to-groundwater transfer, and to estimate the recharge volume. The groundwater abstraction (groundwater recovery after MAR) was then modelled, initially at a small scale (1.5 km^2 with a single pond) and finally at a catchment scale (89 km^2 with 61 ponds). Other issues that were investigated included identifying the appropriate demand to be supplied (potable or non-potable), the extent of volumetric reliability, and all the costs (capital, operation and maintenance) and financial benefits associated with SWH and supply. Non-potable water needs included agriculture, the irrigation of residential gardens and open parks, and supply to toilets in the vicinity of the catchment. This was compared with the cost of harvested stormwater treated to a potable standard and injected directly into the potable water network or transferred to nearby existing water treatment works. The total costs of the production and supply of stormwater as potable or non-potable water was determined and compared with other non-conventional water sources such as seawater, wastewater effluent, and the augmentation of existing systems and groundwater that the City of Cape Town was considering implementing to mitigate the impact of water scarcity.

STORMWATER HARVESTING UTILISING SURFACE WATER STORAGE

In the assessment of SWH from a catchment with seasonal rainfall, largely in the winter period, large storage areas were required to balance the temporal mismatch between the availability of the resource and demand. This was particularly necessary for non-potable water uses, e.g. the irrigation of agriculture, residential gardens and public open spaces as the demands are mainly in the dry summer period. To provide the required storage, an investigation was carried out into the use of available stormwater ponds for both flood control and water supply using RTC techniques. The use of RTC on stormwater ponds provided an opportunity to utilise the available 1 mm^3 capacity (about 5.5% of the modelled mean annual volume of stormwater estimated at 18 mm^3). An assessment was undertaken to determine the reliability and adequacy of the storage to balance the mismatch between the 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 the irrigation of public open spaces). The storage in the ponds was only able to supply 44, 60 and 58% of the demands of 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, storage in vleis was assessed in stepwise incremental volumes of 1 mm^3 to determine the optimal storage required to account for the mismatch between 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) after supply to selected demands and permanent pooling in the vlei 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 a capacity of 5 mm^3. In general, it was determined that stormwater supply to non-potable demand was sensitive to balancing storage and required an optimum capacity of 20 to 30% of stormwater volume.

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

Other factors such as land use and climate change will likely also affect the volume of water to be harvested in the study area in future. An assessment of the impact of climate change utilising the 26 climate change prediction models available suggests a future annual mean reduction in rainfall of 40 to 200 mm with an increase in mean daily temperature of 3 to 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 to 9 mm^3 (15 to 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 of the possibility of utilising groundwater storage. The study area had sufficient aquifer storage with sandy soils that could support surface-to-groundwater infiltration.

STORMWATER HARVESTING UTILISING GROUNDWATER STORAGE

In the hydrological modelling of the study area, the system was adjusted to include MAR and account for the adaptation of stormwater ponds for the infiltration process using the available low-impact development (LID) or sustainable drainage systems (SuDS) tools. The basic approach adopted in this investigation 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.

The most suitable options are those that blend well with the stormwater pond environment and functions. The modification of the existing stormwater ponds to function as infiltration cells, perhaps with elements borrowed from bio-retention cells (in addition to retaining their existing flood control function), seems the most promising. This was to take advantage of the available aquifer storage in the study area as identified in various studies, such as those of Henzen (1973), Tredoux et al*.* (1980) and Adelana et al. (2006, 2010). The groundwater abstraction was modelled in MATLAB using an approach presented in Mahinthakumar and Sayeed (2006) using data from previous research, including "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). In the assessment, it was determined that the physical characteristics of the Zeekoe catchment, i.e. flat terrain, pervious sandy soils and unconfined aquifers, would support borehole yields in the range of 3.5 to 8.1 ℓ/s from 140 boreholes to provide a mean annual groundwater yield of 29 to 33 mm^3. The actual additional groundwater resource due to stormwater infiltration was 9 to 12 mm^3 (about a 30% increase). The attractiveness of the deliberate recharge of stormwater to promote infiltration would increase with increasing imperviousness from land use change. For example, it was determined that the natural mean annual infiltration volume in the study area is likely to decrease to 10 to 13 mm^3 based on a hypothetical 75% level of imperviousness. The results from modelling various potential groundwater abstraction scenarios in the Zeekoe catchment show that, depending on the aquifer parameters (conductivity, porosity and aquifer depth), the suitable borehole pumping rates ranged from 3.5 to 8.1 ℓ/s from 140 boreholes in the 89 km^2 catchment. 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 modelling show that the stormwater harvested from groundwater storage would theoretically be adequate for potable water uses with minimal additional disinfection treatment. For the non-potable water demands, the abstracted water would theoretically not require additional treatment (based on the South African Water Quality Guidelines for Irrigation). Other studies, such as those of Lim et al. (2015), have also shown that microbial pollution in stormwater from groundwater storage can be significantly reduced to levels where the water could be directly used for some indoor residential needs with a limited level of contact, e.g. washing machine use and toilet flushing. Another study, that of Vanderalm et al. (2010), also showed that stormwater recovered from an aquifer after a mean residence time of 240 days was suitable for non-potable water applications. Overall, SWH from groundwater storage provides larger water quantities at a better water quality than the surface storage option.

USE OF STORMWATER AS POTABLE OR NON-POTABLE WATER

The non-potable demands identified in the study, i.e. the irrigation of urban agriculture, residential gardens and public open spaces, were mainly in summer, thus mismatched with the availability of the stormwater resource, which was largely available in winter. In the yield-demand analysis, the impact of toilet flushing (an indoor water use) with regard 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 stormwater 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 volumetric (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. Treatment to potable standards would eliminate the potential public health risks from cross-connections. It was also determined that 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 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 400 m from the ponds and a travel time of 300 days would allow for a reduction in pollution associated with *E. coli* to values less than 10 counts per 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.

ANALYSIS OF COSTS

An analysis of costs (including capital, operation and maintenance costs) was undertaken for the two sources (surface water and groundwater) and two supply requirements (potable and non-potable). The findings were compared with indicative unit costs of water from proposed new sources published in the City of Cape Town Water Outlook Report of 2018 (CCT, 2018). The City of Cape Town Water Outlook Report of 2018 shows that the indicative unit costs of water from existing and various proposed new sources were as follows: existing reservoirs (R5–R6 per kℓ); groundwater (R7–R10 per kℓ); reclaimed water (R8–R11 per kℓ); and large-scale desalination (R12–R19 per kℓ), rising to R35 per kℓ for small-scale desalination. From this comparison, it was determined that SWH was competitive with these alternatives, with managed aquifer recharge and recovery (MAR&R), combined with potable water demand using the City of Cape Town's existing water reticulation system, being the most cost-effective approach.

CONCLUSIONS

This study has attempted to contribute to the debate on alternative water resources by considering the possibilities of SWH from surface and groundwater storage to meet potable and non-potable demands at a catchment scale. The factors that were determined to be important and that needed to be considered for the efficacious application of an SWH system included the availability of storage (surface or groundwater), the catchment characteristics (terrain, soil types, level of development, population density) and the seasonal availability of the stormwater resource (winter or all-year rainfall). The study also assessed the impact of land use and climate change on the quantity of the stormwater. Having considered these factors, this study 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. the irrigation of agriculture, residential gardens and public open spaces in areas such as Cape Town with rainfall limited to the winter period, storage in the range of 20 to 30% of mean annual modelled stormwater volume would be required to balance the mismatch between the 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 the 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 an unconfined aquifer, need to be present. In the selection of groundwater abstraction rate and distance of boreholes from ponds, the study confirmed that a residence period of at least one year should be allowed to provide for a reduction in *E. coli* to values less than 10 counts per 100 mℓ.
- The construction and operational costs of the SWH and distribution infrastructure are a major factor in the selection of the system scale (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 per kℓ) higher than for potable water.
- Based on discussions with City of Cape Town 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 are high.
- The 26 climate change prediction models suggest a future annual mean reduction in rainfall of 40 to 200 mm and an increase in mean daily temperature of 3 to 5 ^{o}C by 2100, compared with the study period (2006–2016). The impact of the reduction in rainfall and an increase in temperature is likely to be a 15 to 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 areas, that minimise loss through spillage, flood control and stormwater quality improvement.

Overall, the study has provided insight into opportunities for stormwater use with the partial or full treatment of 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. Besides the relief on existing water resources by such an alternative water source, additional benefits, e.g. stormwater quality improvement, were identified.

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 areas to balance 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 aquifers. The scope of the research was limited to the selected catchment and did not consider qualitative factors. These areas are recommended for future research, and 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 their reactions need to be categorised according to their demographics (e.g. level of education, income and age group), preferred uses and under what conditions the resources would be accepted or considered (e.g. water scarcity and restrictions, high tariffs).
- A comprehensive study of SWH, considering all the catchments in the City of Cape Town, to determine the total aggregated volumes available and their benefit. Several 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, commerce and institutions to determine if there are significant needs in the rainy season that might make it unnecessary to treat the water to potable.
- It is necessary to determine whether the cost savings that might be achieved through the joint installation of a dual reticulation system in a greenfield 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 an SWH infrastructure by local residents and their willingness to bear the associated maintenance costs and management requirements.
- A pilot study is required to determine the suitability of bio-retention 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.
- A detailed exploration of additional benefits is required, including amenity values such as increasing property values, biodiversity preservation and cooling to minimise the urban heat effect.

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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)
$e(\tau)$	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 and Ampt model determined as the intercept
$K(x, y)$	Hydraulic conductivity in two 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 and 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 (^oC)
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 (^oC)
T_{mean}	Daily mean temperature (^oC)
T_{min}	Mean daily minimum temperature (^oC)
TR	Daily temperature range (^oC) (*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- 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

AWRMS	Atlantis Water Resource Management Scheme
BCA	Benefit-cost Analysis
CEA	Cost-effectiveness analysis
CoR	Cost of Replacement
CCT	City of Cape Town
CFA	Cape Flat Aquifer
CMIP	Coupled Model Intercomparison Project
CPHEEO	Central Public Health and Environmental Engineering Organisation
CSAG	Climate Systems Analysis Group
CTSDF	Cape Town Spatial Development Framework
CVM	Contingent Valuation Method
CWR	Crop Water Requirement
DALY	Disability-adjusted Life Year
DECNSW	Department of Environment and Conservation New South Wales
DEM	Digital Elevation Model
DRI	Double ring infiltrometer
DWA	Department of Water Affairs
DWAF	Department of Water Affairs and Forestry
DWS	Department of Water and Sanitation
EAW	Equivalent Annual Worth
EC	Electrical Conductivity
E. Coli	Escherichia coli
ERR	External Rate of Return
ET	Evapotranspiration
FAO	Food and Agriculture Organisation
GA	Genetic Algorithm
GCM	General Circulation Model
GFS	Global Forecast System
GI	Green Infrastructure
GIS	Geographic Information System
IPCC	Inter-governmental Panel on Climate Change
IRR	Internal Rate of Return
ISE	Integral Square Error
IWR-MAIN	Institute for Water Resources – Municipal and Industrial Needs
LID	Low-impact Development
LIDAR	Light Detection and Ranging
LSE	Least Square Error
MAP	Mean Annual Precipitation

MAR	Managed Aquifer Recharge
MAR&R	Managed Aquifer Recharge and Recovery
MCA	Multiple Criteria Analysis
MUSIC	Model for Urban Stormwater Improvement Conceptualisation
NBS	Nature-based Solutions
NCEP	National Centre for Environmental Prediction
NPV	Net Present Value
NSE	Nash-Sutcliffe Efficiency
O&M	Operation and Maintenance
PBIAS	Percentage Bias
PCSWMM	Personal Computer Storm Water Management Model
PD	Proportional Differential
PHA	Philippi Horticulture Area
PI	Proportional Integrated
PID	Proportional Integrated Differential
R^2	Correlation Coefficient
RBC	Rule-based Control
RCP	Representative Concentration Pathways
RMSE	Root Mean Square Error
RTC	Real-time Control
RWH	Rainwater Harvesting
SAWS	South Africa Weather Service
SCADA	Supervisory Control and Data Aquisition
SEE	Standard Error Estimate
SRTC	Sensitivity-based Radio-tuning Calibration
StatsSA	Statistics South Africa
SuDS	Sustainable Drainage System
SUSTAIN	System for Urban Stormwater Treatment and Analysis Integration
SWH	Stormwater Harvesting
SWMM	Storm Water Management Model
TDS	Total Dissolved Solids
TMG	Table Mountain Group
UCT	University of Cape Town
UV	Ultra-violet
UVQ	Urban Volume and Quality
UWC	Urban Water Cycle
UWM	Urban Water Management
UWOT	Urban Water Optioneering Tool
VR	Volumetric Reliability
WHO	World Health Organisation
WRC	Water Research Commission

WSUD	Water-sensitive Urban Design
WTP	Water Treatment Plant
WWAP	World Water Assessment Programme
WWTW	Waste Water Treatment Works
YAS	Yield After Spillage
YBS	Yield Before Spillage

CHAPTER 1: INTRODUCTION

South Africa is a semi-arid and water-stressed country that relies heavily on surface water from unevenly distributed rainfall with a mean annual precipitation (MAP) of 450 mm (about 50% of the world's MAP) (DWA, 2004; Pitman, 2011). The corresponding streamflow in 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 will 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 the provision of treated sewage effluent to selected users as an alternative to potable water. The measures have been effective in successfully maintaining water withdrawal from the major reservoirs at a mean annual value of about 300 mm^3 per year over more than ten years (2003–2017) (DWS, 2014; GreenCape, 2017). 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, the CCT is now considering alternative water resources such as seawater, wastewater effluent, the augmentation of existing surface water systems and groundwater.

Seawater comprises 97% of total global water and would ideally provide a sufficient and reliable water source for a coastal city such as the 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 or religious grounds seemed to be preventing the possible reuse of treated effluent (Wilson and 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 this target, households were compelled to reuse greywater. Although the reuse of greywater contributed to a reduction in the total amount of water used at the household level, health risks during handling needed to be assessed and guidance provided to safeguard the population.

With no additional major dam sites available near the CCT, there are proposals to augment existing dams, e.g. raising the dam wall of the Voëlvlei Dam to accommodate additional water brought in from various rivers and the 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 per 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, this has potential severe irreversible environmental impacts such as ground subsidence. In Mexico City, the excessive abstraction of groundwater over a long period of time (since the 1950s and greatly increased in the 1980s) has resulted in the subsidence of 0.4 m per year since 1984, reaching 8 m in some areas by 2010 (Ortiz-Zamora and 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 ground subsidence (Tredoux et al., 1980; Tredoux and Cain, 2010)

In the field of urban hydrology, a water cycle management approach aimed at environmental protection has emerged since 1990, variously called water-sensitive urban design (WSUD) in Australia, low-impact development (LID) in the USA and sustainable drainage systems (SuDS) in the United Kingdom (Fletcher et al., 2014). It 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 has been the subject of research by the Urban Water Management Research Unit at the University of Cape Town (UCT) 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). The new approach specifically promotes interventions such as stormwater harvesting (SWH) and reuse (Fisher-Jeffes, 2015; Rohrer, 2017; Rohrer and 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 and Pickering (2006) determined that the mean cost per kilolitre of SWH was lower than many other sources, including seawater desalination, rainwater harvesting and long-distance pipelines.

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 in Cape Town, South Africa. This included the following:

i. Open surface water storage in stormwater ponds enhanced through real-time control (RTC), i.e. the 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 that can function as infiltration cells to transfer stormwater to aquifer storage for the augmentation of groundwater resources
iii. Determining the extent and volumetric reliability of harvested stormwater supply for non-potable water uses such as urban agriculture, the irrigation of residential gardens, open parks and toilet flushing
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 report consists of eight chapters. Chapter 1 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, the assessment of water supply reliability and an economic analysis. Chapter 3 includes an overview and statement of the method, selection of a suitable 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 the 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. the aquifer in the study area. Chapter 6 discusses 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 report.

CHAPTER 2: LITERATURE REVIEW

In this chapter, the relevant published work on similar studies, i.e. new and innovative approaches to stormwater harvesting, 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 the 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 of the potential for managed aquifer recharge and recovery, i.e. the augmentation of groundwater storage with stormwater. It also provides a discussion of the available opportunity for stormwater quality improvement in groundwater storage and a summary of international and local case studies of the 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 essential methods and components for an economic analysis and identification of the benefits from SWH. Section 2.7 presents the expected social issues associated with SWH. Section 2.8 provides a summary of the opportunity for SWH and its use.

2.1 WATER SCARCITY

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 the 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. Raskin et al. (1997), Alcamo et al. (1997, 2000) and Shiklomanov (1998), have shown that urbanisation, with the associated improving standards of living and rising incomes, is also a key factor 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 and 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 the water resource limit has been reached. Some examples of constrained water resources include 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 (Western Australia State Water Strategy, 2003) and Mexico City (Ortiz-Zamora and Ortega-Guerrero, 2010). In Australia, the drop in groundwater levels has been exacerbated by below-mean rainfall due to prolonged droughts. 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 its rapidly growing domestic, commercial and industrial demands, are among the most affected by water scarcity (Rijsberman, 2006). There are about four billion people (more than half of the world's population), mainly in 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 or too costly to meet their needs (Rijsberman, 2006).

A new approach to stormwater management has emerged that allows for the preservation of the environment, and simultaneously promotes the reuse of a resource that would otherwise have been lost using traditional methods. This approach is variously termed water-sensitive urban design in Australia, low-impact development (LID) or green infrastructure (GI) in the USA and sustainable drainage systems (SuDS) in the United Kingdom (Fletcher et al., 2014). In South Africa, these were incorporated into guidelines to assist in the local design and management of SuDS (Armitage et al., 2013), as well as the development of 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 in water supply through what is termed stormwater harvesting. 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 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).

2.2 STORMWATER AS AN ALTERNATIVE WATER RESOURCE

2.2.1 Overview

The conventional management of urban stormwater is aimed at promptly collecting and conveying runoff from a storm event away from locations of rainfall incidence to avoid flooding and minimise inconvenience. However, the disposal of stormwater in this manner results in the loss of a potentially valuable water resource from an urban area. Some studies (e.g. Brown et al., 2008; Dotto et al., 2014; Wong, 2007) have established that cities are potentially important catchment areas in their own right, which, if well managed, can meet a substantial portion of their 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% of respondents support the non-potable use of stormwater for municipal and industrial purposes, 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 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 to using SWH: 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 (Brown and Magoba, 2009; CCT, 2005; Haskins, 2012). Studies that have assessed the potential of SWH as a resource for water supply (e.g. Fletcher et al., 2004, Mitchell et al., 2007, Philp et al., 2008, Tortajada et al., 2013, Fisher-Jeffes, 2015, Hoban et al., 2015, Rohrer and Armitage, 2017) considered the following:

- Short- to medium-term storage of flood runoff – to balance the mismatch between 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
- The 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 the 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 and Mitchell, 2006)

- Construction of a separate reticulation system (sometimes called dual or third-pipe reticulation) – for the distribution of non-potable stormwater to avoid the contamination of potable water in the municipal reticulation system.

2.2.2 Stormwater as a water resource

SWH should not be confused with rainwater harvesting (RWH). RWH is the use of runoff from the rooftops of buildings, while SWH is the utilisation of catchment-scale stormwater management infrastructure to collect, store and treat runoff for reuse mainly for non-potable water purposes (Armitage et al., 2013; Fisher-Jeffes, 2015; Mitchell et al., 2007). RWH is typically practised at a household scale, mostly in remote villages, and has been carried out for centuries (Hamdan, 2009; Mwenge Kahinda et al., 2010). SWH is a more recent concept that is associated with urban areas and is mainly promoted due to water scarcity (Mitchell et al., 2007). SWH is normally implemented at a regional scale and requires a suitably modified urban stormwater management infrastructure to support the practice. Some of the countries that have adopted SWH as a water resource include China (Hamdan, 2009), the UK, the 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 SWH and its use in South Africa is the generally poor water quality and 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).

2.3 STORMWATER HARVESTING FROM SURFACE WATER STORAGE

2.3.1 Overview

The successful implementation of an 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). This storage can be provided by ponds and lakes in the stormwater network. Some local studies (e.g. Fisher-Jeffes, 2015, Rohrer, 2017, Rohrer and Armitage, 2017) have shown that SWH is viable and has the potential to meet a reasonable percentage of demand in a study area. Meanwhile, a study such as that of 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).

2.3.2 Surface water storage options

2.3.2.1 Introduction

The storage options for SWH systems include closed (e.g. underground tanks) or open (e.g. stormwater ponds) systems. The determination and selection of a suitable storage option for an 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 the Department of Environment and Conservation of New South Wales (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 an SWH system should consider maximising volumetric reliability, while minimising storage size and associated costs. A brief overview of closed storage and a discussion of open storage are provided as follows:

2.3.2.2 Closed storage for stormwater

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

2.3.2.3 Open storage for stormwater

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

- **Detention ponds:** These are dry basins constructed to temporarily hold stormwater for short periods of time to mitigate flood risk downstream of the ponds (Armitage et al., 2013; Woods-Ballard et al., 2007). They are probably the most widely used stormwater ponds, comprising some 70% of ponds in Cape Town (Rohrer, 2014). They are popular for flood control due to their sizeable storage capacity, while they have 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 the single function of flood control. Some detention ponds are, however, designed to serve a dual purpose during dry periods, such as recreational facilities or car parks. They can also be adapted to contribute to the aesthetic value of the area (Woods-Ballard et al., 2007). 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 the deposition of solids or 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 (Armitage et al., 2013; Debo and Reese, 2003; Mays 2001; Woods-Ballard et al., 2007). The water quality improvement function of retention ponds is typically characterised by processes such as sedimentation, filtration, infiltration and biological uptake processes to remove pollutants from stormwater runoff (Armitage et al., 2013; Stahre, 2006). Retention ponds provide a 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, mosquito 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% (Dinovo, 1995; USEPA, 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 by 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 (Armitage et al., 2013; Woods-Ballard et al., 2007). 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 makes 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 nutrients, and the removal of some pathogens (Armitage et al., 2013; Field and Sullivan, 2003; Parkinson and Mark, 2005).

Other examples of open storage systems include natural wetlands, reservoirs, lakes, rivers, streams and creeks (Goonrey, 2005). Well-designed open storage systems can provide at least four benefits: the management of water quantity, the improvement of water quality, the provision of an 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 own right (Armitage et al., 2013).

2.3.3 Models used in the estimation of harvested stormwater volume

Many models have been developed since the 1960s, with the earliest (the Stanford watershed model) being 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. Their complexity has increased significantly in the last few years (Elliott and Trowsdale, 2007). These models can be used to estimate the harvested stormwater volume. Some models previously used to determine harvested stormwater volume include the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (eWater, 2013), City Drain (Achleitner et al., 2007), the Urban Water Optioneering Tool (UWOT) (Rozos et al., 2010), the System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) (Lee et al., 2012), the Storm Water Management Model (SWMM) and its various proprietary derivatives, including XP-SWMM (XP Solutions, 2014), the Personal Computer Storm Water Management Model (PCSWMM) (CHI, 2014) and MIKE SWMM (DHI Denmark, 2014), Info-works (Innovyze, 2011), Urban Volume and Quality (UVQ) (Mitchell and Diaper, 2005) and the Aqua Cycle (Mitchell, 2004).

Various studies, such as those of Bach et al. (2014), Breen et al. (2006) 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 the 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 et al. (2008) detailed the impact of data requirements and the calibration of integrated surface water modelling and management. Peña-Guzmán et al. (2017) reviewed various models developed between 1990 and 2015 to assess how they account for all components in the urban water cycle (UWC) and their 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 the natural urban landscape and engineered water cycles. Other authors, such as Chiu et al. (2008), Mwenge Kahinda et al. (2008), Darshdeep and Litoria (2009) and Goonrey et al. (2009) have also provided guidance on the selection of suitable sites for implementing SWH and reuse.

2.3.4 Performance assessment of a stormwater harvesting system

Several performance measures to assess SWH have been developed over the past few decades (Mitchell et al. 2007). These have been determined to be valuable in assessing the efficiency and effectiveness of an SWH system in various environments and circumstances. In a review of the suitability and application of 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 the resilience of an 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 the 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

2.3.5 Real-time control for stormwater harvesting systems

The traditional design of stormwater management infrastructure typically only focuses on flood control with runoff peaks and volumes estimated based on the rainfall and land use of a specific area SANRAL, 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 the designed capacity. To minimise the risk of flooding, increasing the physical capacity of stormwater management infrastructure may be required through the installation of additional channels, pipes and storage components. In a developed urban area, this may not be possible, or may be costly for various reasons, including limited land for expansion, or the need to demolish buildings and roads (García et al., 2014). To address this challenge, real-time control has been developed to provide additional capacity without necessarily increasing physical storage (Borsanyi et al., 2008). Stormwater infrastructure management with RTC involves the dynamic control of the system with specific operational rules to consolidate available storage with the primary objective of minimising redundancy (Colas et al., 2004; USEPA, 2006).

This includes the dynamic management of water levels in the storage components to increase retention time and optimise hydraulic capacity (Vallabhaneni and Speer, 2011). Management includes the continuous monitoring and adjustment of stormwater flow rates and storage volumes with a set of rules, depending on the status and needs of 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 the 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 the implementation of RTC may provide other benefits such as storage for SWH and the 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 and Grum, 2014). The implementation of RTC also allows for dynamic management with the 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 the implementation of RTC in stormwater systems were identified in various studies (e.g. Colas et al., 2004, USEPA, 2006, Vallabhaneni and 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; USEPA, 2006). Adjustments are then applied either manually (opening and closing the 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 the adjustment and control of the RTC systems (García et al., 2015; Rohrer, 2017). According to García et al. (2015), the control rules that must be incorporated would generally be a function of the prevailing or anticipated conditions and measurements. Most researchers prefer the application of local RTC systems due to their simplicity and straightforwardness, site-specific control and independence of the whole system, limiting error to specific sites (e.g. USEPA, 2004, García et al., 2015).
- **Regional control:** The regional control management approach is similar to 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; USEPA, 2004). Hence, a manually operated system with the site-specific opening and closing of valves would not be suitable for regional RTC (USEPA, 2004). A regional RTC system typically requires a remotely controlled regional communication system such as a Supervisory Control and Data Acquisition (SCADA) program located on a central server (Colas et al., 2004; USEPA, 2006). 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 operated automatically 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 the proportional integrated differential (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. proportional integrated (PI) and proportional differential (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 and Modica, 2002). Another advantage of the regional RTC system is the ability to limit optimisation rules to a specific facility and local conditions (USEPA, 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 are 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; Rohrer, 2017; USEPA, 2006). The global control management approach is considered to be 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 (Rohrer, 2017; USEPA, 2004). 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; USEPA, 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).

2.3.6 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 drought (Hoban et al., 2015; IPCC, 2014). 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 coastal cities such as Cape Town could mitigate the increase in temperature, but this advantage would likely not extend to the interior. The urban heat phenomena (i.e. the 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 the treatment and supply of the anticipated flood flows to help address the challenges of climate change. Well-designed SWH could also contribute to addressing the challenges of urban heat by providing water features in cities.

2.3.7 Stormwater treatment from surface water storage

The 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 micro-organisms, and 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 be used cautiously with limited concentrations as their excessive application may lead to other toxic by-products (Rizzo and Dougherty, 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 removing emerging contaminants (Nghiem et al*.,* 2002, 2005; Tambosi et al., 2010), but the capital and operation costs may be prohibitive (Grassi et al., 2013).
- **Adsorption:** A treatment process with the 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 removing emerging contaminants, is less costly than filter membranes and less likely to produce toxic by-products (Westerhoff and James, 2003; Yener et al., 2008; Yu et al., 2008). However, the regeneration costs of adsorption processes, particularly with their energy requirement and off-site transportation, can be prohibitive (Brown, 2004).

2.4 STORMWATER HARVESTING FROM GROUNDWATER STORAGE

2.4.1 Overview

The use of groundwater storage for SWH is possible through managed aquifer recharge and recovery. With this approach, stormwater is temporarily stored in stormwater ponds that have been adapted to function as infiltration cells, with specific features to allow the recharging of groundwater aquifers for future abstraction and supply (Dillon et al*.,* 2010; Wu et al*.,* 2012). The 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 the large storage capacity offered and to benefit from limited loss from evaporation (Philp et al*.,* 2008). The treatment processes associated with MAR&R, i.e. extended retention in the ponds that allow the sedimentation of suspended particles and filtration in the groundwater aquifer, also provide 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. the 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) (Fisher-Jeffes, 2015; Wu et al., 2012).

2.4.2 The Atlantis Water Resource Management Scheme

The Atlantis Water Resource Management Scheme (AWRMS) is an example of an 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 given in Figure 2-1.

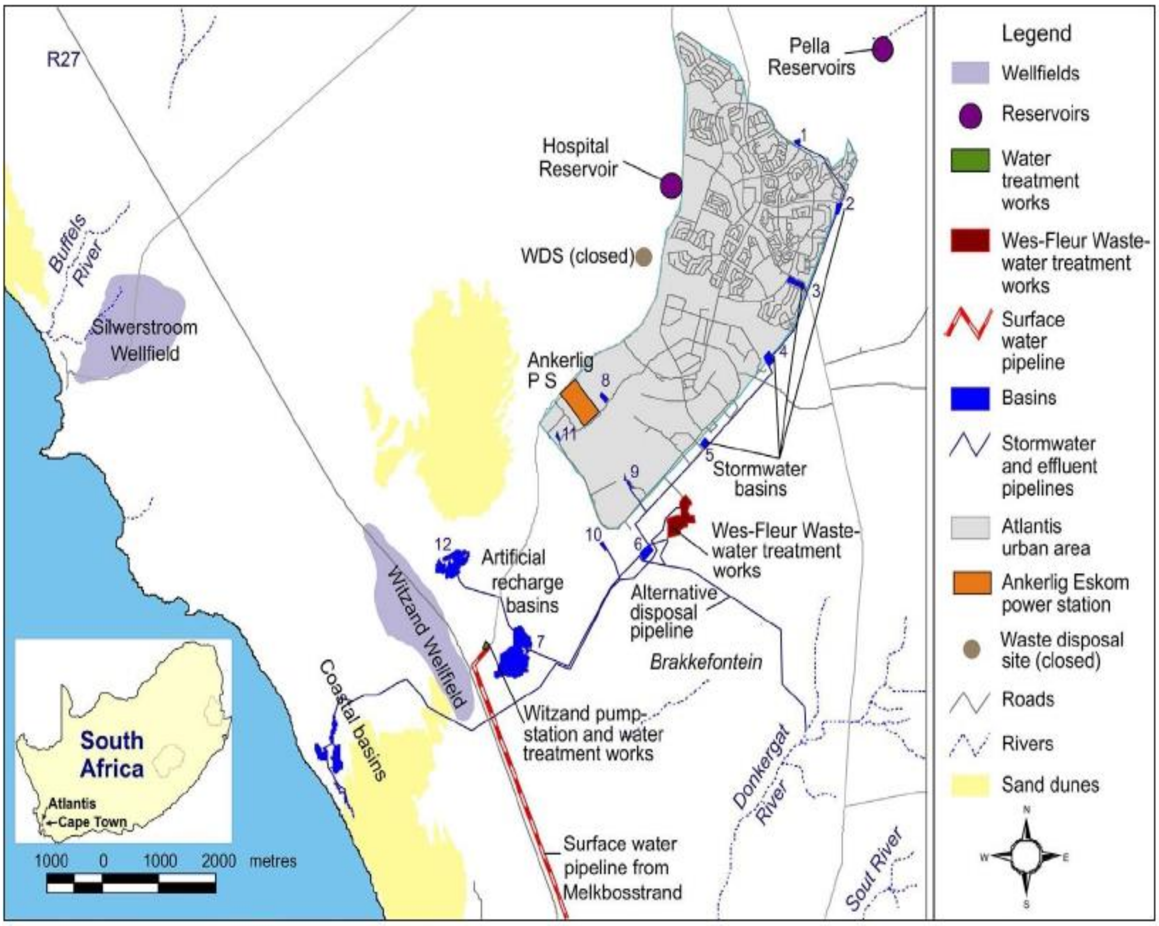

Figure 2-1: Atlantis Water Resource Management Scheme (Bugan et al., 2016)

The system was established to supply water to the town of Atlantis, located on the West Coast 50 km north of Cape Town, which was not then linked to the city's 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 the artificial recharge of stormwater and wastewater to ensure adequate water supply. After almost 40 years in operation, the AWRMS can be considered a local time-tested SWH scheme that can be used for best-practice benchmarking making use of MAR&R, where stormwater is temporarily stored in a groundwater aquifer and later recovered for reuse. In the AWRMS system, approximately 7,500 m^3 per day of stormwater and wastewater is recharged to boost the groundwater resource by more than 2.7 x 10^6 m^3 per year, i.e. an increase of about 30% (DWA, 2010). In a detailed groundwater model developed in 2017 to assess the performance of the AWRMS system, it was determined that there was potential to increase the groundwater resource to 6 x 10^6 m^3 per year, i.e. an increase of about 60% (Jovanovic et al., 2017). The lessons from the 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:** One needs to avoid over-abstraction that may result in a significant drawdown of the water table. This would cause seawater intrusion and the 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 the 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 and 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 the 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 metropolitan municipalities in South Africa, including Gauteng and eThekwini (Durban), have explored the potential for similar systems to maximise the sustainable use of water supply from groundwater storage (Quayle, 2012).

2.4.3 International case studies

MAR&R has been implemented in some countries worldwide, including Australia (Dillon et al*.,* 2010; Miotliński et al., 2014; Page et al., 2009), the 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, 80 and 100 mm^3, respectively (Dillon et al., 2010). 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 the irrigation of sugar cane and other crops (Dillon et al., 2010; Miotliński et al., 2014; Page et al., 2009). 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 the injection of treated water into the groundwater aquifer and the recovery of about 68,000 and 9,500 m^3 per day, respectively. In Namibia, MAR&R is a water resource for 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 per year (Murray et al*.*, 2007; Redux et al., 2009).

2.4.4 Stormwater treatment from groundwater storage

2.4.4.1 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) (Fletcher et al*.,* 2014; WWAP, 2018). 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 (Armitage et al., 2013; Mitchell, 2006; Woods-Ballard et al., 2007). Furthermore, the method could 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 SuDS terminology (Armitage et al., 2013; Woods-Ballard et al., 2007). Although the selection of technology to include in the "treatment train" is case specific, the objective of 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 to 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 (Fisher-Jeffes, 2015; Philp et al*.,* 2008). Guidelines such as those of Woods-Ballard et al. (2007) and Armitage et al. (2013) assist in the identification, selection and design of a suitable "treatment train", including providing information on 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 (Armitage et al., 2013; Woods-Ballard et al., 2007). Additional treatment would be required to use the harvested stormwater for potable water purposes.

2.4.4.2 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 2.3.7. Disinfection processes include chlorination, ultraviolet (UV) radiation, oxidation and/or membrane filtration (Mitchell et al., 2007; Philp et al., 2008). 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 (Fisher-Jeffes, 2015; Philp et al., 2008).

2.5 POTENTIAL DEMAND FOR HARVESTED STORMWATER

2.5.1 Overview

In many studies, such as those of Mitchell et al. (2007), Goonrey et al. (2009), Fisher-Jeffes (2015), Rohrer (2017) and Rohrer and 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 irrigation and other non-potable domestic water demands, such as toilet flushing and washing machine use. The determination of appropriate non-potable demand to supply with stormwater should consider various factors, such as water quality at the source, the distribution requirement and end-use (Buchmiller et al., 2000). The likely non-potable water demands and potential for stormwater reuse are discussed below.

2.5.2 Urban agriculture

About 70% of the available global freshwater resource is used for agricultural production (Prathapar, 2000). As shown in Figure 2-2:, the per-capita annual water availability of most African countries will be less than 1,000 m^3 by 2025.

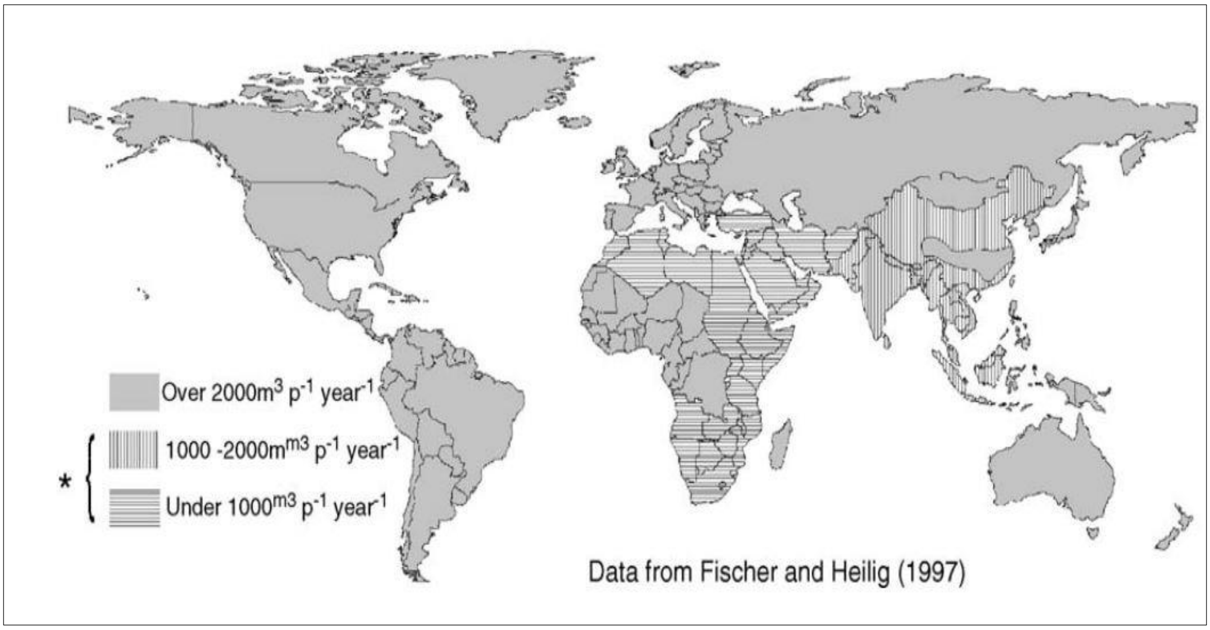

Figure 2-2: **Global mean annual per capita renewable water by 2030 (Oweis and Hachum, 2006)**

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 countries in West Asia and North Africa, where about 75% of freshwater is used for agriculture, much of the water has since been reallocated to other sectors due to rapid industrialisation, urbanisation and the high population (Oweis and Hachum, 2006). Further, the mean annual per-capita renewable water in countries in West Asia and North Africa has reduced from 3,500 m^3 in 1960 to below 1,500 m^3, and is expected to decrease to less than 700 m^3 by 2025 (Oweis and 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 per capita per 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. The possible presence of nutrients would decrease the required amount of fertilizers used (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 those of Oweis and 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 (ET_o) needs (Allen et al., 2006). Various approaches have been developed and used for the estimation of ET_o. The American Society of Civil Engineers (Jensen et al., 1990) and the 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 (Choisnel et al., 1992; Jensen et al., 1990). 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 the faster computation of CWR, the FAO developed the CROPWAT software, based on the climatic data and soil properties of over 100 countries compiled by the FAO's Agrometeorological Group (Smith, 1992).

2.5.3 Irrigation of residential gardens and public open spaces

The increasing competition for 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 the irrigation of residential gardens and open parks (Mesa-Jurado et al*.,* 2012; Milano et al., 2012). 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 (Rajaganapathy et al*.,* 2011; Xu et al., 2010). In Cape Town, South Africa, properties with residential gardens typically use 20–40% of the domestic water demand for garden irrigation (Jacobs and Haarhoff, 2004). With increasing water supply restrictions in Cape Town and other cities worldwide, e.g. in Australia (Begum and 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 (Benetti, 2008; Hatt et al*.,* 2006). In Queensland, Australia, SWH and reuse have already been accepted as an alternative to potable water for demands such as garden irrigation (Mitchell, 2006).

2.5.4 Domestic non-potable indoor water demand

A water resource such as stormwater could be suitable for domestic non-potable indoor water needs with minimal envisaged 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), findings show that toilet flushing with LID treated stormwater was below the USEPA's annual risk benchmark of $\leq 10^{-4}$ per person per year and within the World Health Organisation (WHO)'s recommended disease burdens of $\leq 10^{-6}$ DALYs per person per year. 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 the treatment of water contaminated by high microbial pollution with processes that include membrane filtration, chemical disinfection (chlorine and bromine compounds) and UV radiation (DWA, 2011). Chlorination would be necessary for indoor water use such as toilet flushing and washing machine use as a precautionary measure. Since stormwater for non-potable water use would be of a 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 the treatment of 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 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 an existing municipal or dual-reticulation system, respectively. The following methods are available to estimate domestic water demand (Rinaudo, 2015):

- **Time-dependent extrapolation method:** This method is suitable for use with time series as it utilises growth rates for the projection of future demand based on previous circumstances. The disadvantage of this 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. This method accounts for site-specific characteristics and is suitable for the estimation of the 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). This method estimates water use based on anticipated changes in variables that correspond with historical observations.

The 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 accuracy. Historical trends may vary significantly from the present situation. Various temporal-spatial drivers also determine where, how, when and why water is used (Wu et al., 2012). The drivers may range from human behaviour and attitudes towards water to general factors, e.g. property size, people in a household, affluence and climate (Fisher-Jeffes, 2015).

Tools such as the Institute for Water Resources – Municipal and Industrial Needs (IWR-MAIN) that combine spatial-temporal data have been developed and used extensively in the USA (Bauman and Boland, 1998; Wurbs, 1994) and globally (Mohamed and Al-Mualla, 2010). With the advent of geographic information system (GIS) software, some tools have been developed and used by water utility agencies, e.g. the UK Environment Agency and California Bay-Delta Authority (Davis et al., 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 and Van Beek, 1997) and Cape Town (Jacobs and 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 being 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 and day of the week, as highlighted in Roberts (2005) and Heinrich (2006), were not considered (Van Zyl et al*.,* 2008).

2.5.5 Approaches for stormwater harvesting and distribution

Two main approaches are available for the supply of potable and non-potable water, i.e. centralised (with a single abstraction location and a water distribution network covering the entire study area) and decentralised (with several abstraction locations and distribution networks covering sections of the study area) (Fisher-Jeffes, 2015; Hatt et al., 2004; Mitchell et al., 2007; Philp et al*.* 2008; Rohrer, 2017). The centralised system, with a source, intake, treatment and extended distribution network, has been the primary approach to 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, e.g. the 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: potable water from the municipal mains and non-potable water for demands that accept water of a lower quality. Such systems are common in Australia, where non-potable water is used for irrigation that is typically limited to catchments smaller than 200 ha (Mitchell et al., 2007; Philp et al. 2008).

2.5.6 The challenges associated with stormwater reuse

The main challenge of the 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 regard to non-potable water – particularly irrigation – is the mismatch between 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, the 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. the Lotus River and the 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 and Pfaff, 2008). On the contrary, religious views that tend to emphasise responsibility to the environment and sustainability could support stormwater reuse as they could be readily implemented in an equitable and just manner with costs equitably distributed, and environmental issues are taken into account (Fisher-Jeffes, 2015).

Another study, that of Ilemobade et al. (2009), showed that 94% of respondents supported the 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 the 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, the 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's 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's acceptance and the future use of the approach (Hatt et al., 2004).

2.6 ECONOMIC ANALYSIS

2.6.1 Overview of costs

In the traditional economic analysis for water systems, costs are attributed to the construction, operation and maintenance of the system, while benefits are typically limited to the level of service provided. The estimated benefit is obtained when the 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 only be viable after the 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; Dobes and Bennett, 2009; Fisher-Jeffes, 2015; Philp et al., 2008; Roebuck, 2007; 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.

2.6.2 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:** Capital costs comprise all the costs associated with the installation of the system components, including 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:** Operation 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 costs linked to O&M costs would typically include rented land (where applicable), operating staff, energy, labour and material for maintenance and repairs, planned periodic renovations, insurance and taxes, finance 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:

2.6.3 Cost-effectiveness analysis

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 in 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 and 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.

2.6.4 Benefit-cost-analysis

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 and 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. The use of BCA is further constrained by a lack of quantitative data on most of the benefits associated with SWH (Akram et al., 2014; Goonrey et al., 2009; Hatt et al., 2004; Philp et al., 2008).

2.6.5 Multiple criteria analysis

Multiple criteria analysis (MCA) is a technique typically applied where it is impractical to allocate attributes of a similar nature, i.e. it does not only consider a single attribute, e.g. the 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). The use of MCA may require experience with the method, especially in the valuation of unquantifiable elements (Dodgson et al., 2009).

2.6.6 Life-cycle costing analysis

Life-cycle costing analysis (LCCA) is an approach that considers all relevant costs and revenues related to the construction, operation and maintenance of a water supply system for the entire lifetime of the assets (Clift and 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. by placing a time value on money, where future expenditure is brought back to a present base year to allow a direct comparison between alternatives (Van Vuuren and Van Dijk, 2006). Several methods are used for economic analysis and evaluating investments in engineering, including 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 and capitalised equivalent (ADB, 2017).

2.6.7 Overview of benefits

Unlike costs that are relatively easy to estimate, the valuation of 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 at an alternative location (e.g. Lampe et al., 2005, Fisher-Jeffes, 2015, ADB, 2017).

2.6.8 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 a reduction in demand is a postponement of the need to provide additional capacity or the construction of a new water supply system. 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 (Hatt et al., 2006; Marsden and Pickering, 2006).

2.6.9 Flood mitigation and management

Various studies (e.g. Woods-Ballard et al., 2007, Fletcher et al., 2008, 2013, Huang et al., 2009, Fisher-Jeffes, 2015) have indicated that SWH can mitigate floods through peak flow attenuation and a reduction in runoff volumes. For example, SWH case studies in Australia have shown peak reductions of around 5 to 10% for the 100-year recurrence interval event (Fletcher et al., 2008; Hatt et al., 2006). 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).

2.6.10 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 (Brown and Magoba, 2009; CCT, 2005; 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 pre-development conditions. This has a positive impact on ecosystem health and contributes to biodiversity preservation.

2.6.11 Local amenity

Stormwater ponds designed according to the 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 (Armitage et al*.,* 2013; Haddock, 2004; Woods-Ballard et al., 2007). There is also evidence that such a landscape, designed according to the SuDS philosophy, can provide an economic benefit by increasing the selling price of nearby properties by 10 to 25% (Dinovo, 1995; USEPA, 1995).

2.7 SOCIAL ISSUES LINKED TO STORMWATER HARVESTING AND REUSE

2.7.1 Social acceptance

Non-conventional water supply approaches such as SWH are usually associated with poor water quality and are perceived to be prone to failure due to limited management experience (Hatt et al., 2006; Mitchell et al*.,* 2007; Philp et al. 2008). 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 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), Israel (Friedler et al., 2006), Jordan (Al-Jayyousi, 2004), Namibia (Murray et al., 2007; Tredoux et al., 2009), 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 a constrained water resource due to population growth include China (Junying et al., 2004), Germany (Nolde, 1999), Japan (Dixon et al., 1999), the 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 the 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 in the feasibility study. Various studies (e.g. Coombes and Mitchell, 2006, Ilemobade et al., 2009, Dobbie et al., 2012, Wu et al., 2012) have all shown that communities accept the alternative water sources with limited treatment for non-potable water uses.

2.7.2 Public health and safety

For SWH to gain public confidence and acceptance, the system should be set up so that there is minimal likelihood of failure and a 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. the safety of children from exposure to non-potable water, and the risk related to the consumption of fruit 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 flooding from failing embankments, the potential risk of drowning and mosquitos breeding in stagnant water (DECNSW, 2006; Fisher-Jeffes, 2015; NRMMC et al., 2008). These can often be managed by reshaping embankment slopes, limiting access or allowing an adequate water depth beyond levels for the breeding of mosquitoes.

2.8 SUMMARY

Water resources management around the world is rapidly changing due to water scarcity, which is being exacerbated by the growing demands of 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, and will 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 (Fletcher et al., 2004; Wong, 2011). The main changes identified include a shift away from the sole reliance on traditional sources, i.e. a fresh surface and a growing emphasis on environmental and ecological considerations (Wong, 2011). To sustainably provide water to meet increasing demands, new methods are required that do not need the construction of new systems or large-scale water transfer from one region to another (Gleick, 2000). Although SWH is not widely practised (Akram et al., 2014; Ilemobade et al. 2009; Wilson and Pfaff, 2008), 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, the expansion of existing reservoir capacity and importing water from resources in remote areas (Marsden and Pickering, 2006). Additional benefits of SWH are the avoidance of the 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 Cape Town.

CHAPTER 3: METHOD

3.1 OVERVIEW

In this study, the prospects for stormwater harvesting, utilising surface and groundwater storage, were 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 for transferring the water to an existing water treatment plant (WTP). An overview of the method is presented in Figure 3-1:.

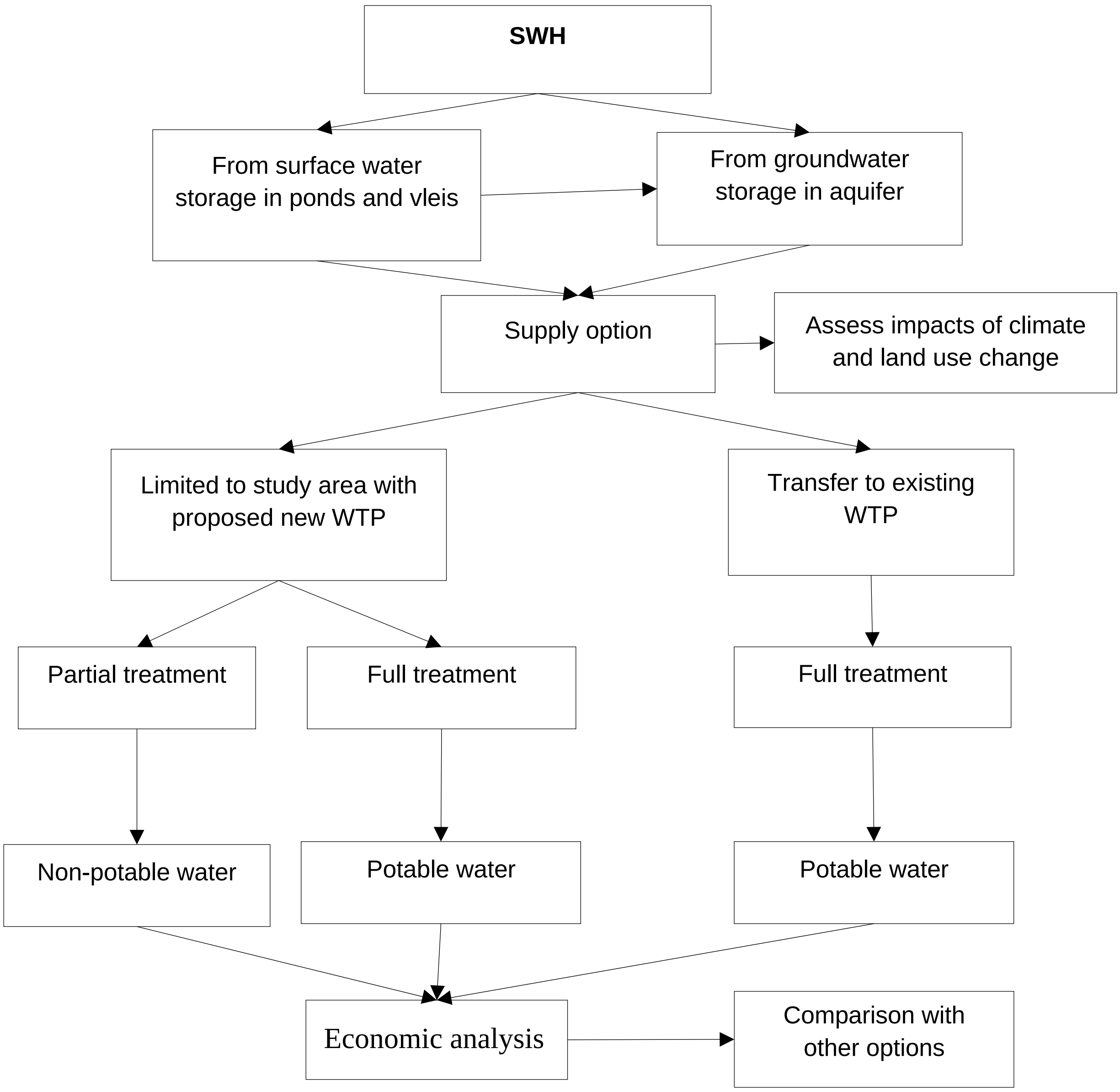

Figure 3-1: Summary of the components considered in the study

This chapter comprises 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: directly from surface storage or indirectly via groundwater aquifers. The assessment of the prospects for SWH from surface water storage included modelling the hydrological process to estimate the quantity of the stormwater resource, identifying the appropriate volumetric capacity and constraints, assessing the effectiveness of real-time control to address the challenges of storage, and evaluating 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 surface-to-groundwater transfer (managed aquifer recharge) and estimate recharge volumes. The second step was to model 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 identifying 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 to identify areas where the water would be used economically. Initially, the study assessed the potential to utilise stormwater for non-potable water needs such as agriculture, the irrigation of residential gardens and open parks, and for toilet flushing. Then, assessments were made of the opportunity and cost of the stormwater treated to potable standards and distributed locally in the study area or the transfer of partially treated water (to non-potable levels) for blending with raw water at an existing water treatment works. The total costs of the production and supply of stormwater as potable or non-potable water were determined and compared with other sources, such as treated effluent and seawater desalination, which the 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 were the availability of 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% were retention ponds and 7% were 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 the preservation of biodiversity (Rohrer, 2014).

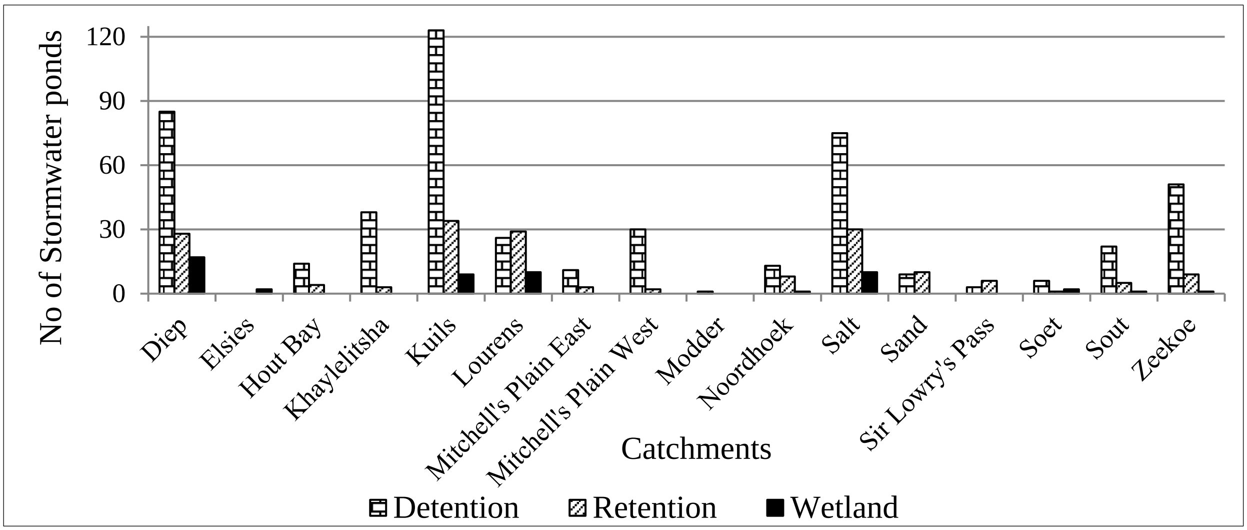

Figure 3-2: Stormwater ponds per catchment (Rohrer, 2014)

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 amenities. Some of the stormwater ponds are presented in figures 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

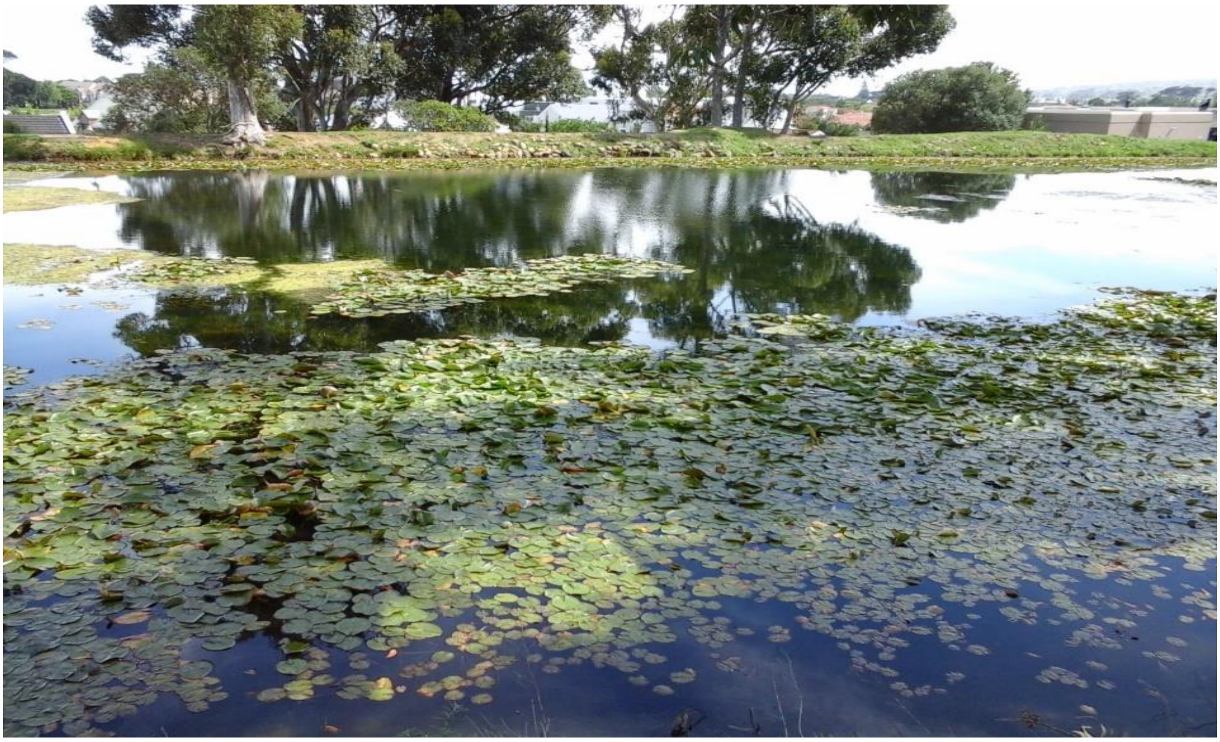

Figure 3-5: A typical retention pond

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 in which there were many 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:

- Open water bodies such as vleis (shallow lakes) and stormwater ponds with potential for adaption to store and supply stormwater
- The availability of good-quality data to model the hydrological process in the catchment
- A catchment with characteristics such as an unconfined aquifer with high porosity, hydraulic conductivity and groundwater yield (potential ground water source) that provide opportunities for surface-to-groundwater transfer
- Proximity to potential stormwater users, e.g. agriculture, residential and public parks
- Proximity to an existing WTP to minimise the cost of conveyance for the 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 their 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 is situated at the south-western tip of Africa, while the Zeekoe catchment is located in the south-central part of the City of Cape Town (Figure 3-9).

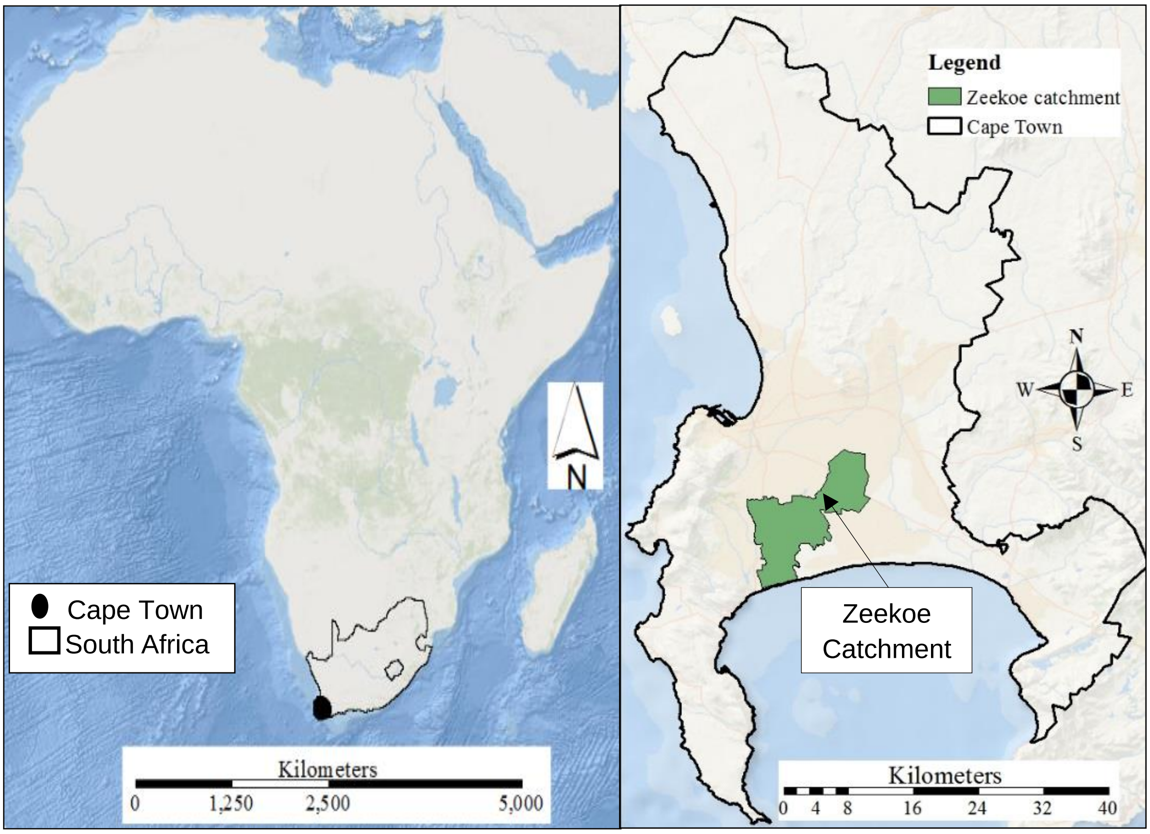

Figure 3-9: Zeekoe catchment in Cape Town (CCT, 2012)

The Zeekoe catchment was chosen from the various catchments in the 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 with a suitable location for the infiltration of stormwater to augment an unconfined aquifer. The catchment is located in an area with sandy soils with relatively high groundwater flow rates, as shown in Figure 3-14. The study area is relatively flat terrain (an average slope of less than 3%) with deep unconfined aquifers, ranging in depth from 20 to 50 m, that offer an opportunity for MAR&R to store stormwater for harvesting later as groundwater. The aquifer had previously been identified as a potential groundwater resource in various studies (e.g. Tredoux et al., 1980, Seward, 2009, 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.

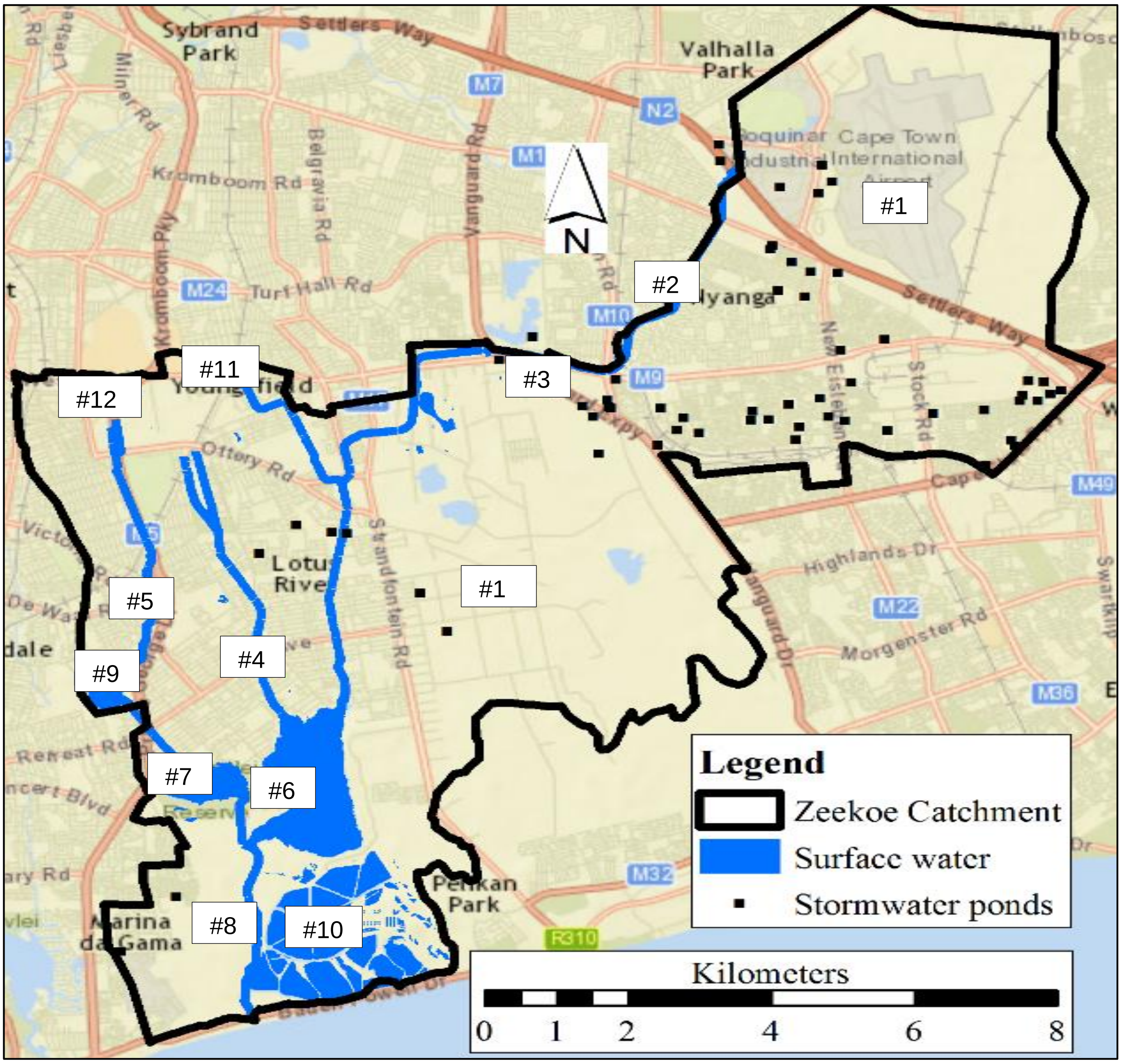

Figure 3-10: Main features in the Zeekoe catchment (CCT, 2012)

The main drainage channel of the catchment is the Great Lotus (#2), which rises in the precinct of the Cape Town International Airport (#1), a significant feature in the study area. It flows through a large portion of the catchment into Zeekoevlei (#6), the Zeekoe Canal (#8), and finally discharges into the ocean. The other streams in the Zeekoe catchment are the Little Lotus (#4) and the Southfield Canal (#5), which 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 photographs taken along the Great Lotus River are presented in Figure 3-11.

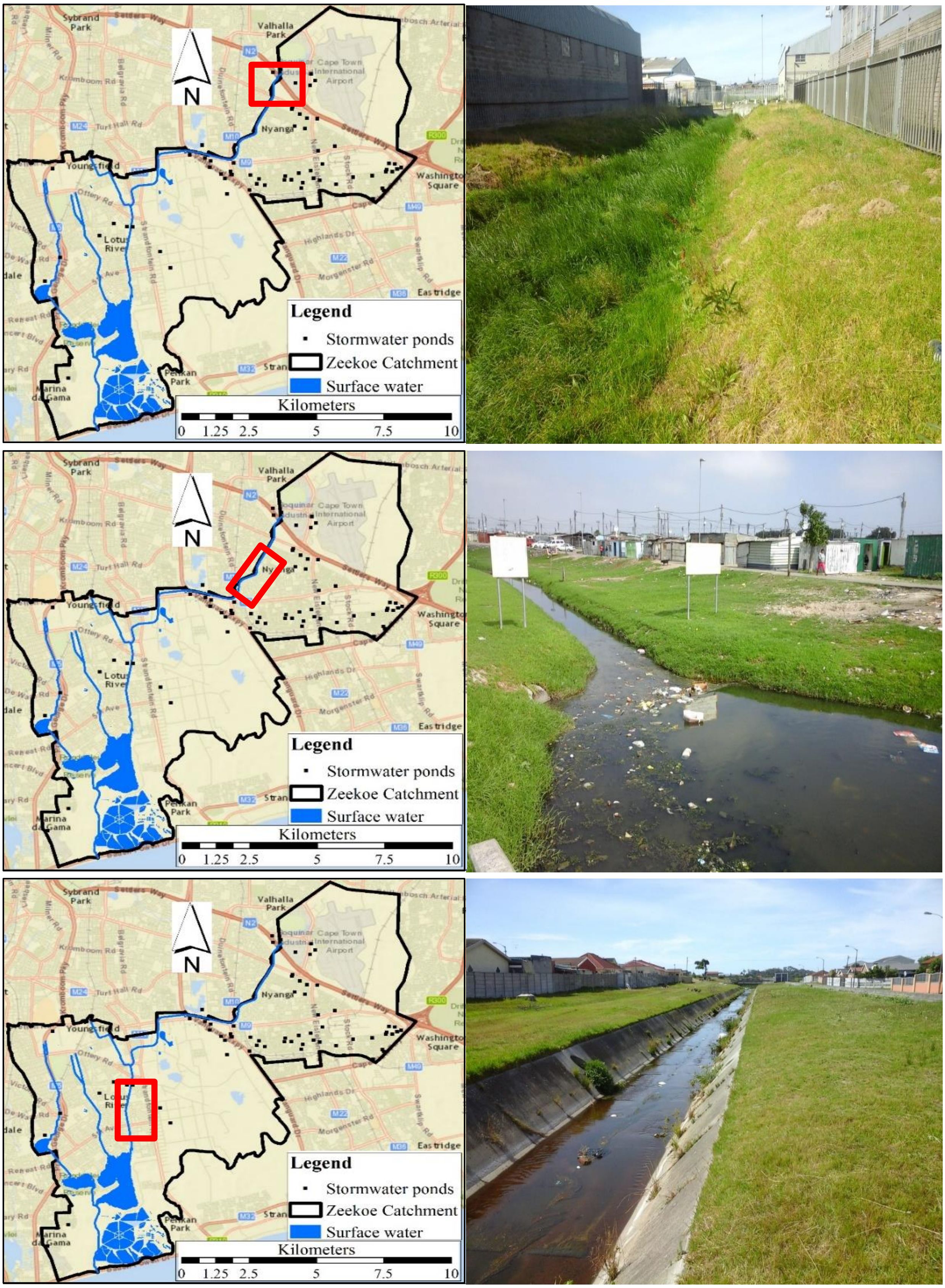

Figure 3-11: Views of typical sections along the Great Lotus River

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 (those labelled #6, #7 and #9 in Figure 3-10) were unconnected, with no rivers (Brown and 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 not linked directly to the ocean either, 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 and Magoba, 2009).

In the process of urbanisation, naturally occurring marshland was converted into largely impervious pavements. This hardening of the earth's 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 canals (Brown and 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 and Dallas, 1995) and a generally high water table (Ziervogel and Smit, 2009).

The Zeekoe catchment is now largely defined by stormwater drains. The Great Lotus River, which was mainly constructed to drain Cape Town International Airport, also drains the adjacent industrial area (the Boquinar industrial area), as well as densely populated informal settlements and light industrial, and low- to middle-income residential areas, before discharging into Zeekoevlei. Along the way, it 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 and Magoba, 2009). The Great Lotus carries the highest pollution load of all the streams in the area as a result 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 lined with concrete, some upstream sections are lined with earth, 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, and 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 to improve the flow in the channels for flood management.

3.5 CHARACTERISTICS OF THE STUDY AREA

In CCT, the mean annual precipitation varies from 350–2,500 mm, distributed as shown in Figure 3-14, with the rainfall in the Zeekoe catchment ranging from 500–1,100 mm. The soil type is mainly sandy with typical borehole yields in the range of 0.5 to 5 ℓ/s (Figure 3-14).

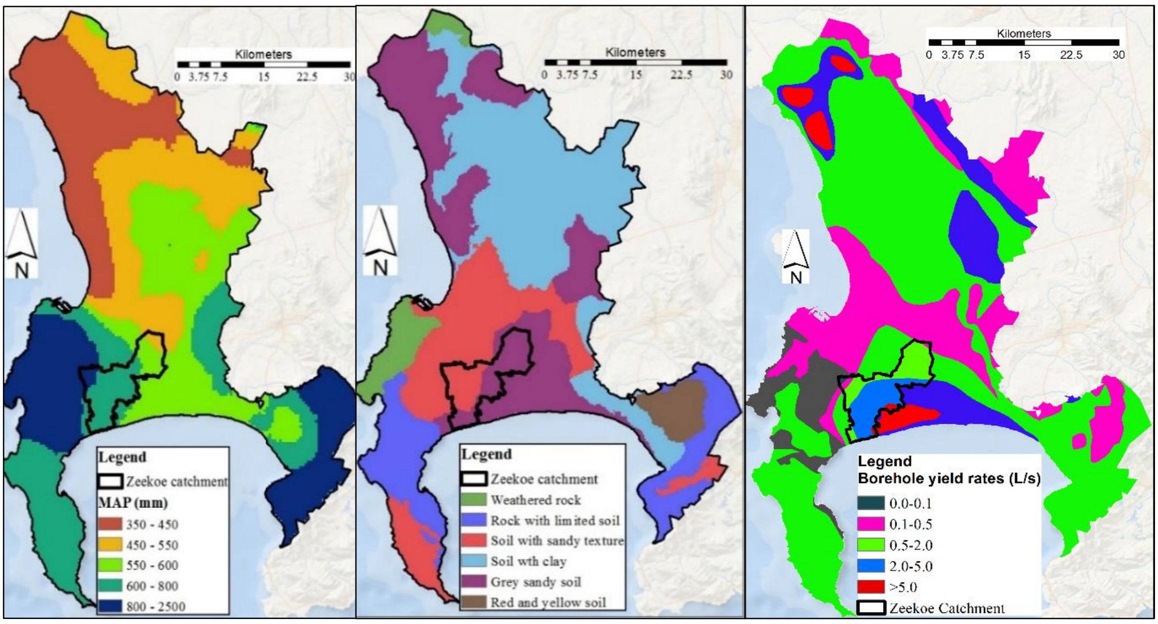

Figure 3-14: Rainfall, soils and groundwater yield in Cape Town (CCT, 2015)

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

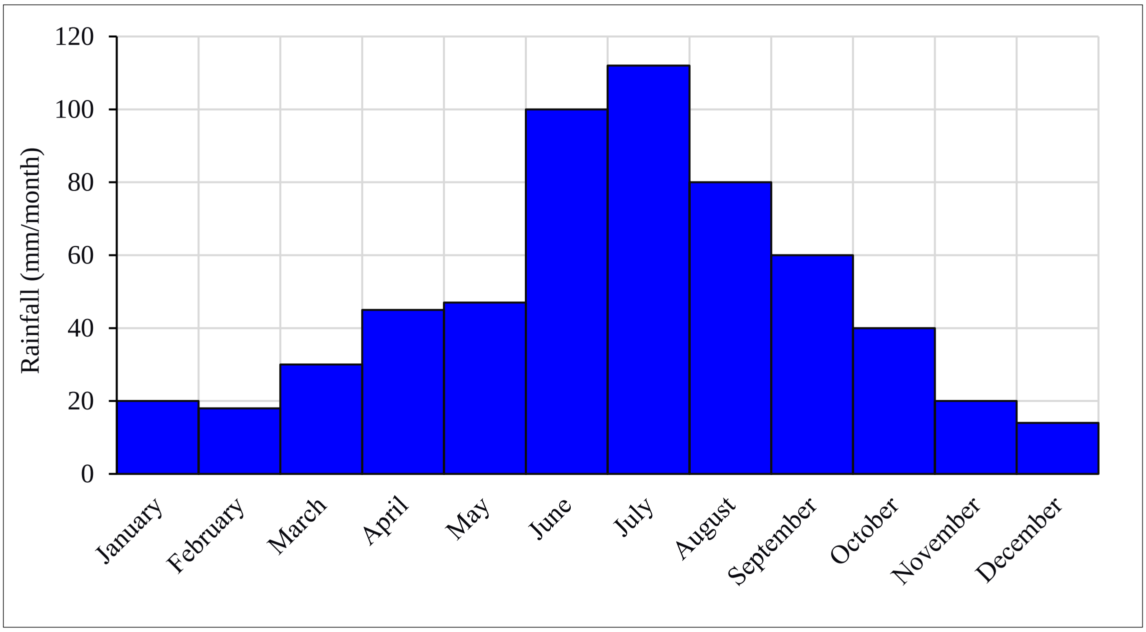

Figure 3-15: Mean monthly rainfall in the study area (CCT, 2015)

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

Item	Data	Location	Resolution	Period	Sources
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*, total suspended solids, temperature, total nitrogen, total phosphorus, electrical conductivity, dissolved oxygen, pH level	Zeekoe catchment	Monthly grab samples	1992–2015	CCT
Water quality data	Arsenic, cadmium, chromium, lead, mercury, *E. coli*, temperature, electrical conductivity, total dissolved solids, pH level	Zeekoe catchment	Daily grab samples for five days	20 June 2016 to 24 June 2016	Swiss Tropical and Public Health Institute and UCT sampling

3.6.2 Rainfall and flow data monitoring

There are various rainfall measuring stations in and around the study area. Three of these stations are managed by the South African Weather Service (SAWS) with long time series, i.e. greater than 10 years collected at a daily time scale. These include Cape Town Airport (1992–2015), Rondevlei (1952–2015) and Mitchell'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 within a limited period, i.e. 2012–2015. The stations include Southfield, Hanover Park, Cape Flats Waste Water Treatment Works (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 within a limited period, i.e. 2012–2015.

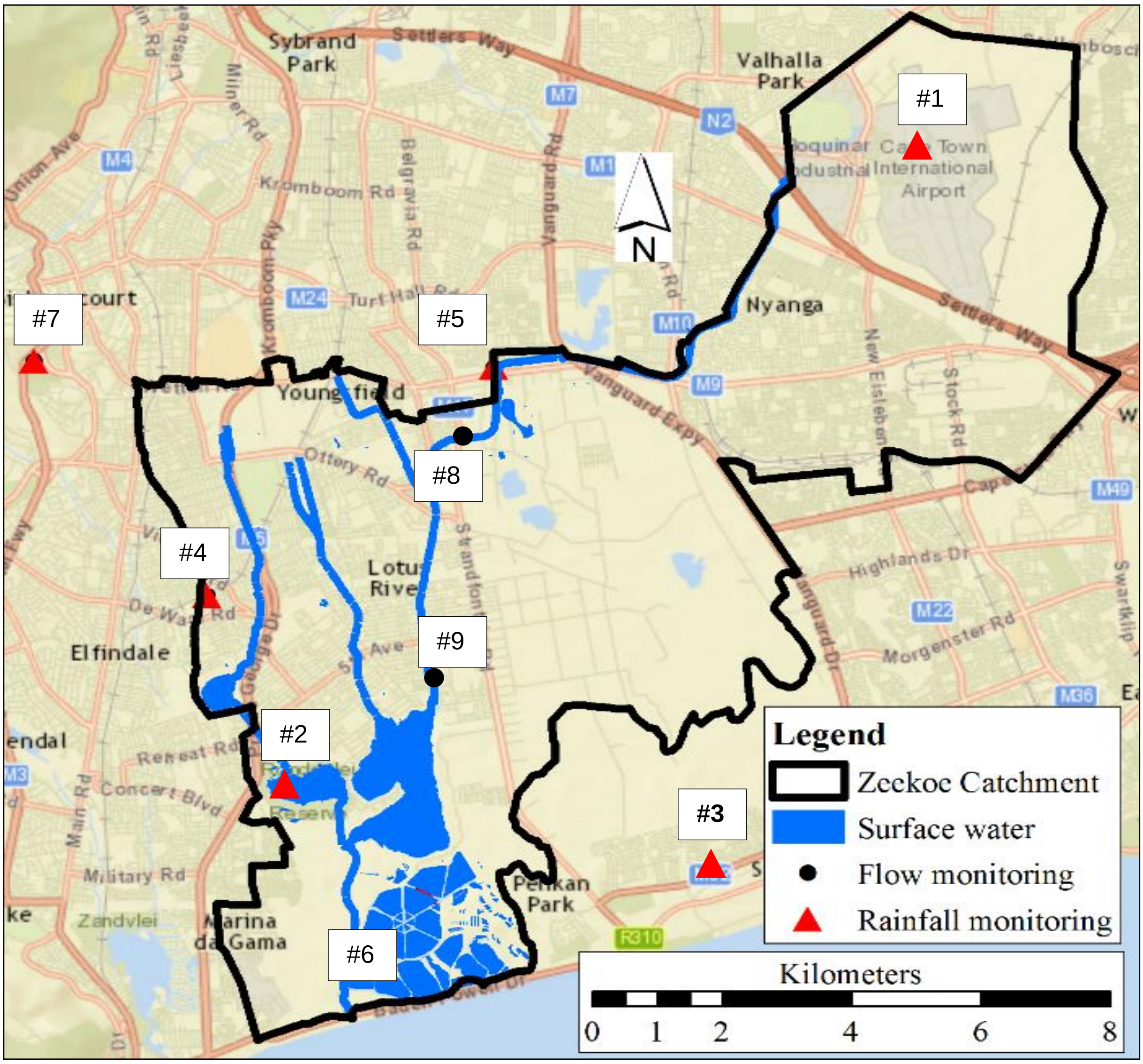

Figure 3-16: Rainfall and flow measuring stations (CCT, 2015)

3.6.3 Evaporation data

Evaporation is a critical process in hydrological modelling as it represents 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 values from empirical methods, e.g. Hargreaves (commonly used in hydrological models).

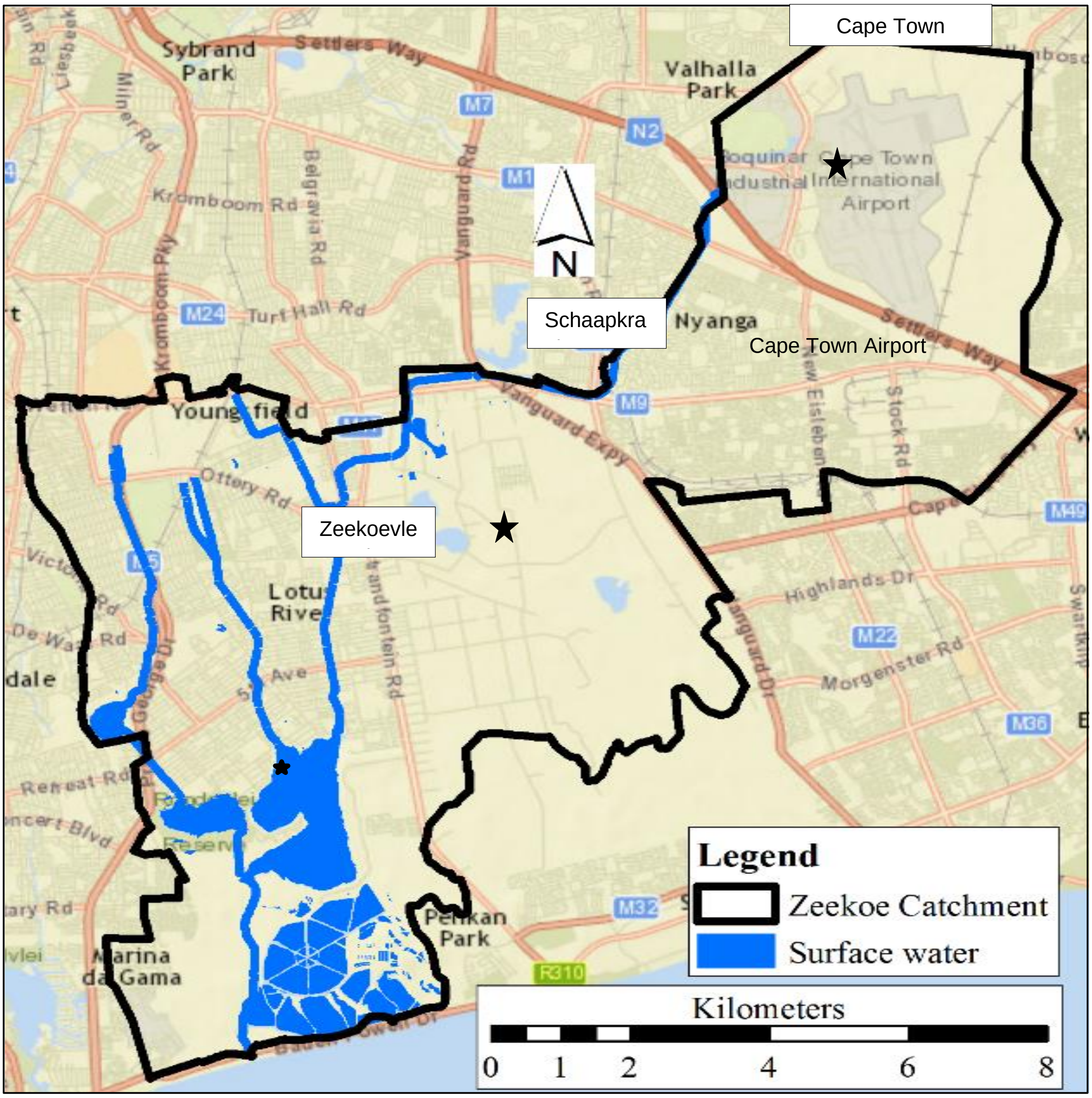

Figure 3-17: Evaporation measuring stations (CCT, 2015; DWS, 2015)

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 26 models using the statistically downscaled Coupled Model Intercomparison Project Phase 5 (CMIP5) was acquired from the UCT's Climate Systems Analysis Group (CSAG) for the Rondevlei and Cape Town Airport stations. The statistically downscaled data is from the 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 the 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 rainfall, evaporation, temperature, river flow, land use and soil. This section discusses the data collected and used in surface water modelling. In the selection of a hydrological model for the study, the following factors were considered:

- A tool that can comprehensively model an urban catchment at high spatial and temporal resolution
- A tool that makes use of the available data and physical characteristics of the study area
- A tool that provides the opportunity for real-time control analysis
- A tool that provides the opportunity to model surface-to-groundwater transfer
- Widely used software in South Africa and internationally with user support
- Software that is available at low or no cost (i.e. research or education edition)

The tools assessed included the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) (eWater, 2013), MIKE SWMM (DHI Denmark, 2014), City Drain (Achleitner et al., 2007), System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) (Lee et al., 2012), the 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 and 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 to estimate harvested stormwater volume and evaluate stormwater quality improvement during storage and conveyance (Hutchins et al., 2017). Although reviews (e.g. Breen et al., 2006, Akram et al., 2014, Bach et al., 2014) have shown that most models focus on only one component of urban drainage, some models, such as MUSIC and PCSWMM, have shifted towards an assessment of the integrated natural urban landscape and engineered water cycles. After evaluating 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. extending the detention of water in a pond and the opportunity for the water 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 in 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 a 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. the unsaturated soil layers), percolation (i.e. infiltrated water into groundwater layers), the interflow between the groundwater and the drainage system, the non-linear 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 with 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 existing stormwater ponds. To enhance infiltration and augmentation of the groundwater, the stormwater ponds were modelled as infiltration basins.

The second part included the 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, so 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 would benefit from the 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 by a matrix (Boskidis et al., 2012). The main limitation of MODFLOW in the application of the 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 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 the IWAS-Toolbox that connects SWMM with OpenGeoSys (a sub-surface 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 and Sayeed (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 that provide 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).

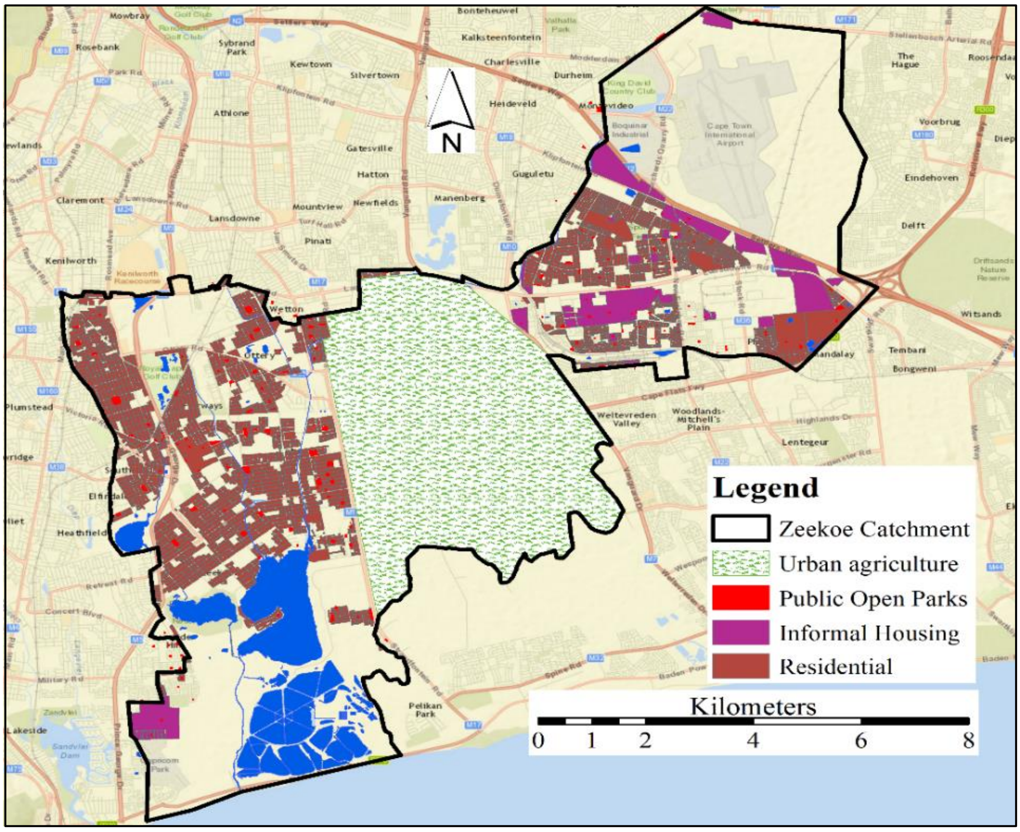

Figure 3-18: Land use in the Zeekoe catchment

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, and the irrigation of residential gardens, open parks and urban agriculture. Alternatively, the harvested stormwater would be treated to potable water standards, distributed with the existing reticulation system and used for all requirements in the study area. The modelling of stormwater harvesting from surface water storage was based on 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 the water abstraction, treatment and distribution processes are discussed in Chapter 7.

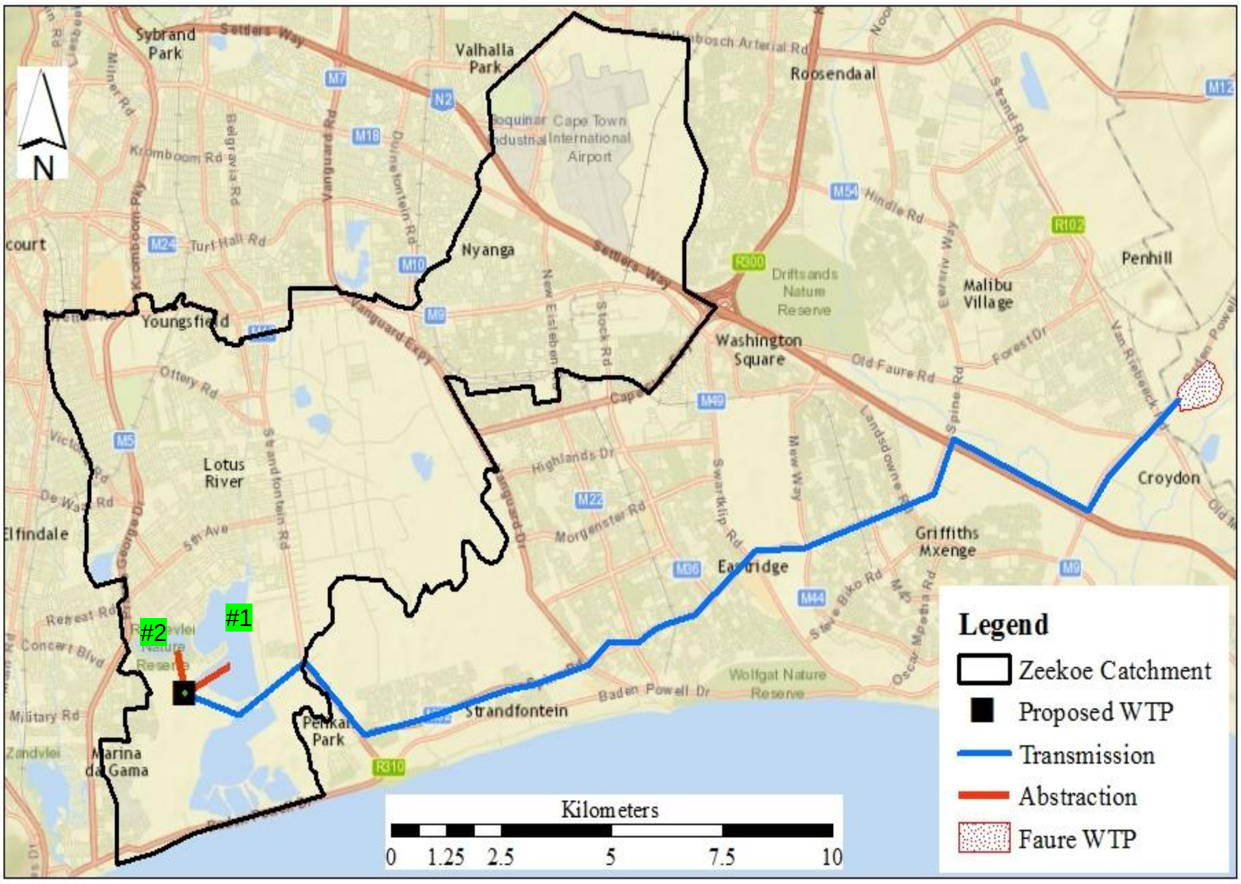

Figure 3-19: Centralised system with abstraction and conveyance

3.10 SUMMARY OF THE METHOD

The method adopted can be summarised as follows:

i. The hydrological process in the Zeekoe catchment was modelled with the aid of PCSWMM software to quantify the stormwater resource. The opportunity for the 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. The use of RTC was assessed to determine the potential storage enhancement, while safeguarding the original purpose, i.e. flood control, and to address the identified challenges of limited capacity in the existing ponds to capture a significant portion of the runoff.

iii. Even with the application of RTC, there was limited additional benefit, and considerable amounts of stormwater were 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 storage in existing ponds only to give time for the water to infiltrate into the aquifer.
iv. An assessment was undertaken to determine the viability of two water supply options: harvested stormwater treated at 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 at a proposed new WTP, an assessment was undertaken to determine the treatment requirement and what could be delivered at each stage of the system as non-potable or potable water.
vi. The study also assessed likely impacts of climate and land use change on harvested stormwater in the future.
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.

CHAPTER 4: STORMWATER HARVESTING FROM SURFACE STORAGE

In this chapter, the method and associated results relating to the prospects for SWH from surface water storage 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 rainfall, evaporation, temperature, river flow, land use and soil. This section discusses the data collected and applied in 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 in the Zeekoe catchment ranges from 500 to 1,100 mm. To reasonably represent the significant range and variability in rainfall, various monitoring stations were used as input in modelling the hydrological processes of the catchment, as shown in Figure 3-7. The data from the CCT was at five-minute 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 was 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 4-8, 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 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 converted to ET_o through an empirically derived pan coefficient (k_p) using Equation 4-1 (FAO,1998; Savva and Frenken, 2002):

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

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 and 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 and 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	1,000	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	1,000	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	1,000	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	1,000	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 figures 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.

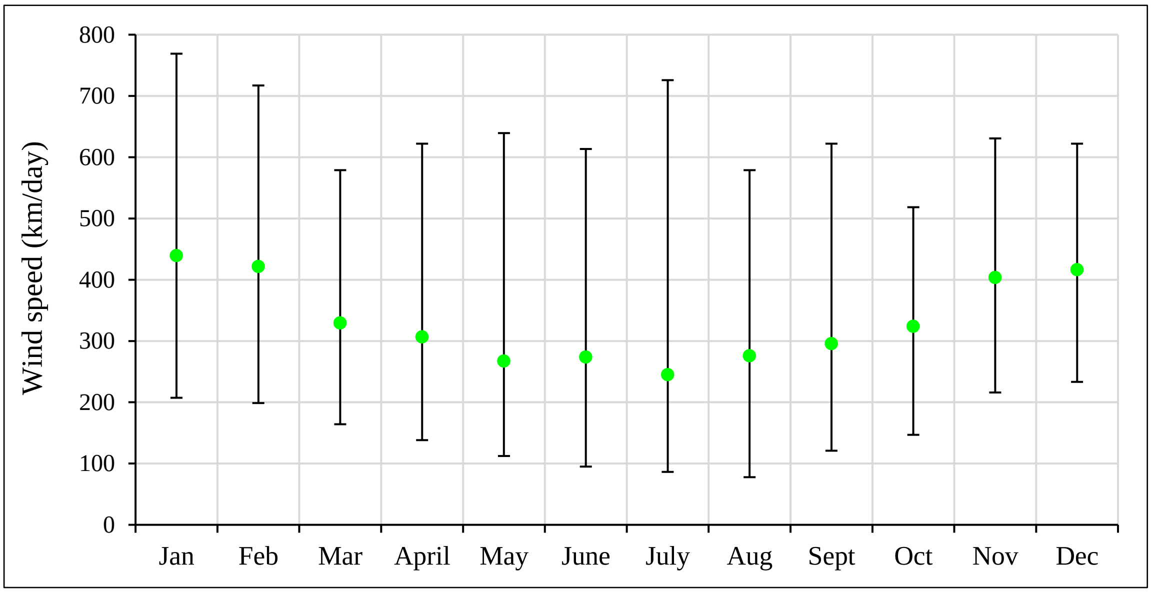

Figure 4-1: Mean daily wind speeds (CCT, 2015)

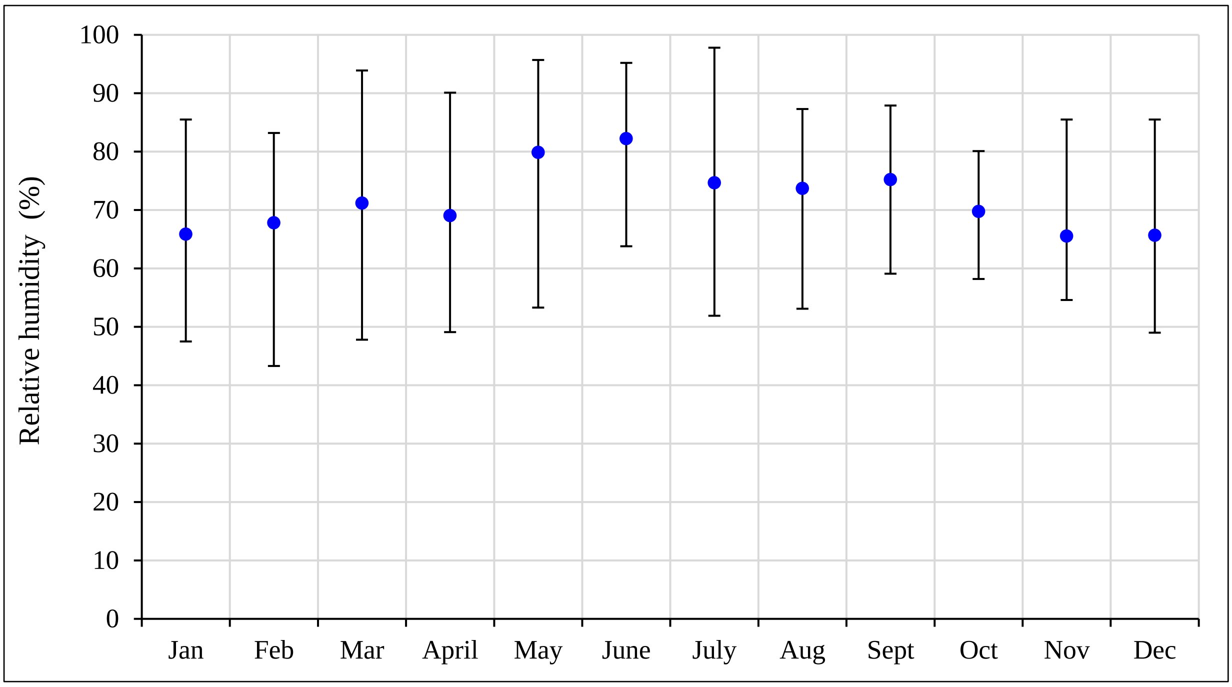

Figure 4-2: Mean daily relative humidity (CCT, 2015)

Jensen et al. (1990) compared results from directly measured ET_o 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 a 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 (the mean temperature data is presented in Figure 4-3:).

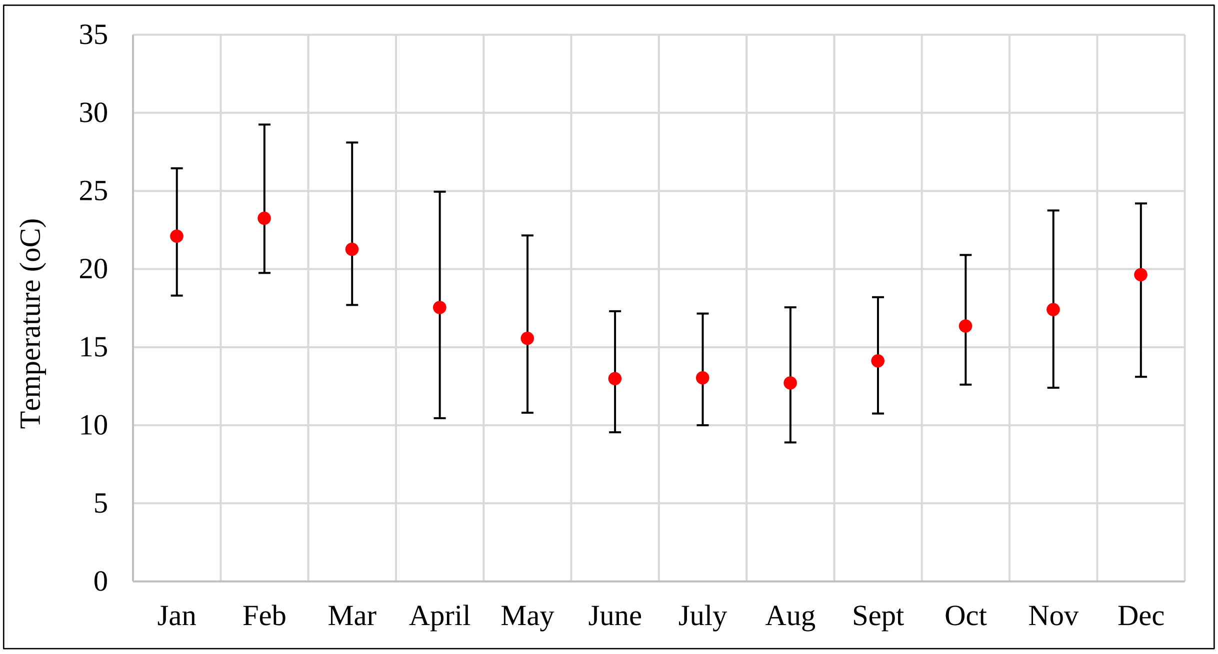

Figure 4-3: Mean daily temperature (CCT, 2015)

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

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

where ET_o = Evapotranspiration (mm day^{-1}); R_a = Extraterrestrial radiation (MJ m^{-2} day^{-1}); T_{mean} = Daily mean temperature (°C); TR = Daily temperature range (°C) (*i.e.* T_{max} - T_{min} where T_{max} and T_{min} are the mean daily maximum and minimum temperatures, respectively).

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

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

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

Table 4-2: Mean daily percentage of annual daytime hours and extraterrestrial 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: .

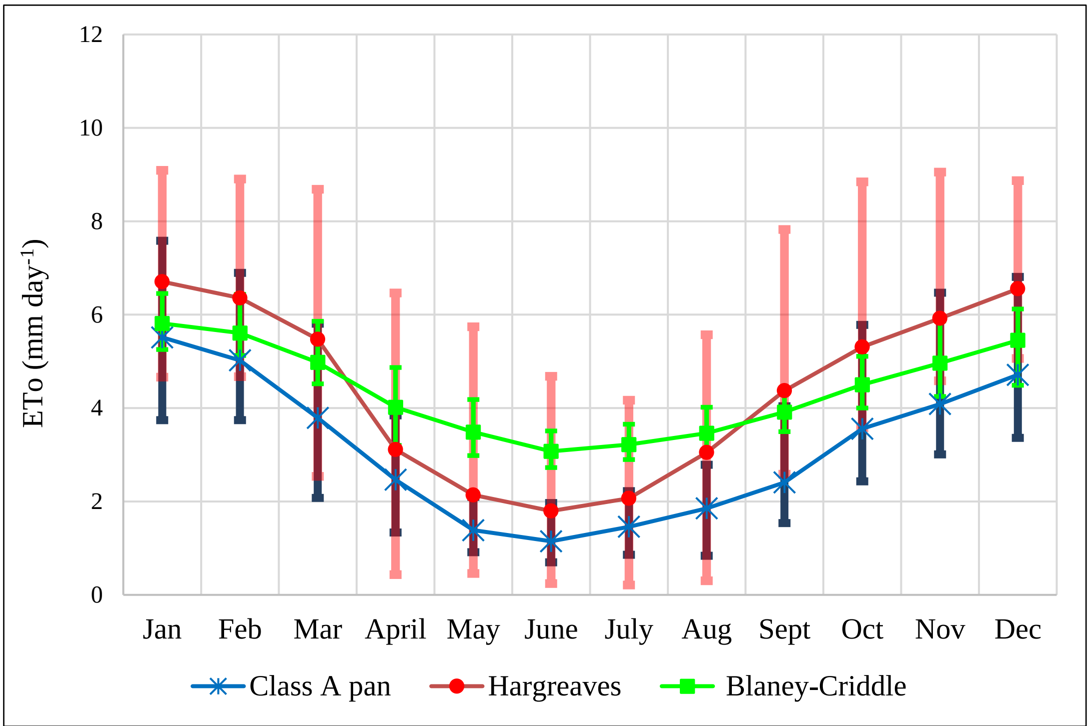

Figure 4-4: Estimated mean daily evapotranspiration values and trends

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 CMIP5 were acquired from UCT's Climate Systems Analysis Group for two stations in the study area, Rondevlei and Cape Town Airport. The five-year and 13-year moving mean of various climate models at Rondevlei are shown in Figure 4-5:.

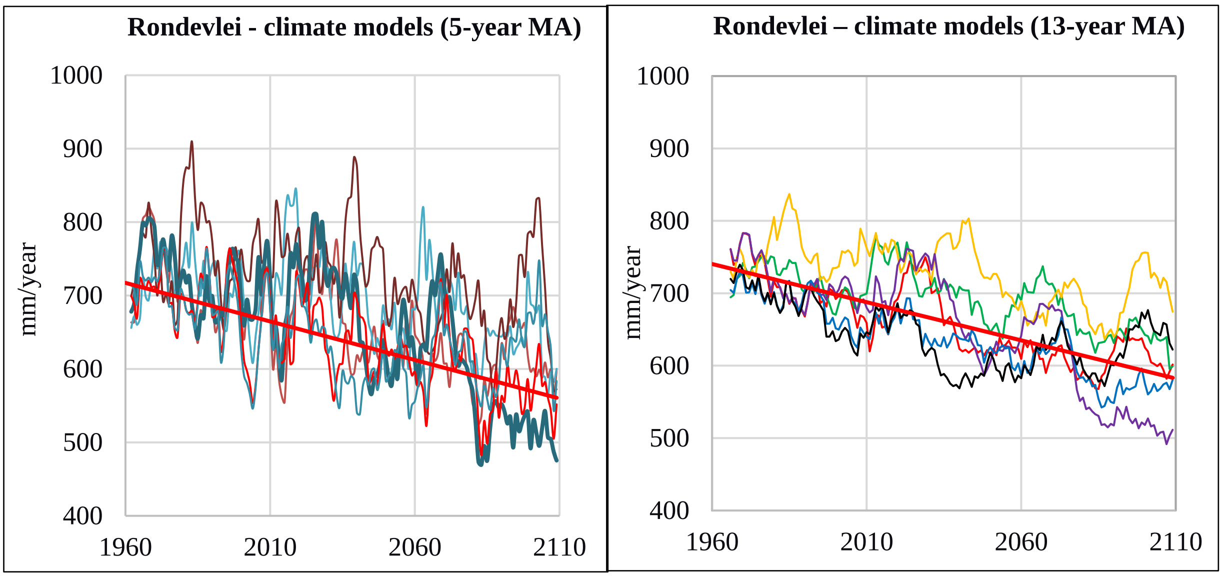

Figure 4-5: Rainfall trend time series from climate models at Rondevlei station (after Hewitson and Crane, 2006)

The climate data was from two different representative concentration pathways: 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 and 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.

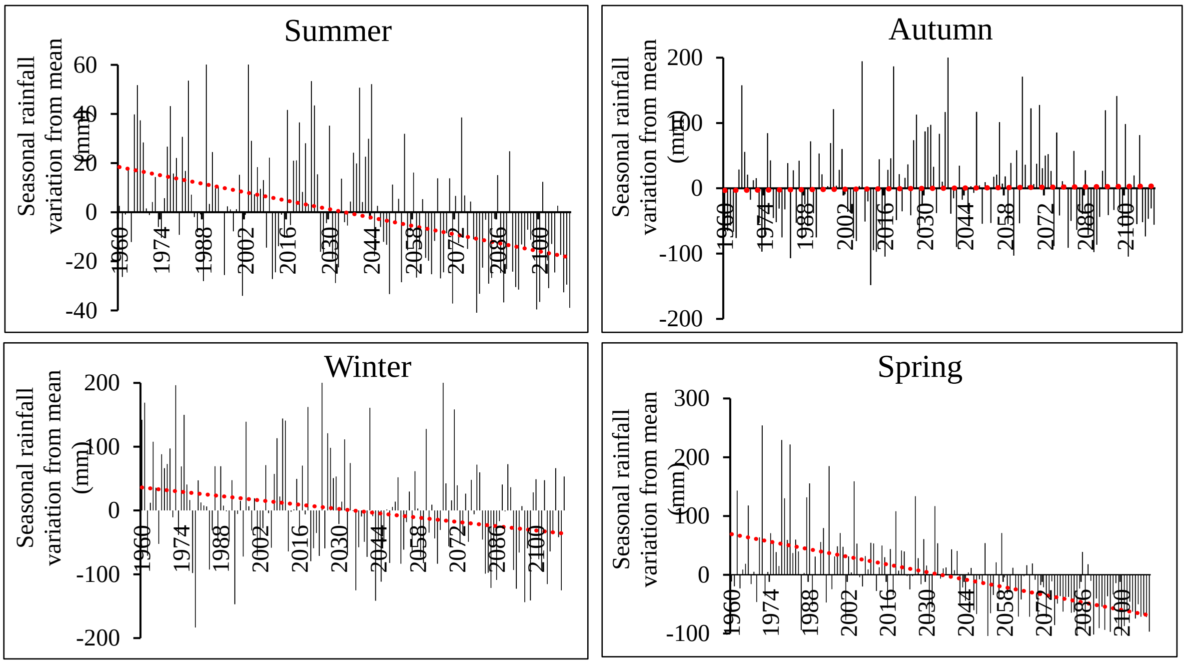

Figure 4-6: Seasonal rainfall variation from the historical mean (2006–2015) (after Hewitson and Crane, 2006)

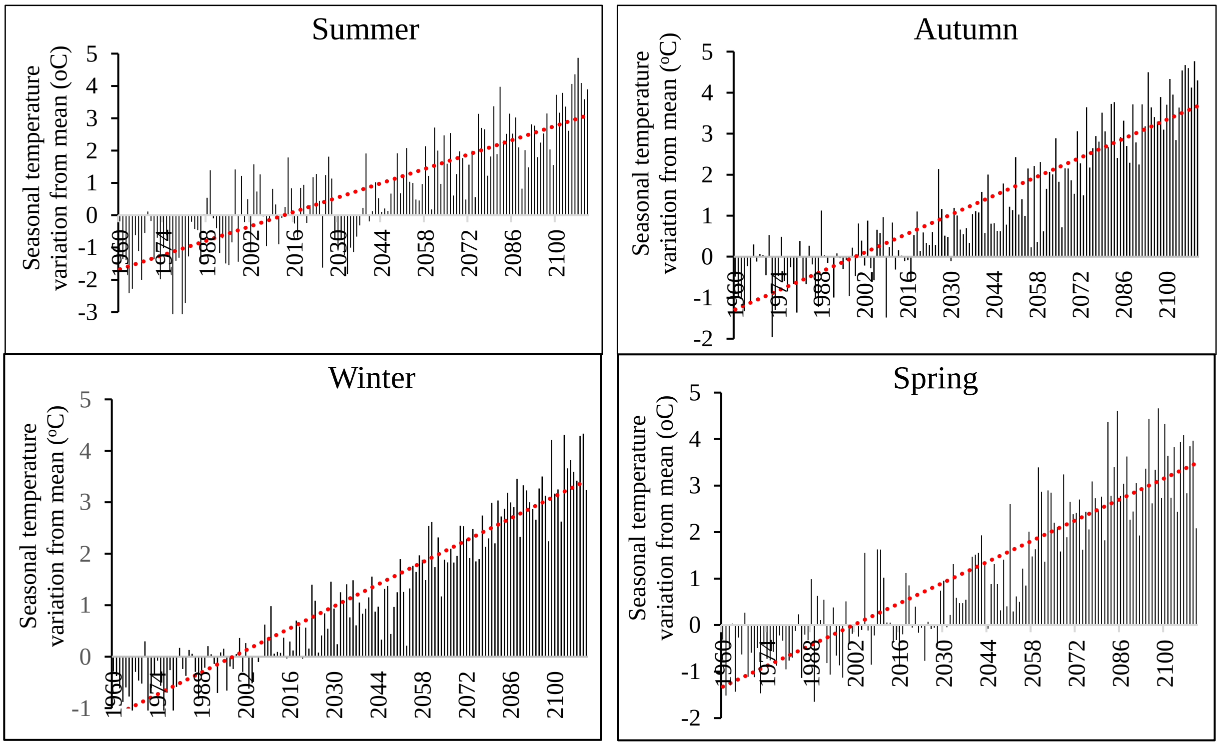

Figure 4-7: Seasonal temperature variation from the historical mean (2006–2015) (after Hewitson and Crane, 2006)

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:.

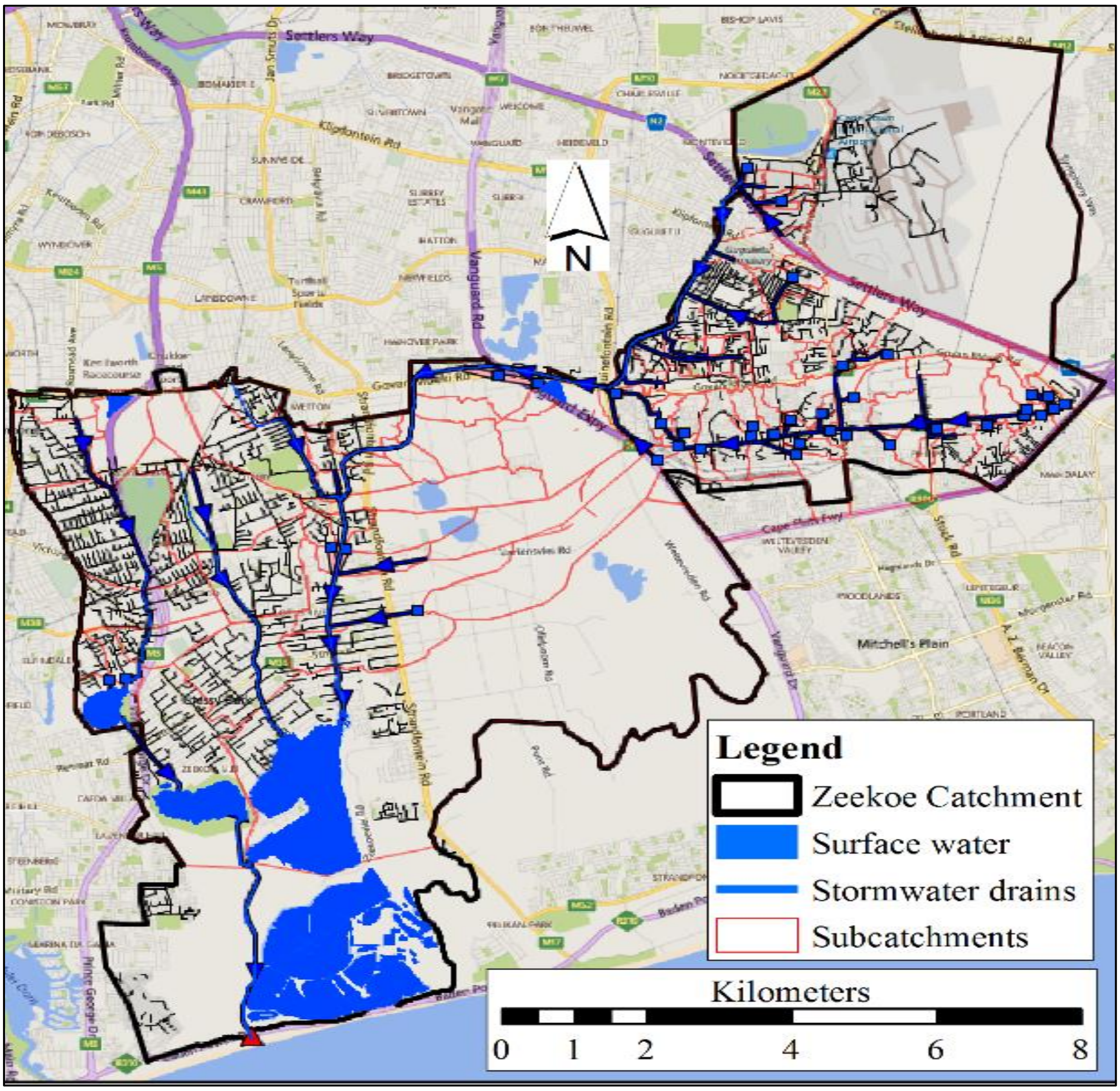

Figure 4-8: Sub catchments and stormwater network (after CCT, 2015)

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 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 light detection and ranging (LIDAR) digital elevation model (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, being confirmed with spot checks during 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 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 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 the 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 the 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 field measurements using a double ring infiltrometer (DRI), combined with laboratory experiments on samples brought back from the field. The parameters that were estimated included maximum infiltration rate in dry soils, minimum infiltration rate in saturated soils, infiltration rate decay constant, i.e. the rate at which the 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 $$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\,\frac{\partial H}{\partial x} + gA\,(S_f - H_L) = 0$$ **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 the 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 at three sites across the study area (Figure 4-9 and Table 4-3).

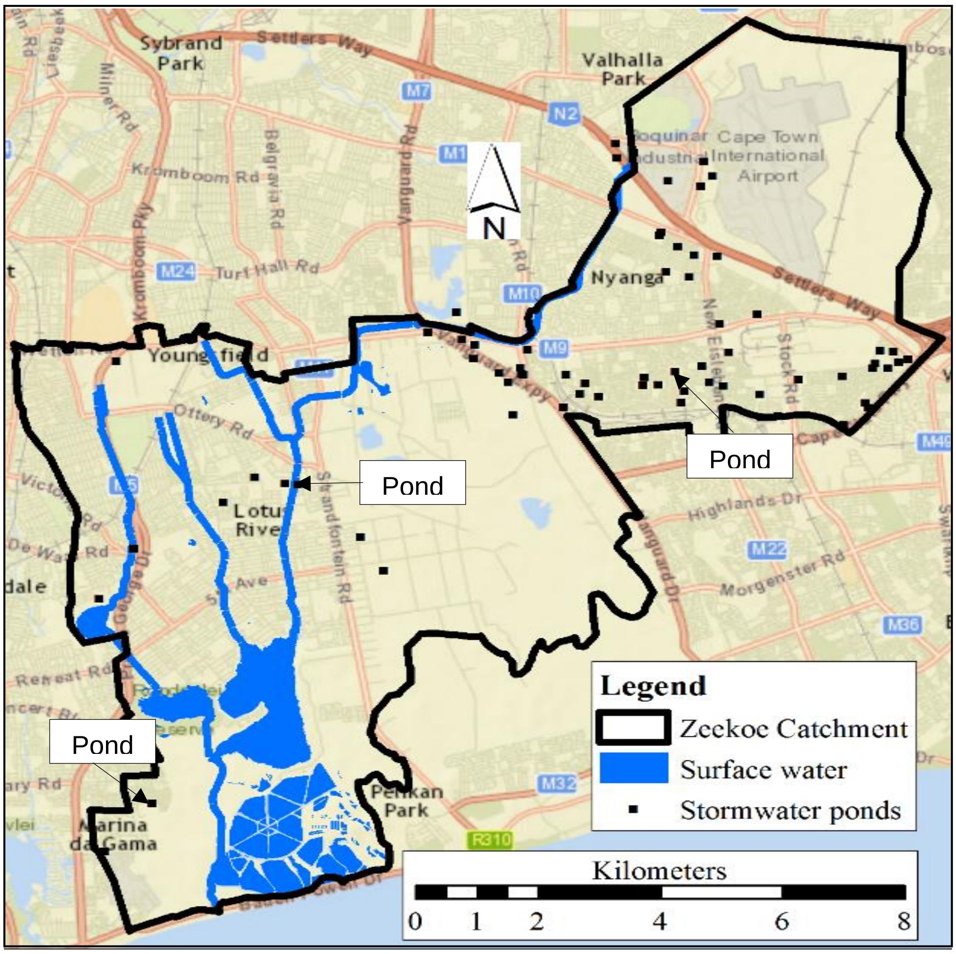

Figure 4-9: The three infiltration test sites (after CCT, 2015)

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

Pond No.	Pond type	Surface area (m^2)	Suburb	Road	Latitude	Longitude	Elevation (m above sea level)
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 six-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 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 to 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 an 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 of particles passing versus sieve size (grain size) (Figure 4-10). A summary of the findings is presented in Table 4-4.

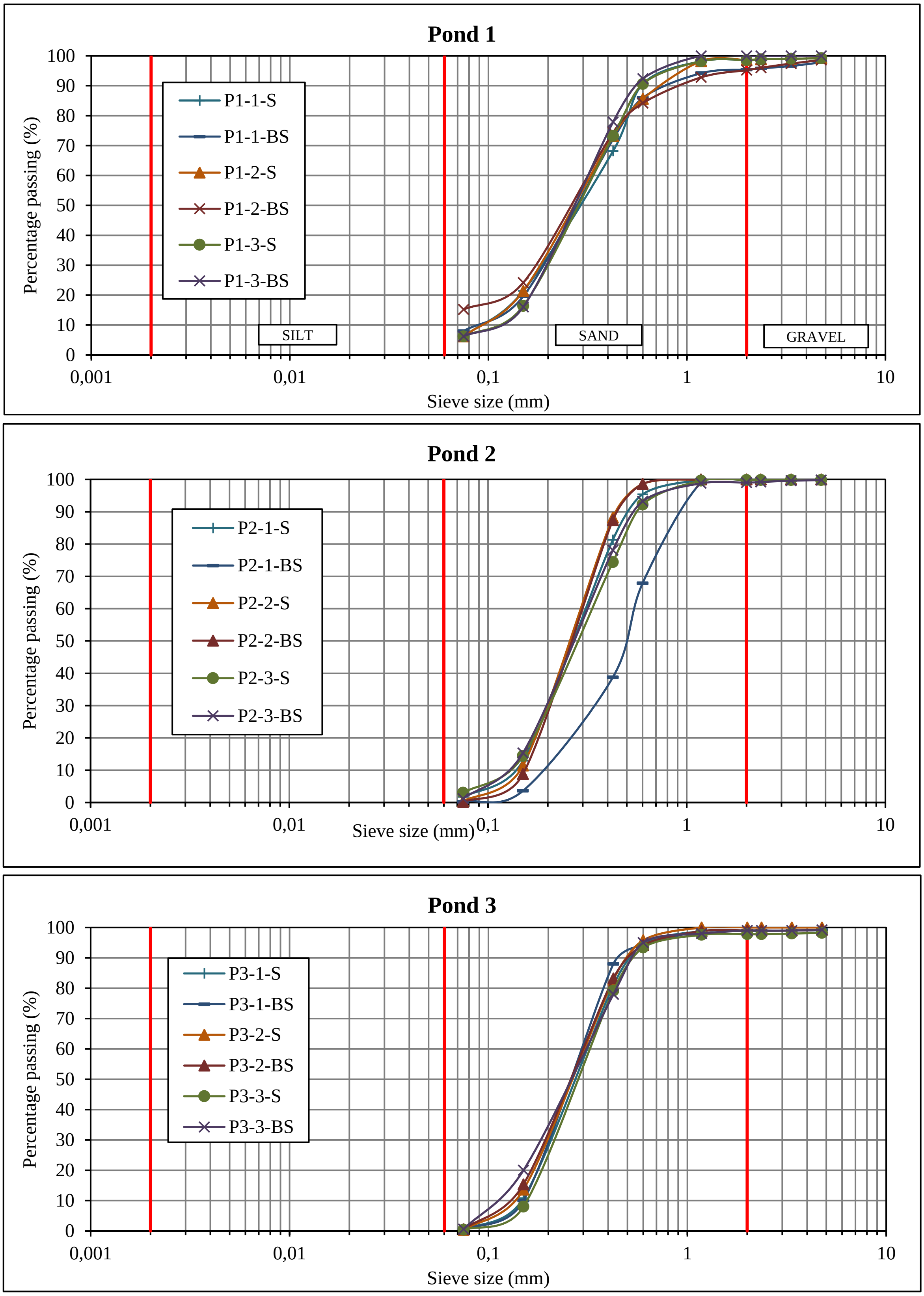

Figure 4-10: Percentage of particles passing versus sieve size (after Mavundla, 2018)

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/m^3	1,733	1,889	1,460	1,917	1,635	1,930
Saturated density		kg/m^3	2,091	2,074	1,834	2,278	1,892	1,981
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, 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).

$$f = f_c + (f_o - f_c)e^{-\lambda t}$$ **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 depends on soil characteristics and vegetation cover (hr^{-1}).

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.

$$ln(f - f_c) = ln(f_o - f_c) - \lambda t$$ **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	**fo**	**fc**	**λ**	**fo**	**fc**	**λ**	**fo**	**fc**
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 and Ampt, 1911).

$$f = k(1 + \frac{\gamma S_c}{F})$$

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 Equation 4-8 to be re-arranged as Equation 4-9.

$$f = k + \frac{n}{F}$$

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 Horton's approach in Pond #1 and #3. After equilibrium had been reached, however, Horton's approach represented the measured asymptotic infiltration rate curve better than the Green-Ampt method in Pond #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.

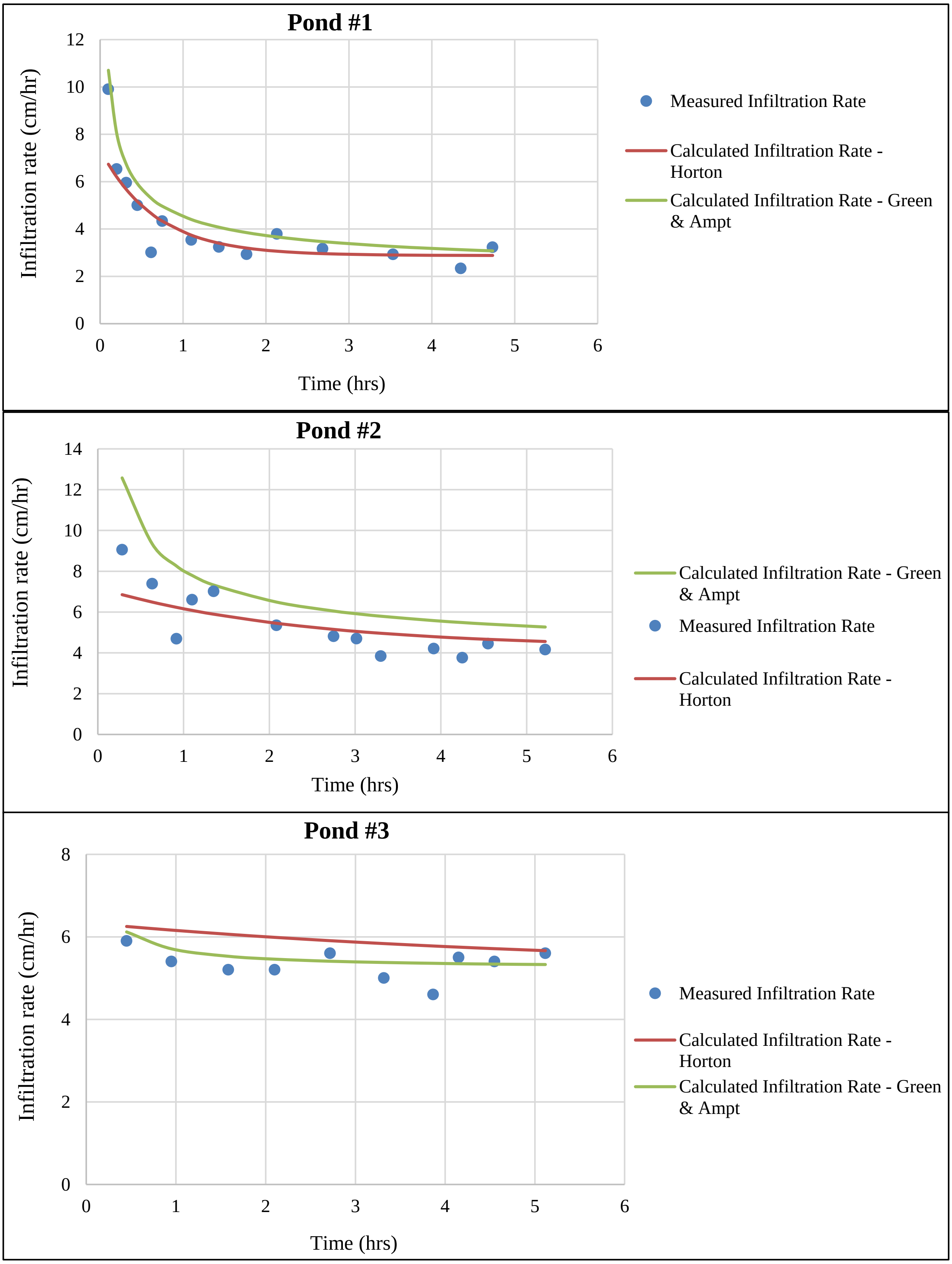

Figure 4-11: Compared measured and calculated infiltration rate (after Mavundla, 2018)

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	1.2	0.3	2.3
	n	2.4	0.1	7.4
	γ	31	30	33
	Sc	7.3	0.1	23.1
Pond 2	k	22.1	3.8	32.6
	n	29.3	9.9	48.5
	γ	40	30	45
	Sc	31.4	22.7	36.8
Pond 3	k	9.6	0.1	21.6
	n	3.9	0.6	9.7
	γ	38	32	50
	Sc	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 a 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 coefficients (R^2) 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 methods both provided reasonable approximations of infiltration rates, except in Pond #3 with its compacted soils. The final selection of the Horton 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 using at least a ten-year rainfall time series with several dry, normal and wet years 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.
v. All the calibrated parameters were inspected to confirm that they were within acceptable ranges as recommended in "Rules for responsible modelling" (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 the Nash-Sutcliffe Efficiency method, percent 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, coefficient of determination, standard error of estimation, simple least squares (LSE), simple least squares dimensionless (LSE dim), root mean square error and root mean square error dimensionless (RMSE dim). Since the model's 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). 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).

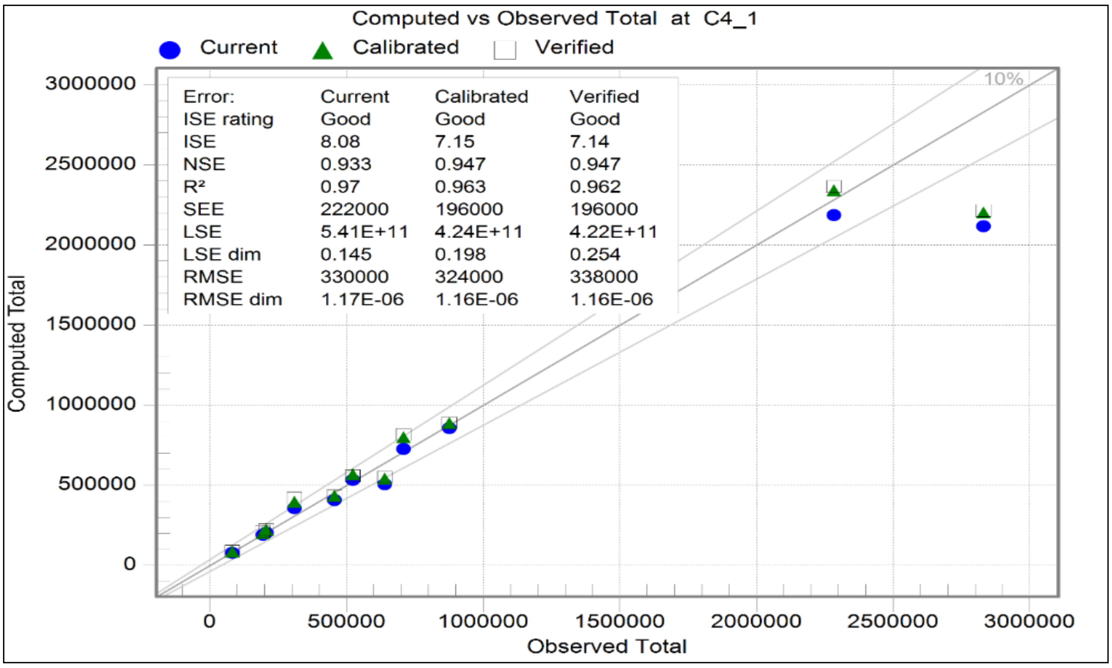

Figure 4-12: Calibration and verification results of flow volume totals

4.2.7 Model verification

Verification of the model outputs was undertaken to confirm the reliability of the results with regard 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 scatter plot in Figure 4-12:, with standard deviation represented by the solid lines, allows a visual assessment of the correlation relationship between the computed model results and the 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^o bi-sector line shown in Figure 4-12:. The presence of high scatter (i.e. a large deviation from the 45^o bi-sector line) 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^o bi-sector line was minimised to 10%. A summary of the results from the calibration and verification processes, including model continuity errors and routing continuity errors, is 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 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 techniques, as discussed in Section 0, to determine if the effective storage could be increased as used for SWH and supply.

4.3.2 Selection of the appropriate real-time control 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) managed by the National Centre for Environmental Prediction (NCEP) and used for the RTC application. The

GFS provides global forecasts up to two weeks in a spatial form. The GFS 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 6 May 2011 (system commencement date) to two weeks after the date of download. The available GFS forecast data was 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 events, as shown in Figure 4-13:. Although the difference in magnitude of the 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 the recorded data.

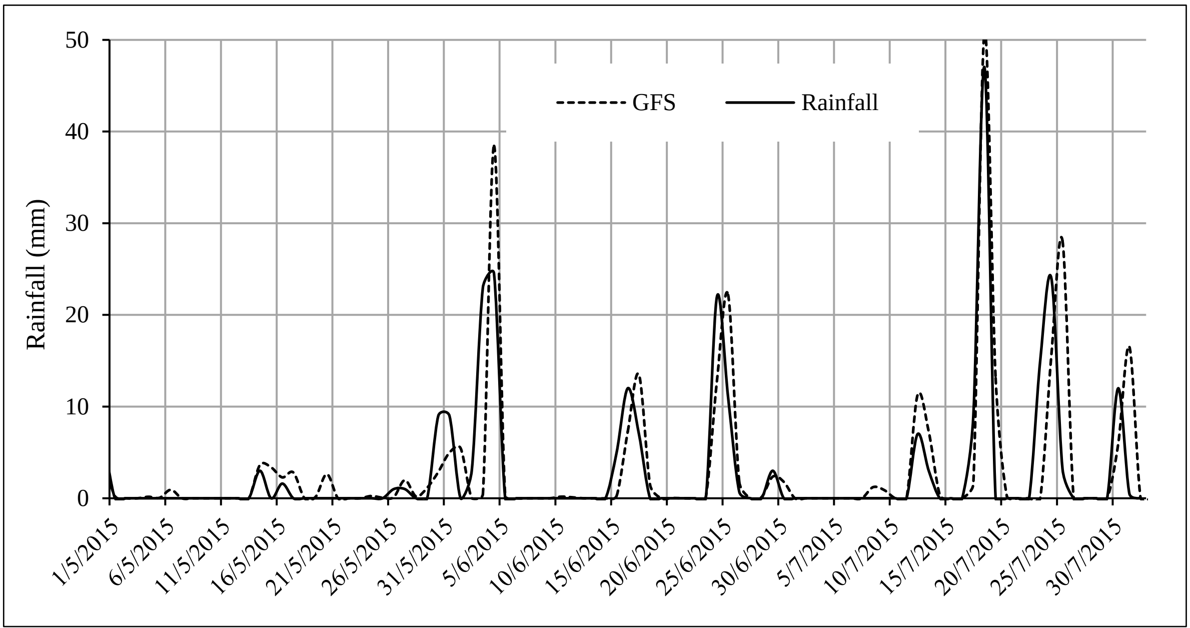

Figure 4-13: Comparison of GFS and measured rainfall data (CCT, 2015; NOAA, 2017)

The disadvantage of an overestimation in the forecast would be the release of water from storage without the subsequent occurrence of a flood. Since one of the main aims of RTC is to ensure the release of water from storage, whenever rain was forecast 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 the 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:** This 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 to 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)

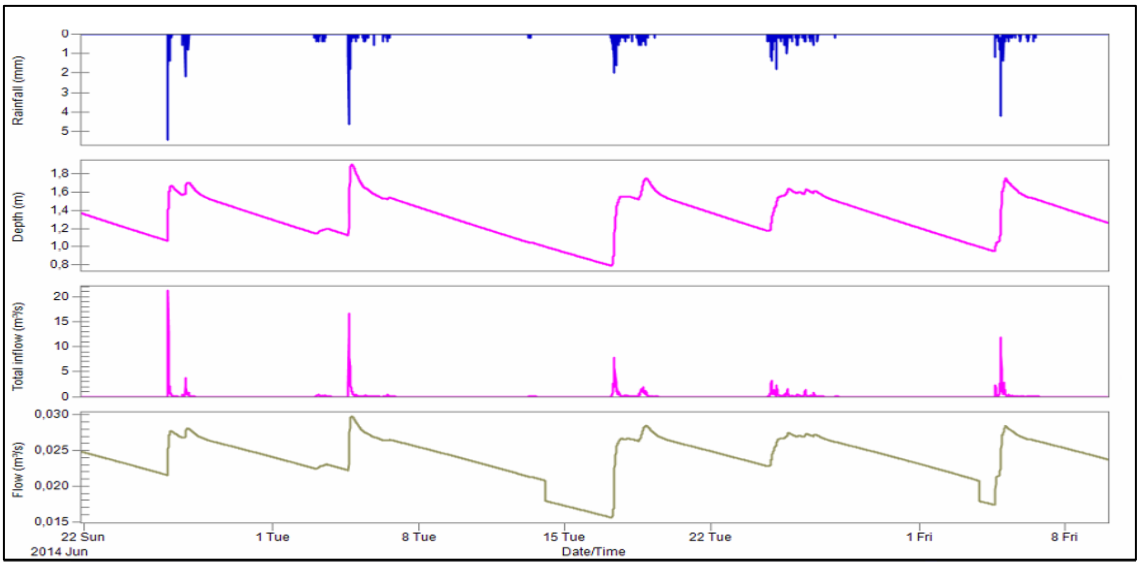

Figure 4-14: Sample results with control rules linked to specific values

The use of the option is common due to the simplicity and straightforwardness of site-specific control with any errors limited to the site and independent of the whole system (García et al., 2015; USEPA, 2004). The downside of this 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 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 ≥ 31/05/2015
  and simulation date ≤ 05/06/2015
  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 forecast rainfall event, which would generate runoff that exceeds the available capacity. The success of the option depends on the reliability and accuracy of the timing and magnitude of the forecast rainfall data. As shown in Figure 4-13:, the difference between forecast and measured data would result in an inaccurate determination of the "open and close" periods.

- **Proportional-integral-derivative 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 the 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(τ)* = 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 was no forecast 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 was forecast rainfall that exceeded the 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) were given 0 values. The P value was then changed stepwise to values such as 0.1, 1, 10, -0.1 and -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-level 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 the anticipated flow from forecast rainfall, i.e. RTC 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 the 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 simulation time and was thus suitable for the study when the dynamic management of the outlet was needed.

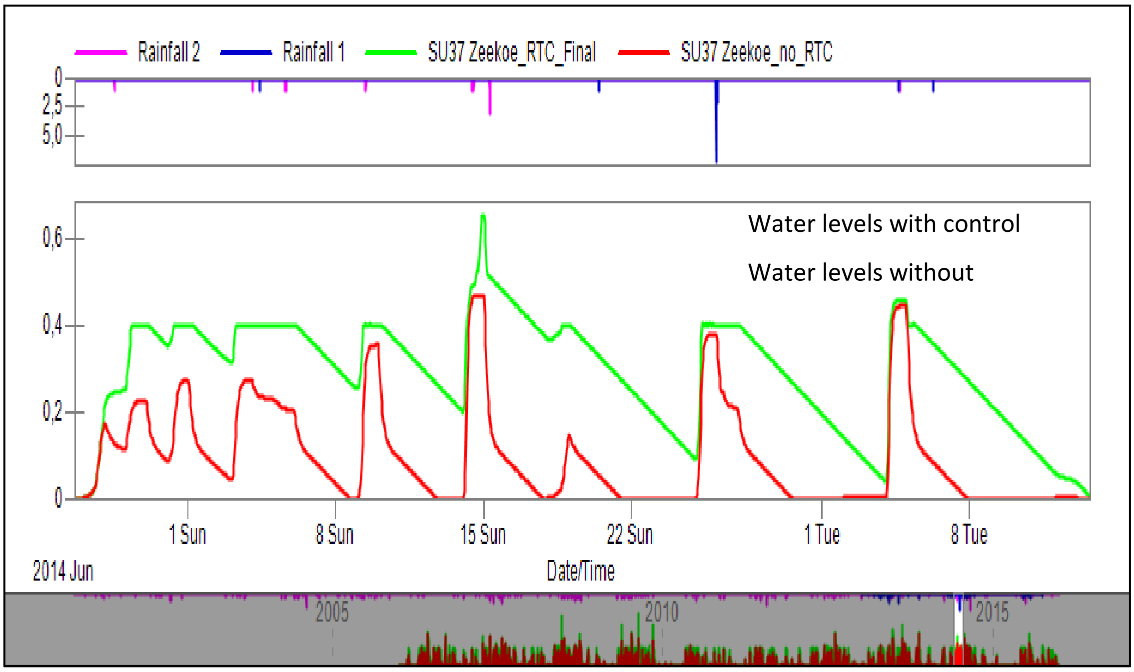

Figure 4-15: Water-level variation in the selected stormwater pond (maximum depth of 1 m)

4.4 MODELLING THE STORMWATER HARVESTING PROCESS

4.4.1 Method for modelling the stormwater harvesting process

Two fundamental approaches are used to model SWH with storage units: yield after spillage and yield before spillage (YBS) (Mitchell et al., 2008). The mathematical models that describe the operating rules are presented in Equation 4-10 and Equation 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; 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 and Modica, 2014). The YAS approach is the most widely used method and provides more conservative results compared with YBS (Campisano and Modica, 2014; Palla et al., 2011). In the selection of an appropriate time-step to model SWH, studies (e.g. Campisano and 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 the 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-minute 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-minute 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 and Schmida, 2000, Konig, 2001, Liaw and Tsai, 2004, Mitchell et al., 2007, Neumann et al., 2011, Roebuck, 2007, Seo et al., 2015, Yuan et al., 2003) 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 periods (1996–2015). The mean monthly modelled flow volumes over the periods are presented in Figure 4-16:.

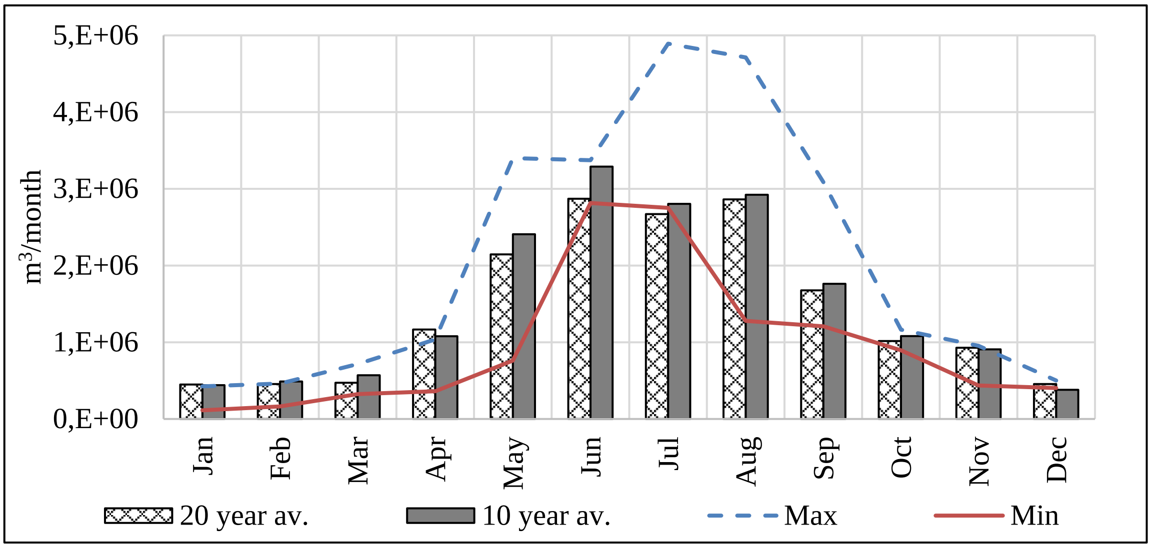

Figure 4-16: Modelled mean annual flow volume and limits in the study area

The mean annual modelled flow volumes over the ten-year (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).

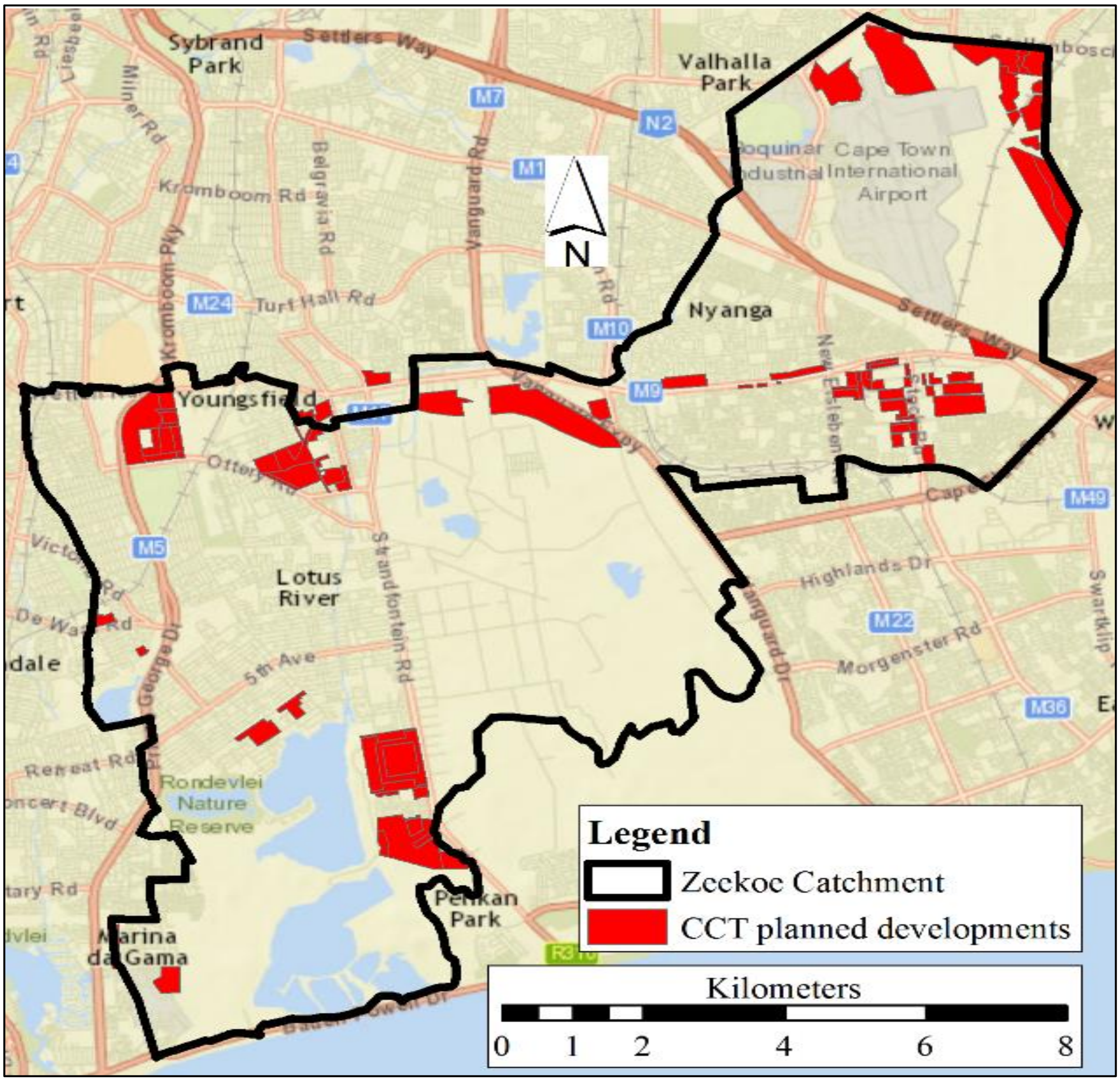

Figure 4-17: CCT planned developments in the study area (CCT, 2012)

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 SuDS. The level of imperviousness as a result of the land-use change without the 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 reuse.

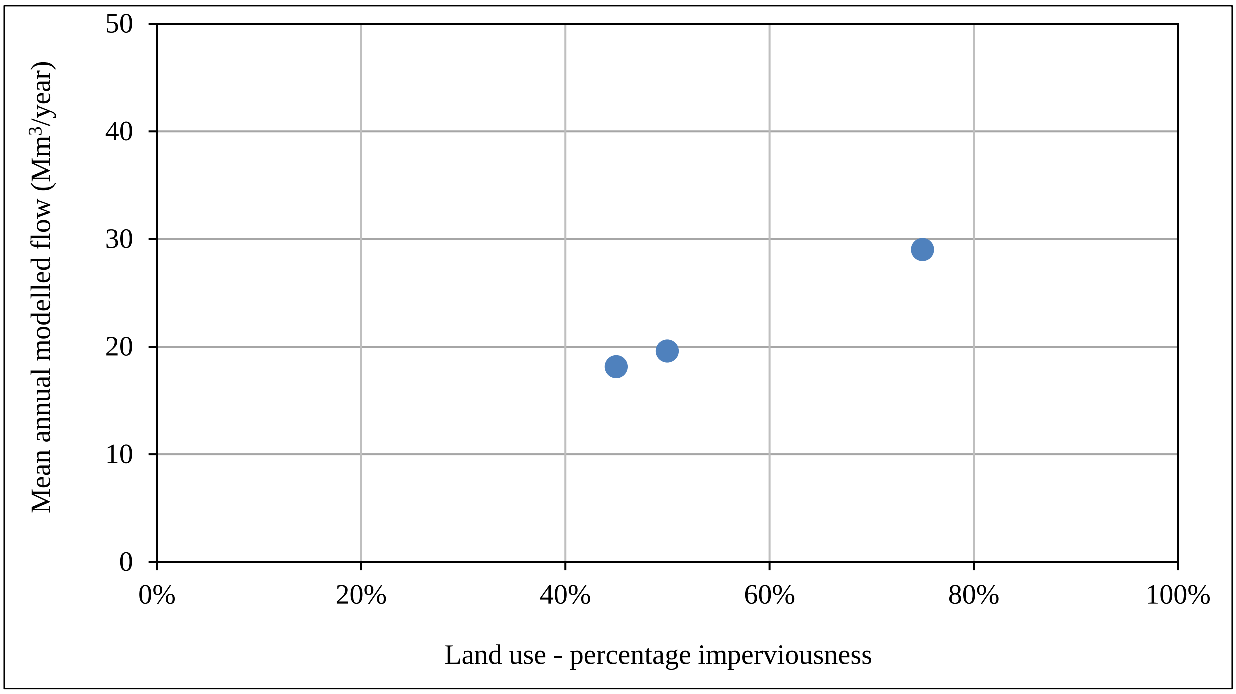

Figure 4-18: Plot of runoff increase as a function of land use

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's CSAG, as discussed in Section 0 and Section 0. 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 showed 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.

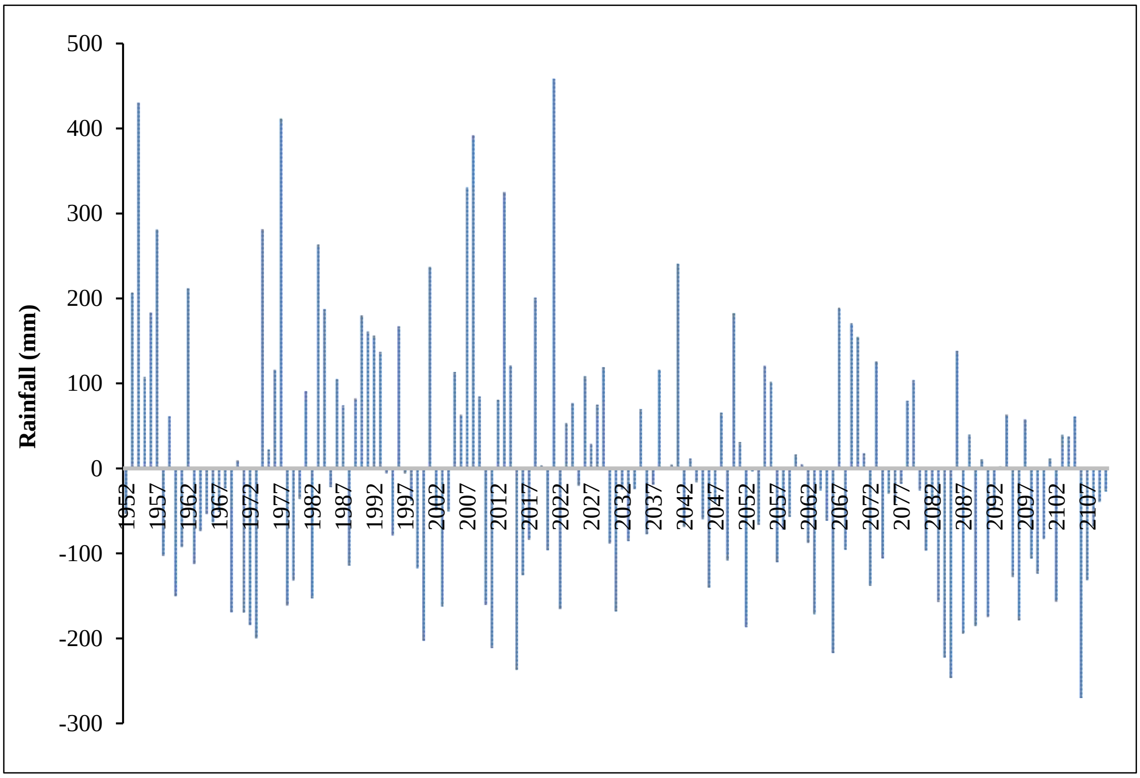

Figure 4-19: Annual rainfall variation from the mean of the modelled period (2006–2015) (after Hewitson and Crane, 2006)

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 that of wet years will 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 of 2090–2100 was estimated and compared with the historical period of 2006–2015 as the base case. The volumetric change is shown in Figure 4-20: . The climate predictions show a mean decrease of 30% in potential harvestable surface water resource with some models showing an over 50% reduction (Figure 4-20:).

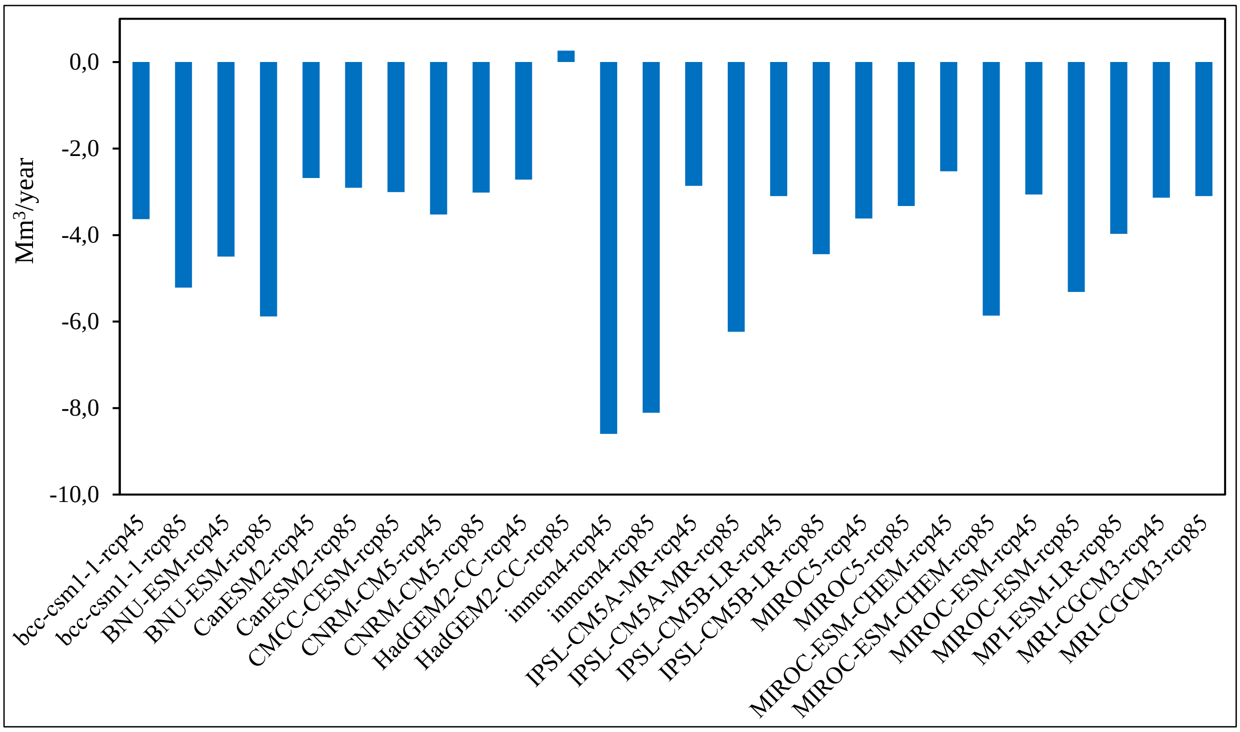

Figure 4-20: Change in potential future surface water resource from the study catchment

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:.

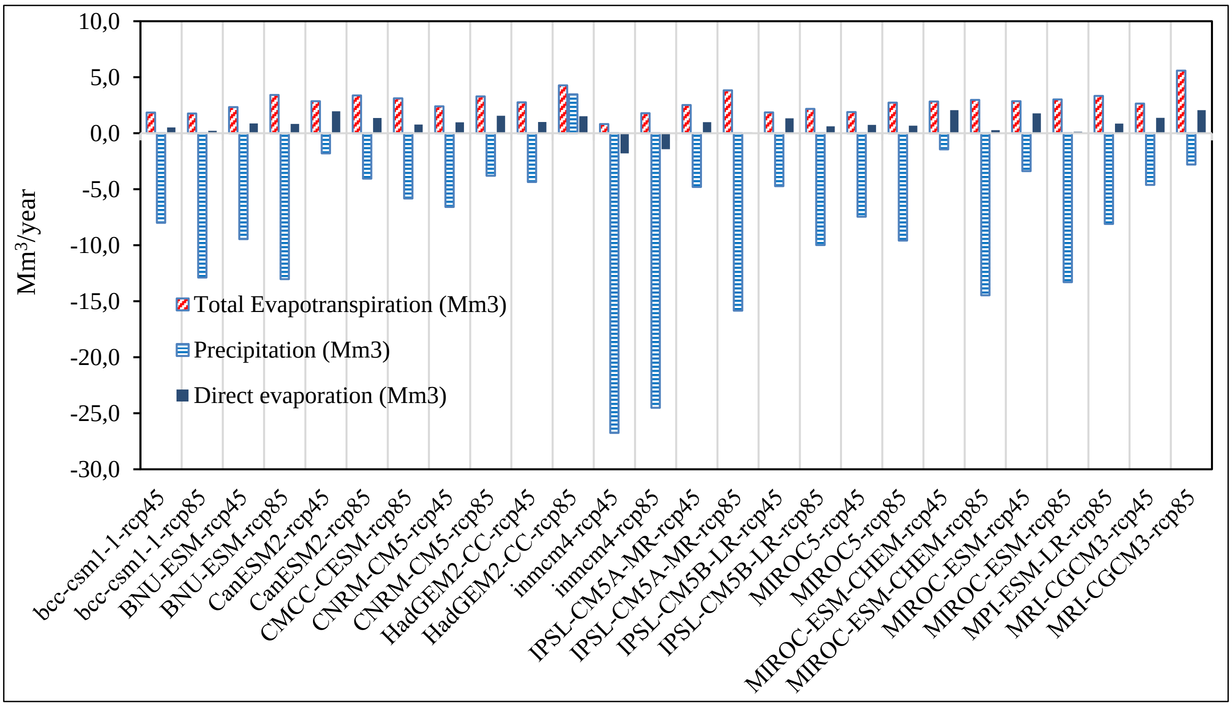

Figure 4-21: Change in rainfall and evapotranspiration in the study catchment

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's water demand in 2018). The mean annual modelled volumes for the wettest and driest years are 25 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 a significant increase in the potential harvestable water resource as surface runoff – in turn, requiring additional storage to enable its capture for reuse. The 26 climate-change prediction models also show a likely decrease in harvestable stormwater volume of 3 to 9 mm^3 (15 to 50% of the mean annual modelled volumes).

CHAPTER 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 0 discusses the modelling of surface-to-groundwater transfer i.e. managed aquifer recharge and the estimation of recharge volumes based on the PCSWMM model (the hydrological model discussed in Chapter 4). Section 0 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 0 and 0 present the set-up 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 top level of the surface water in the node above the bottom of the aquifer (m).

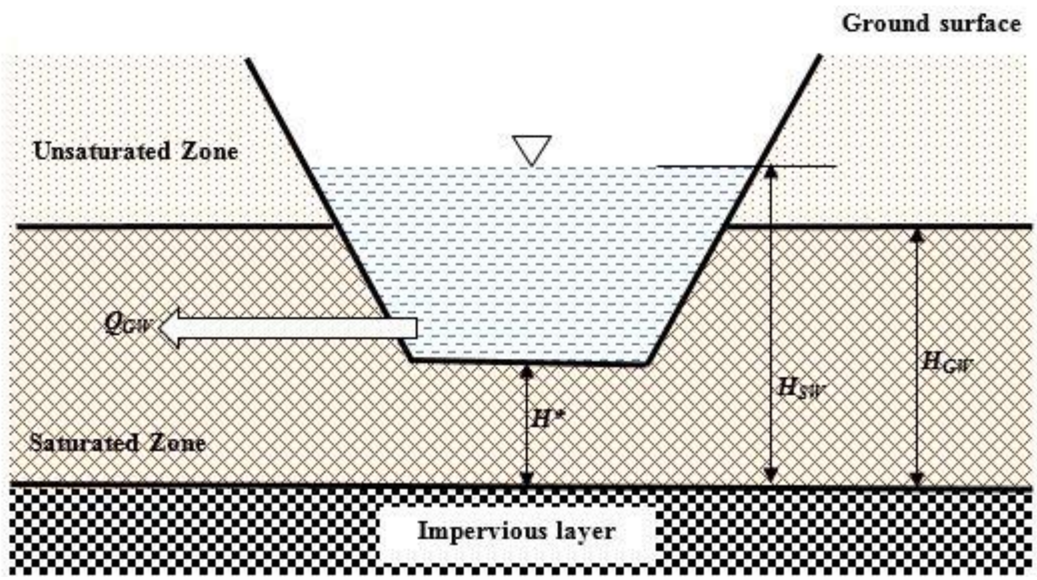

Figure 5-1: The two-zone groundwater layers in SWMM (James et al., 2010)

where Q_{GW} = Groundwater flow (cm/ha); H_{GW} = Height of saturated zone above aquifer bottom (m); H_{SW} = Height of top level of the surface water in the node above the bottom of the aquifer (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 the 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 modelling of stormwater ponds that is 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 to 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 modelling of surface-to-ground transfer with the modification of the existing stormwater ponds to function as infiltration cells was implemented using elements borrowed from bio-retention cells. The use of bio-retention cells has been a subject of various studies, such as those of Clary et al. (2008), Hathaway et al*.* (2008), Hunt et al. (2008), Trowsdale and 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 (that 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 of the filter media layer from time to time 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 those of 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).

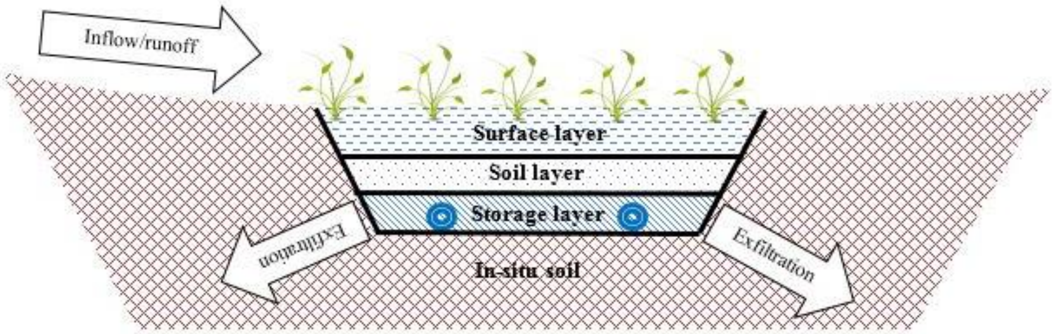

Figure 5-2: A bio-retention cell (after Brown et al., 2011)

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

i. **Surface layer:** This is 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 shape of the stormwater pond) and vegetation cover (100% of the pond's surface area). The modelling of the water balance in the surface layer was determined from Equation 5-1.

$$\Phi_1 \frac{\partial d_1}{\partial t} = i + q_o - e_1 - f_1 - q_1$$ **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/hour).

ii. **Soil layer:** This is the middle section of the bio-retention cell, which 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 to 900 mm (James et al., 2010). The soil properties in the study area, i.e. porosity (0.30 to 0.44), field capacity (15.6 to 17.6%), wilting point (4.4 to 5.2), conductivity (4.8 to 19.9 cm/hour), conductivity slope (9.7 to 9.9 cm/cm) and suction head (5.9 to 114.5 cm), were determined by Mavundla (2018) and are suitable as filter media. The modelling of the water balance in the soil layer was determined from Equation 5-2.

$$D_2 \frac{\partial \theta_2}{\partial t} = f_1 - e_2 - f_2$$ **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/hour); f_2 = Percolation rate of water through the soil layer into the storage layer (mm/hour).

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

$$\Phi_3 \frac{\partial d_3}{\partial t} = f_2 - e_3 - f_3 - q_3$$ **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/hour); e_3 = Storage layer evapotranspiration rate (mm/hour); f_3 = Exfiltration rate of water from the storage layer to native *in-situ* soil (mm/hour).

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

i. **Surface layer:** The plan area of the stormwater ponds that was adapted to function as a bio-retention cell in PCSWMM was constant for the entire depth. The inflow was uniformly distributed over the entire surface area. 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 those of Henzen (1973), Geber (1981) and Adelana et al. (2010). The surface elevation was from LIDAR DEM data from the CCT. The data was interpolated across the catchment to determine the mean aquifer depth and surface elevation at all the stormwater ponds in the study area, as shown in Figure 5-3.

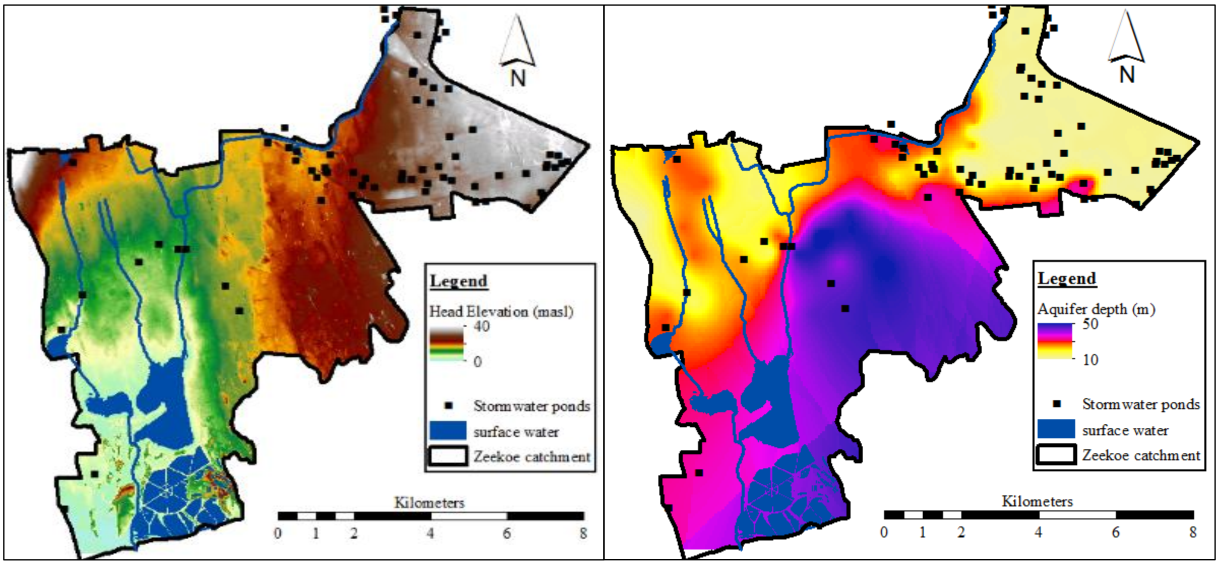

Figure 5-3: Aquifer depth and surface elevation (after Adelana et al., 2006; CCT, 2015)

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.

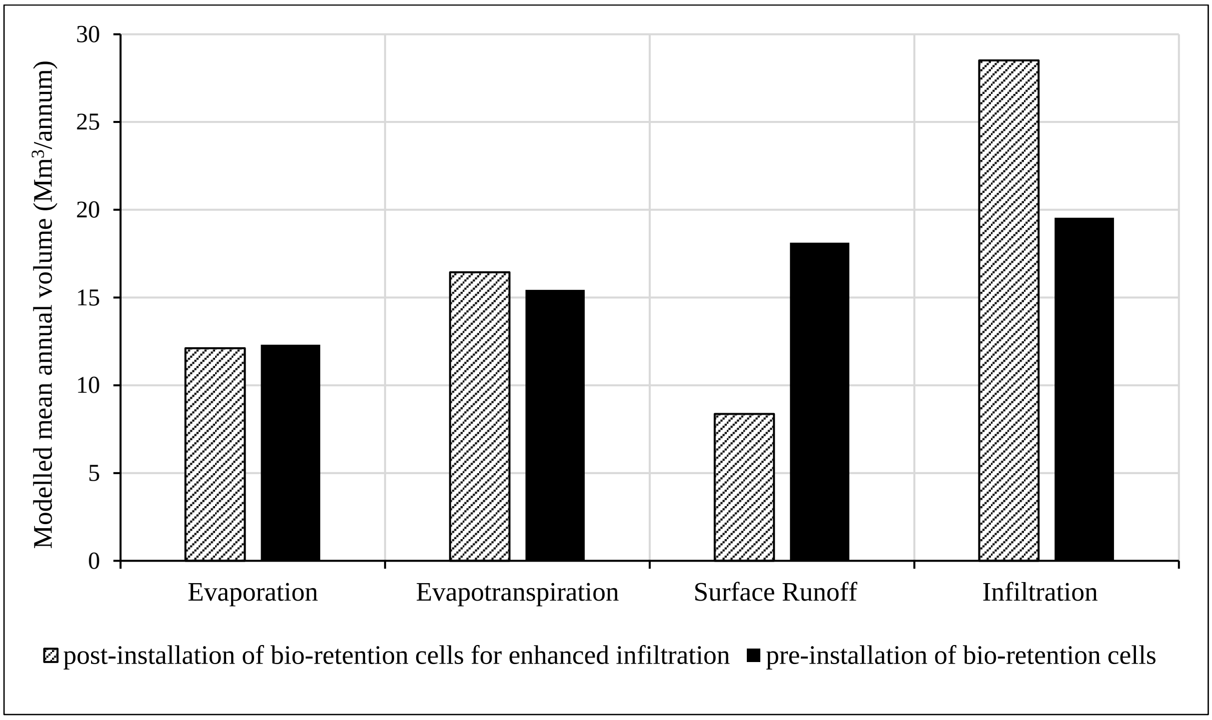

Figure 5-4: Comparison of existing and post-modification to bio-retention (45% imperviousness)

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 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 an 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. The results are given in Figure 5-5.

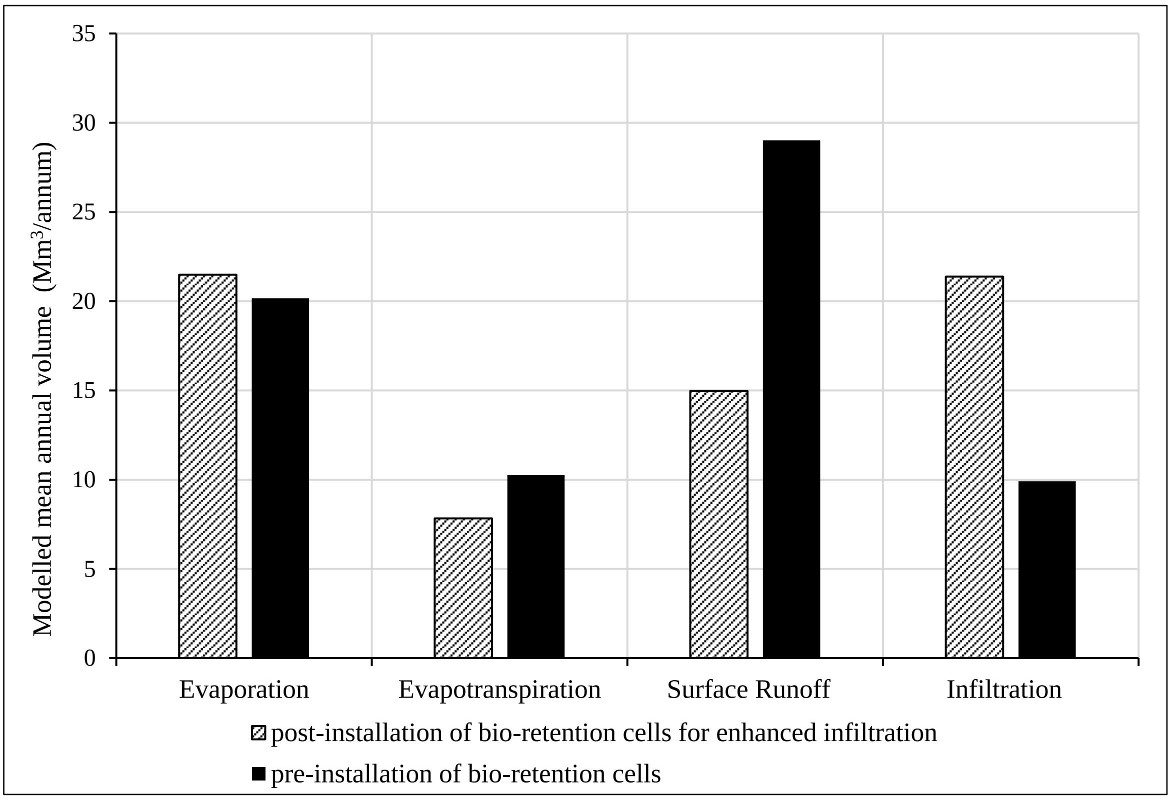

Figure 5-5: Comparison of existing and post-modification to bio-retention (75% imperviousness)

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 and Sayeed (2006) using data from previous research, including "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.

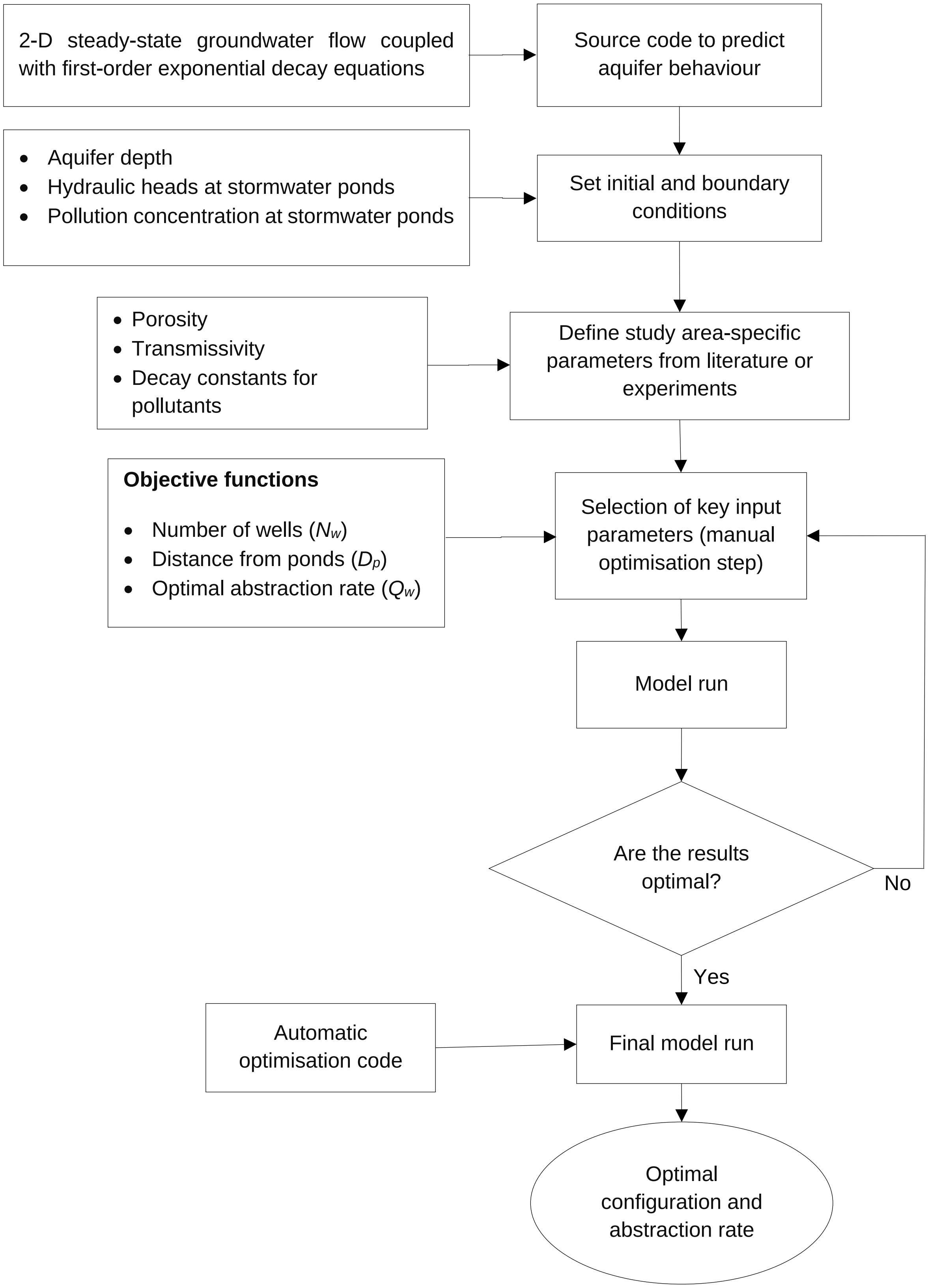

Figure 5-6: Groundwater abstraction model structure

These studies provided information on the geological conditions and aquifer depths in the study area. Other key information included soil type and the seasonal water table variation needed to define 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 0). The model development and implementation included discretising the aquifer into a matrix with the nodes needed to numerically solve the groundwater flow equations and determine 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 and Sayeed (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 and Sayeed (2006), the hybrid optimisation approach used to model groundwater flow and the 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).

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

where v_x and v_y = Velocity (flux) in x-direction and y-direction; $K(x, y)$ = Hydraulic conductivity in two 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:.

$$\frac{h_{i+1,j,t} - 2h_{i,j,t} + h_{i-1,j,t}}{(\Delta x)^2} + \frac{h_{i,j+1,t} - 2h_{i,j,t} + h_{i,j-1,t}}{(\Delta y)^2} = \frac{1}{K(x, y)} \frac{h_{i,j,t} - h_{i,j,t-1}}{\Delta t}$$ **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 the 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.

#2

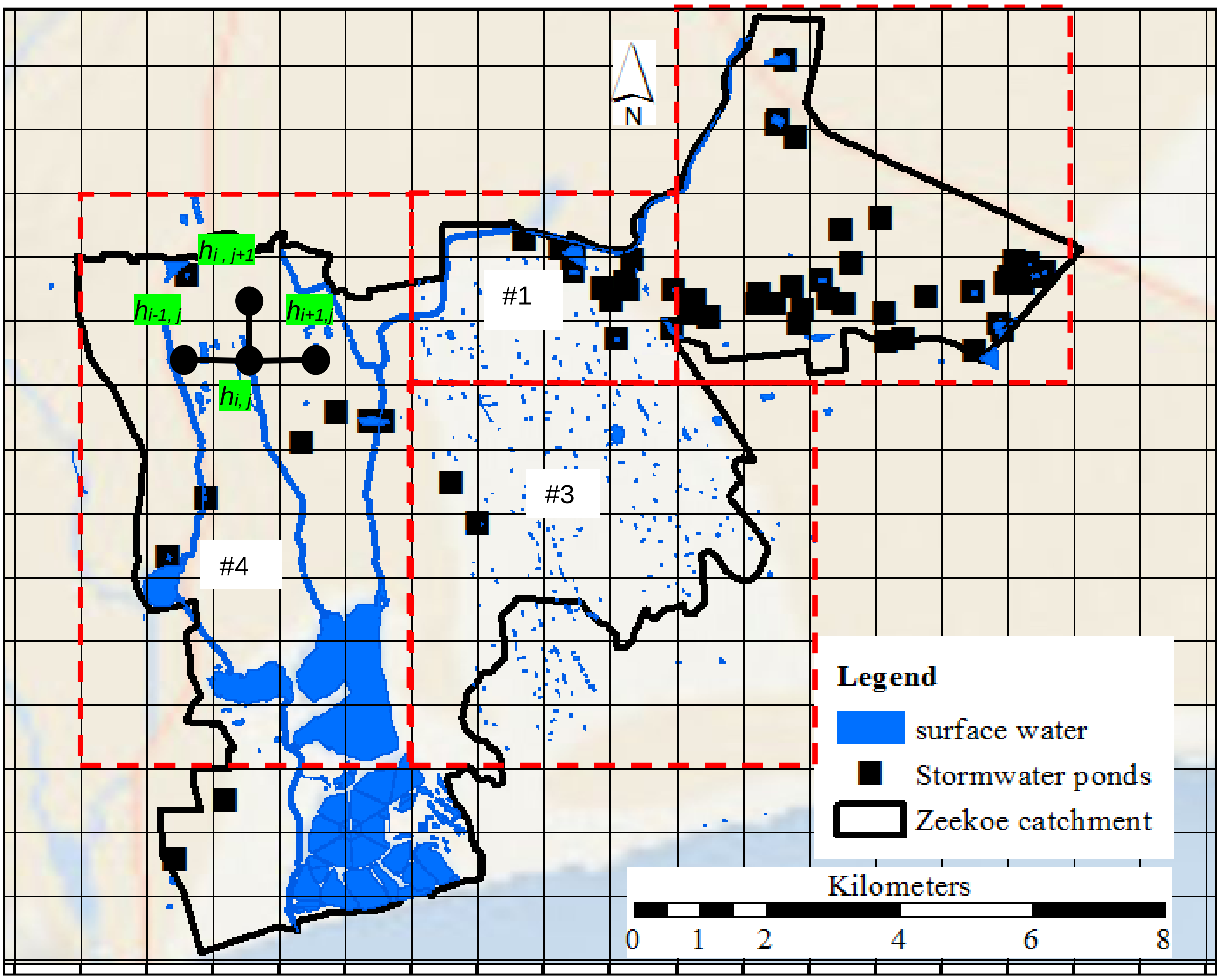

Figure 5-7: A discretised section of the Zeekoe catchment (after CCT, 2015)

5.2.3 Groundwater abstraction

Groundwater abstraction modelling aimed to determine the appropriate number of abstraction boreholes (N_w), the distance of the boreholes from the 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 0. 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 the groundwater aquifer, it was necessary to ensure that the abstracted water was from the stormwater ponds. Secondly, it was desirable that harvesting stormwater in this manner (i.e. recharge and recovery from the 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. There is a trade-off between maximising the quantity of the harvested stormwater (the assumption is that harvesting water from regions closest to the ponds is likely to be from stormwater ponds) and the desirable water quality improvement in terms of 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 cause 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). These seasonal groundwater-level fluctuations provided the basis for setting initial conditions and hydraulic heads in the model.
iii. The groundwater abstraction rates were set so 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.
v. The retention time of the water in the aquifer was kept at around one year to ensure die-off of the bulk of potentially pathogenic organisms as established by Doll (2017) in 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 the 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 an MSc student from ETH Zurich (the Swiss Federal Institute of Technology in Zurich). The selection of the 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 0. The boreholes were initially placed randomly, with the final positions determined during the modelling process based on the 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.

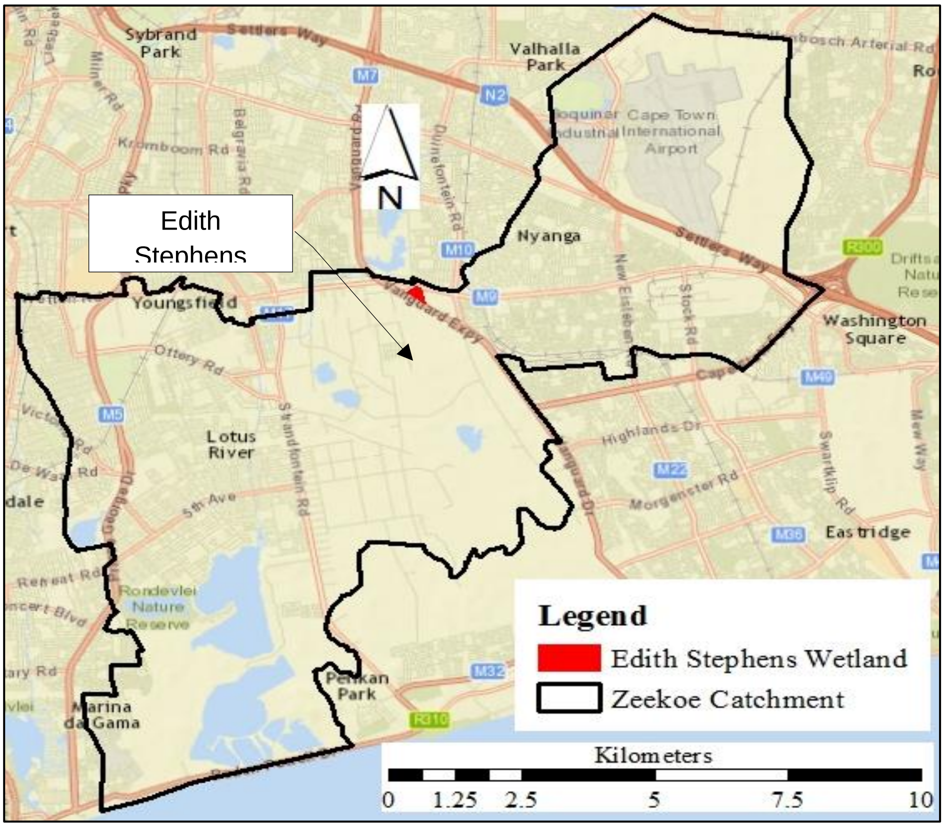

Figure 5-8: Trial section in the Edith Stephens Wetland (after CCT, 2015)

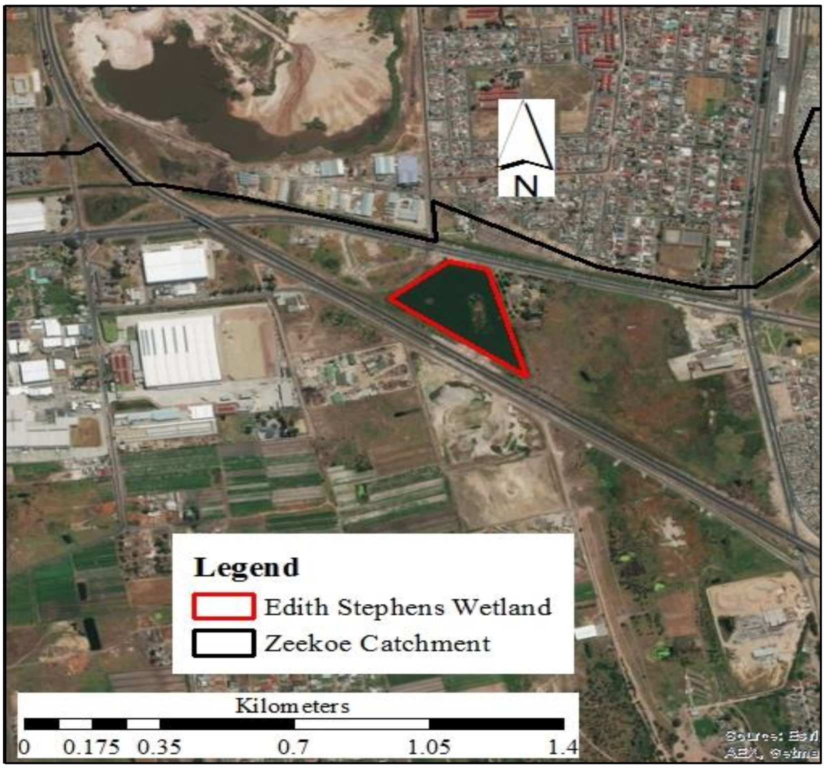

Figure 5-9: The Edith Stephens Wetland (after CCT, 2015)

In determining the ideal abstraction rate, the modelling of the rate of withdrawal was increased stepwise with a 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 the groundwater flow was increasingly originating elsewhere. The maximum abstraction rates were thus determined as 5.8 ℓ/s per borehole.

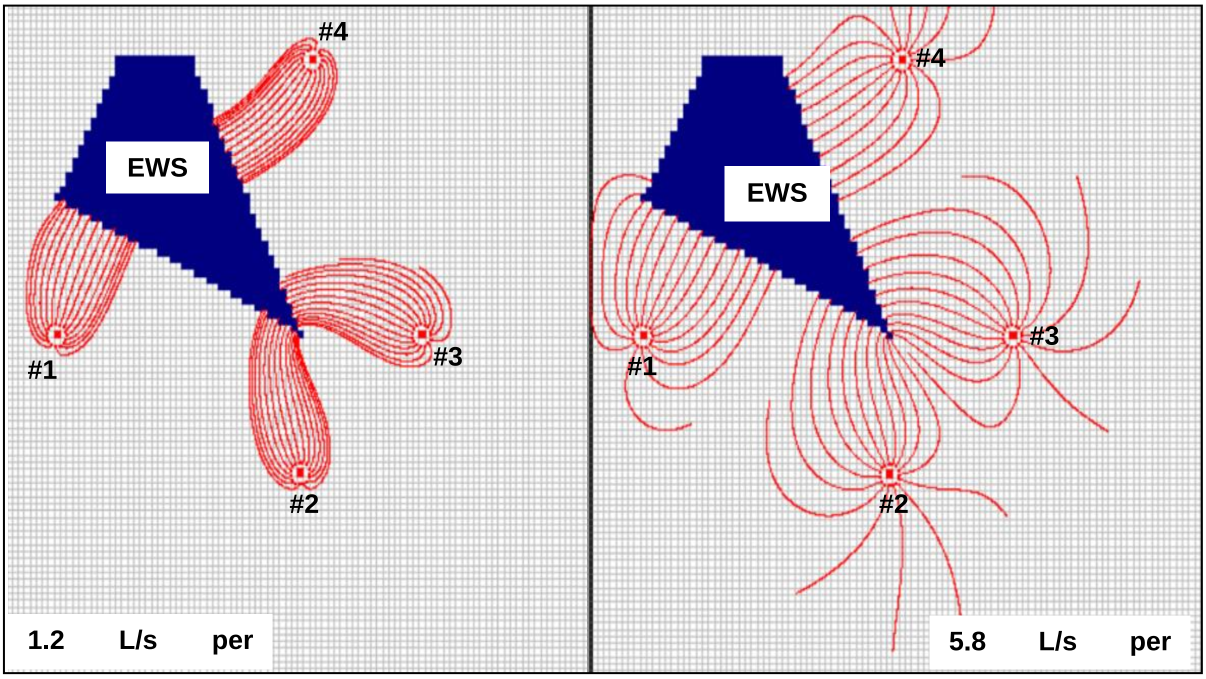

Figure 5-10: Impact of abstraction rates on the origin of the groundwater flow fields at the Edith Stephens Wetland (Doll, 2017)

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. Schulze et al., 2005, Zimmerman et al., 2016) 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 the availability of nutrients in the water and the exposure to UV radiation and temperature (Delleur, 2007). Nevertheless, simplification was adequate for the study as the goal was to provide an 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 the transport and decay of *E. coli* as an indicator organism using an abstraction rate of 5.8 ℓ/s per borehole and a distance of 400 m. The values used in the simulation of pollution transport were based on the monthly grab samples collected by the CCT from various locations in the Edith Stephens Wetland. The data collected at the inlet and outlet from 2006 to 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 the 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 a conservative value of 3,400,000 CFU/100 ml (the maximum value). The results of the modelling of water quality improvement are presented in Figure 5-12.

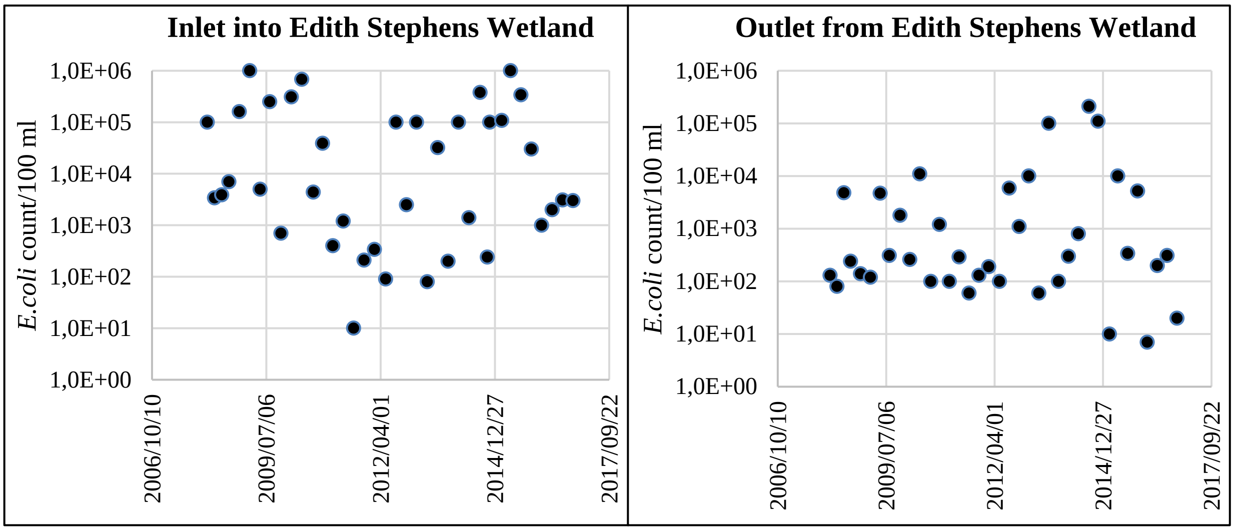

Figure 5-11: *E. coli* measured at the Edith Stephens Wetland (CCT, 2017)

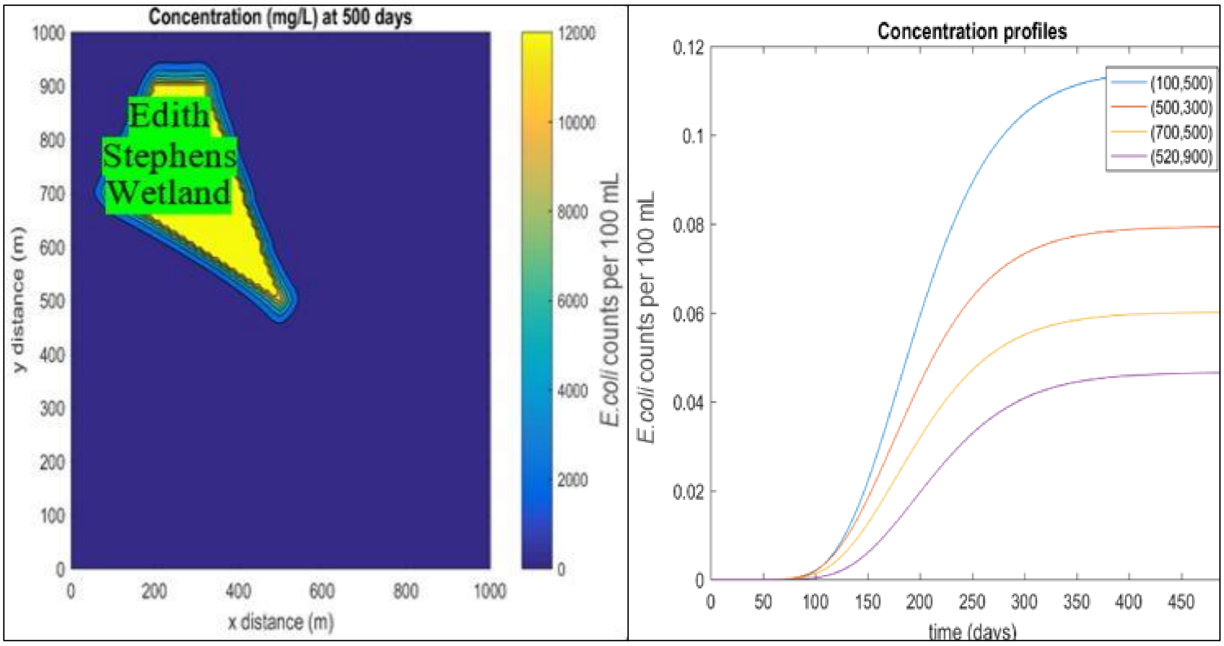

Figure 5-12: *E. coli* counts in the Edith Stephen Wetland (left) and at the boreholes abstracting at 5.8 ℓ/s (right) with pumping commencing at Day 0 (Doll, 2017)

The results from the trial section with a single pond (the Edith Stephens Wetland) and *E. coli* as an indicator organism for faecal pollution show 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 mℓ (Figure 5-11) to values below one count per 100 mℓ (Figure 5-11). With the South African National Drinking Water Standards (SANS 241:2015) requiring a zero *E. coli* count, 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 MODEL

5.4.1 Groundwater abstraction model

The findings from the trial section model (Section 0) were extended to a catchment-scale model. Additional information on typical borehole yield rates for the study area were obtained from the CCT (Figure 5-13) and various studies on the Cape Flats aquifer, such as those of Vandoolaeghe (1989), Fraser et al. (2001), DWA (2008) and Mauck (2017).

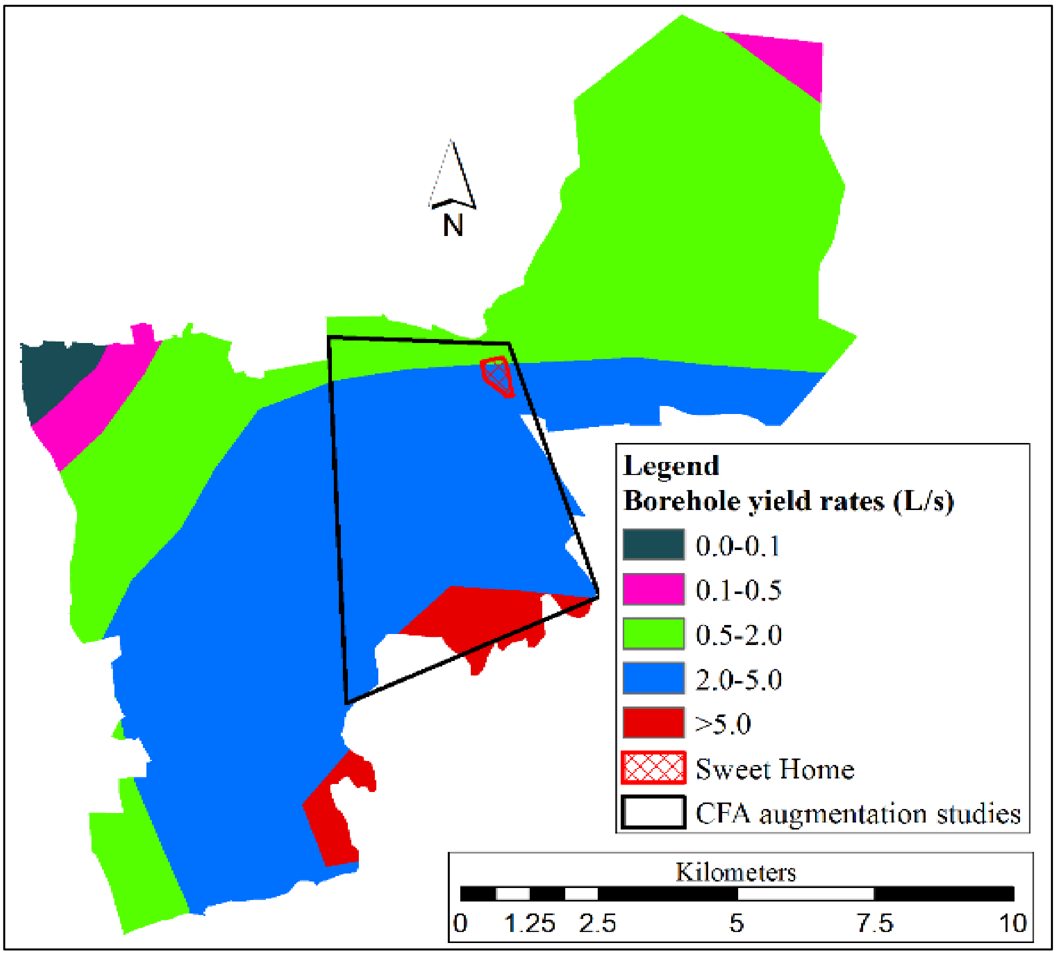

Figure 5-13: Borehole yield rates in the study area (after CCT, 2005)

Vandoolaeghe (1989) determined that a total of 10 mm^3 per 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 (Fraser et al., 2001) suggested a total groundwater yield of 18 mm^3 per 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, comprising 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 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 0), the CCT and various references (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 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 dashed 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 Figure 5-15, respectively.

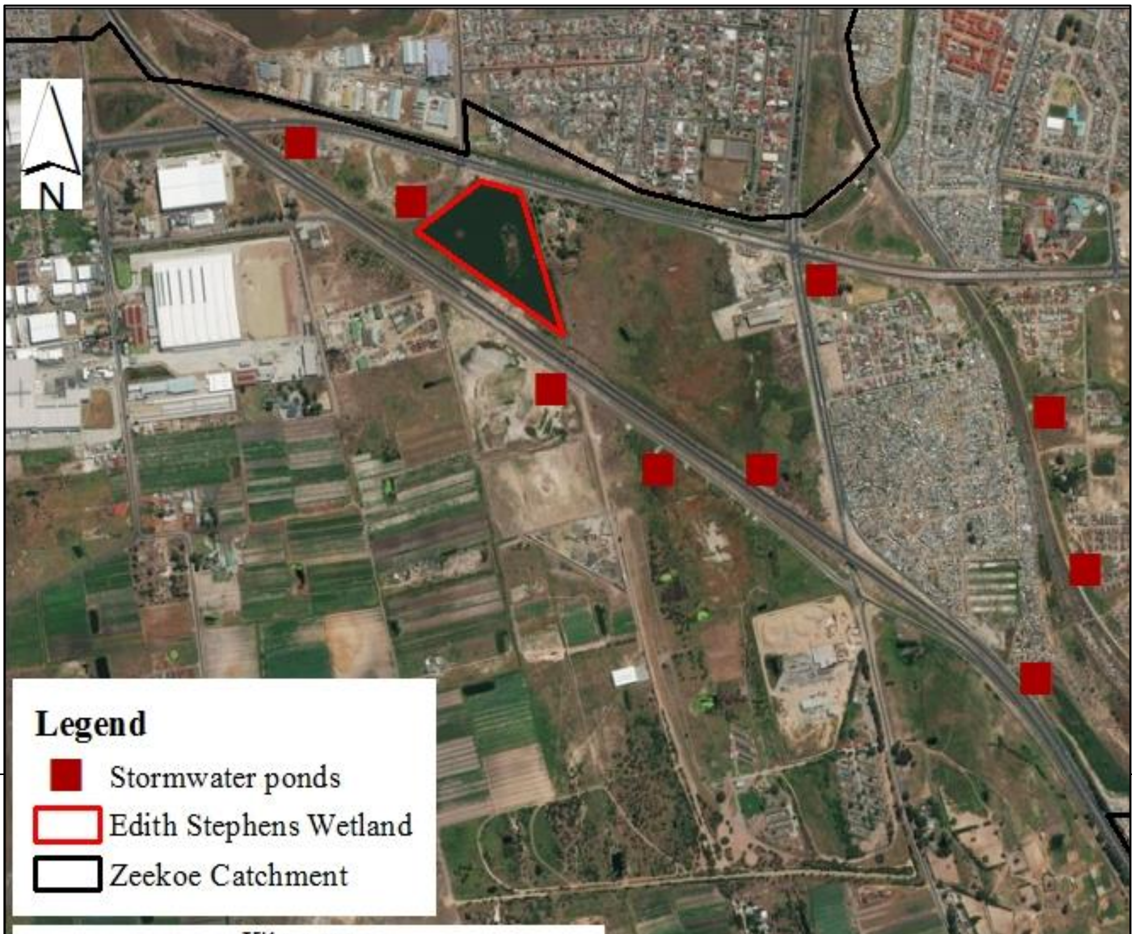

Figure 5-14: Location of the stormwater ponds in area #1

The modelling of the groundwater abstraction in this study was implemented in two main steps: a manual trial and an optimisation process. The manual trial consisted of initially placing four boreholes per pond in each of computational areas #1, #2, #3 and #4 (Figure 5-7). The boreholes were initially placed randomly around the stormwater ponds and each was simulated with an abstraction rate of 5.8 ℓ/s as determined in the trial section (Section 0). The number of boreholes and abstraction rates were then adjusted as discussed in Section 0 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 and Sayeed (2006) and discussed in Section 0 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.

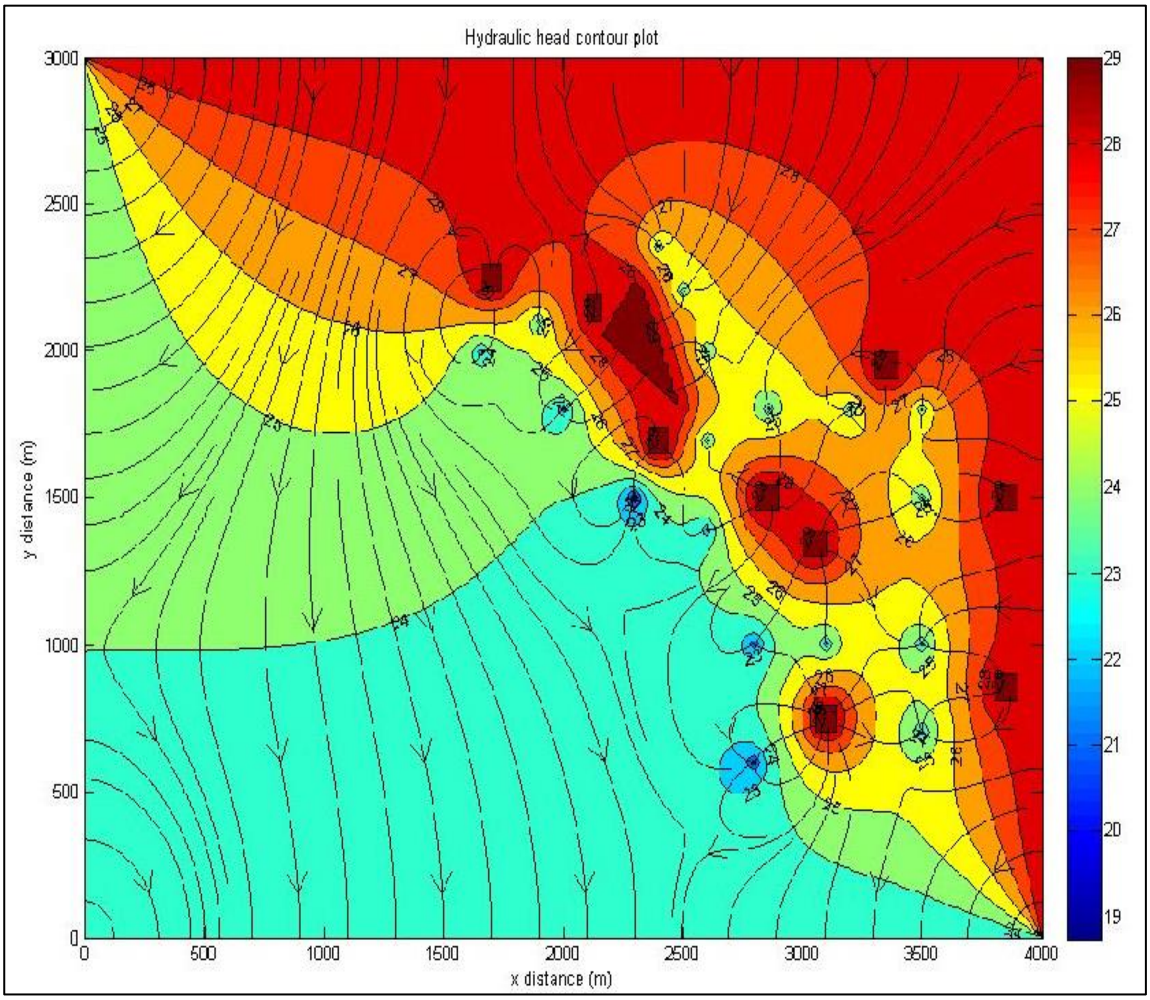

Figure 5-15: Modelled phreatic flow fields from the stormwater ponds to the boreholes

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/hour)	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 the 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 to 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 to draw 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 *E. coli*, total suspended solids, pH level, electrical conductivity, nutrients (i.e. total persulphate oxidisable nitrogen, nitrate and nitrite, and 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 1,000 counts per 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 the CCT at locations shown in Figure 5-16 with transport and decay Equation 5-6.

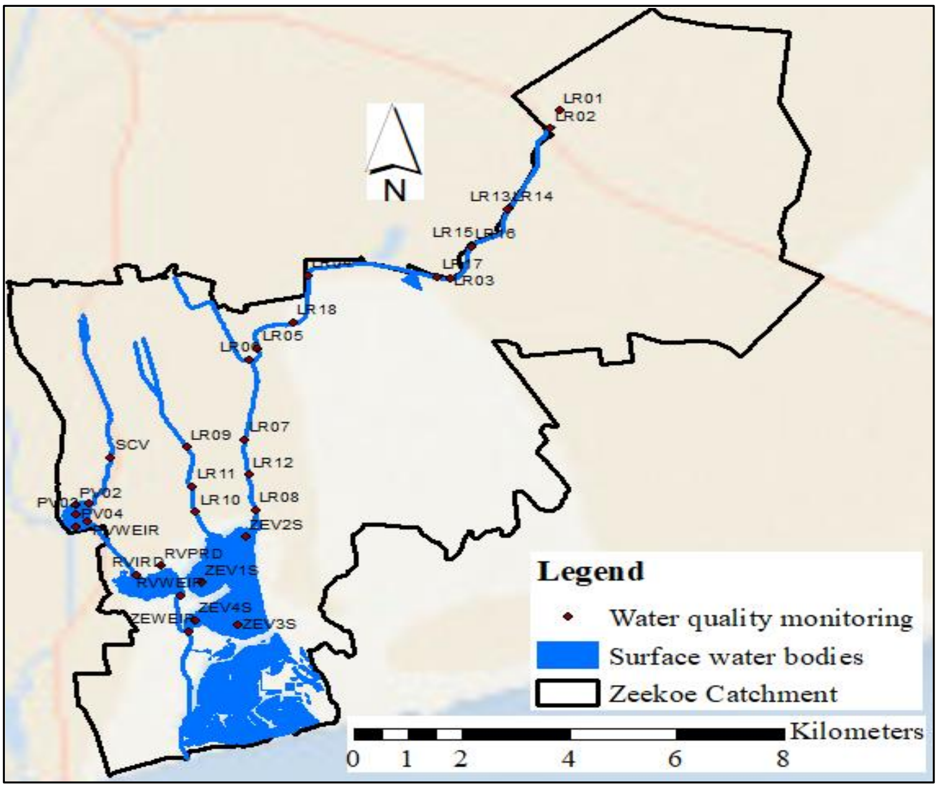

Figure 5-16: Locations of water quality motoring in the study area (after CCT, 2017)

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 the Edith Stephens Wetland (Figure 5-11) and Equation 5-6. The modelling was based on the procedure discussed in Section 0. The results from water quality modelling based on the final positions of the 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 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 eight counts per 100 mℓ.

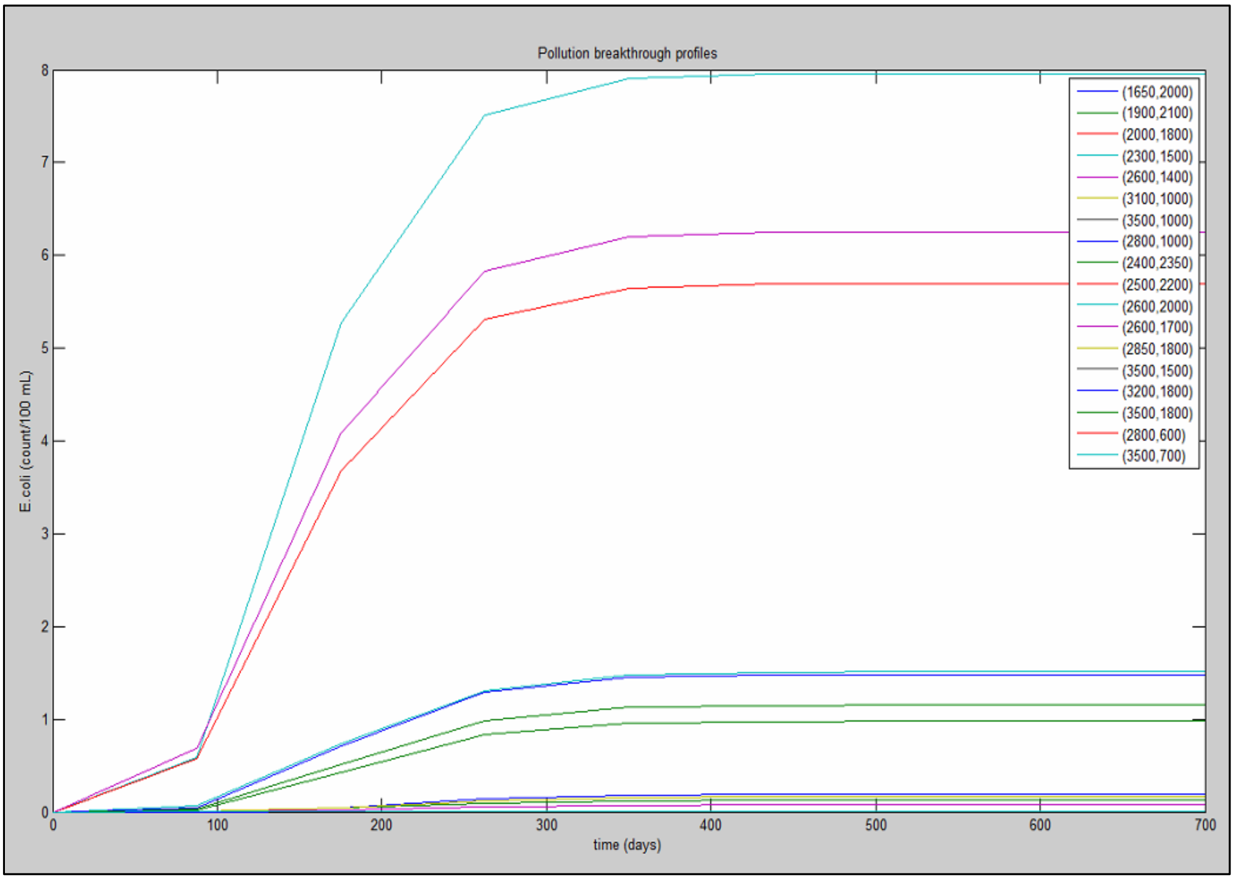

Figure 5-17: Estimated *E. coli* counts in the abstraction boreholes following the commencement of pumping

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 mℓ 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 the rate of accumulation increased rapidly over a 400-day period, and stabilised at about 600 days (the 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.

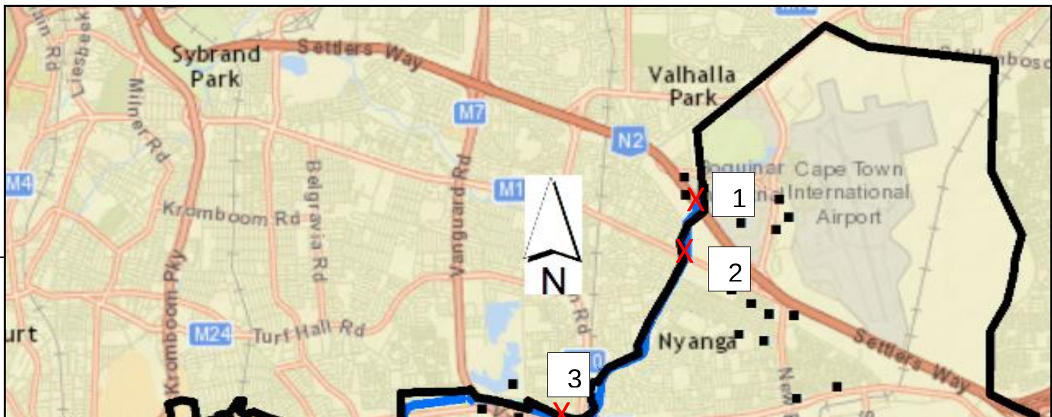

Figure 5-18: Locations of heavy metal sampling

The data on heavy metals collected in the study included arsenic, cadmium, chromium, lead and mercury. These have been linked to chronic diseases such as cancer (USEPA, 2016; WHO, 2008). A total of 35 samples per heavy metal were collected over a one-week period from 20 to 24 June 2016, as shown in Table 5-3:.

Table 5-3: Heavy metals 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 arsenic)	20 June	<3	<3	<3	<3	<3	<3
	21 June	<3	<3	-	<3	<3	22
	22 June	<3	8	<3	<3	<3	<3
	23 June	<3	<3	<3	<3	<3	<3
	24 June	<3	<3	<3	16	7	13
Cadmium (µg/ℓ as cadmium)	20 June	<1	<1	<1	<1	<1	<1
	21 June	<1	<1	-	<1	<1	<1
	22 June	<1	<1	<1	<1	<1	<1
	23 June	<1	<1	<1	<1	<1	<1
	24 June	<1	<1	<1	<1	<1	<1

Heavy metals	Sample date	Locations where samples were collected for testing heavy metal concentration					
		1	2	3	4	5	6
Chromium (µg/ℓ as chromium)	20 June	<7	9	10	<7	<7	<7
	21 June	<7	20	-	<7	13	12
	22 June	<7	<7	<7	<7	<7	<7
	23 June	<7	<7	<7	<7	<7	<7
	24 June	<7	<7	<7	<7	<7	<7
Lead (µg/ℓ as lead)	20 June	<7	<7	<7	<7	<7	<7
	21 June	<7	<7	-	<7	<7	<7
	22 June	<7	<7	<7	<7	<7	<7
	23 June	<7	<7	<7	<7	<7	<7
	24 June	<7	<7	<7	<7	<7	<7
Mercury (µg/ℓ as mercury)	20 June	<5	<5	<5	<5	<5	<5
	21 June	<5	<5	-	<5	<5	12
	22 June	<5	<5	<5	<5	<5	22
	23 June	<5	18	<5	14	<5	<5
	24 June	<5	<5	<5	<5	<5	<5

< represents heavy metal concentrations below the 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 the 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, which has been used in other studies, such as Marsalek et al. (2001), Davis et al. (2003) and Weiss et al. (2008).

$$\frac{C_s}{t} = C_w \frac{A_r}{A_i} \frac{MAR}{d.\rho}$$ **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 = Ratio of runoff area catchment to infiltration area; MAR = Mean annual rainfall (mm); d = Thickness of the soil layer (mm); ρ = Soil bulk density (kg/m^3); t = Time (years)

In the determination of the accumulation of 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 analysing them in the laboratory. They were found to have a mean of 1,541 kg/m^3, with minimum and maximum values of 1,467 and 1,616 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 Table 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
A_r/A_i	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	2,519	55	2,629	8
Protection of ecosystem health (years)	1,873	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
A_r/A_i	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	3,353	46	3,501	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
A_r/A_i	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	2,476	54	2,585	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/year)	480	480	480	480	480
A_r/A_i	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 in the catchment area to the infiltration area

5.4.3 Summary of results for stormwater harvesting from groundwater storage

The 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 an unconfined aquifer, would support abstraction rates of 3.5 to 8.1 ℓ/s from 140 boreholes to provide a mean annual groundwater yield of 28.51 to 32.61 mm^3. With the South African National Drinking Water Standards (SANS 241:2015) providing a zero *E. coli* count 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 that of Lim et al. (2015), have also shown that microbial pollution in stormwater from groundwater storage was significantly reduced to levels where the water could be used directly for some indoor residential needs with a limited level of contact, e.g. washing machine use and toilet flushing. Another study, i.e. that of 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 pathogens 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.

CHAPTER 6: POTENTIAL DEMAND FOR STORMWATER

In this chapter, the method and results relating to the potential demand for stormwater, including non-potable water supply in the Zeekoe catchment and potable water supply, both locally and in the Greater Cape Town area, are provided in nine sections: Section 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 toilet flushing. 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. Section 6.7 and 6.8 cover the stormwater quality and proposed water treatment for potable and non-potable water, and Section 6.9 provides 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.

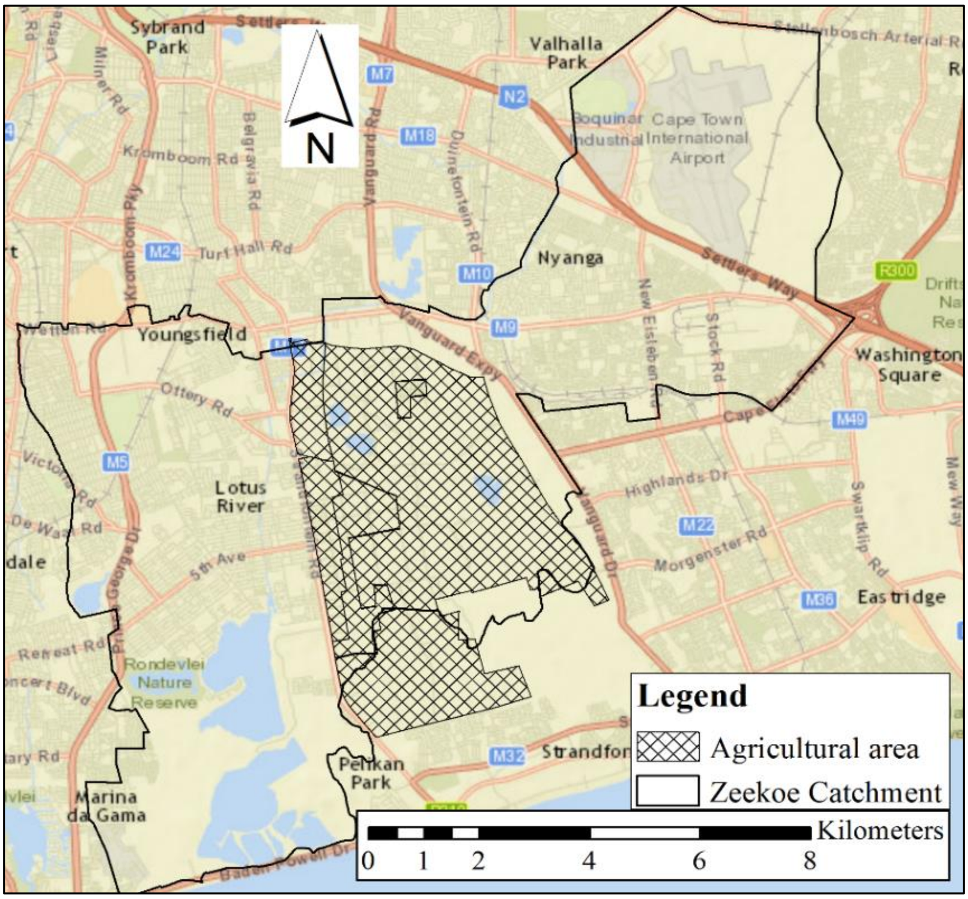

Figure 6-1: Urban agriculture in the study area (after CCT, 2016)

The agriculture irrigation water demand may be estimated from the crop water requirement needed to meet precipitation deficit (FAO, 2012). The FAO has developed the CROPWAT model to assist in the estimation of CWR based on climatic and soil data compiled by the FAO's 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; ET_o = 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 the 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 ET_o = 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) = Saturation vapour pressure deficit (kPa); Δ = Slope of the vapour pressure curve (kPa ^{o}C^{-1}); γ = Psychrometric constant (kPa ^{o}C^{-1})

Using Equation 6-2 with 24-hour 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). ET_o was calculated using data from the SAWS and New LocClim, the FAO's software for the estimation of various agroclimatic data based on the spatial interpolation of existing data in the FAO's database. The primary data for the study area required in the estimation of ET_o with Equation 6-2 and the ET_o values computed with CROPWAT are presented in Table 6-1.

Table 6-1: Data for the computation of evapotranspiration

	Tmax ^{o}C	Tmean ^{o}C	Tmin ^{o}C	RH max %	RH mean %	RH min %	Vapour pressure kPa	Wind speed m/sec	Sun-shine hours	Rad MJ/m^2.day	ET_o 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 ET_o values (with the modified FAO Penman-Monteith method) were compared with estimates from the Class A pan, Blaney-Criddle and Hargreaves methods discussed in Section 0. The results are as shown in Figure 6-2.

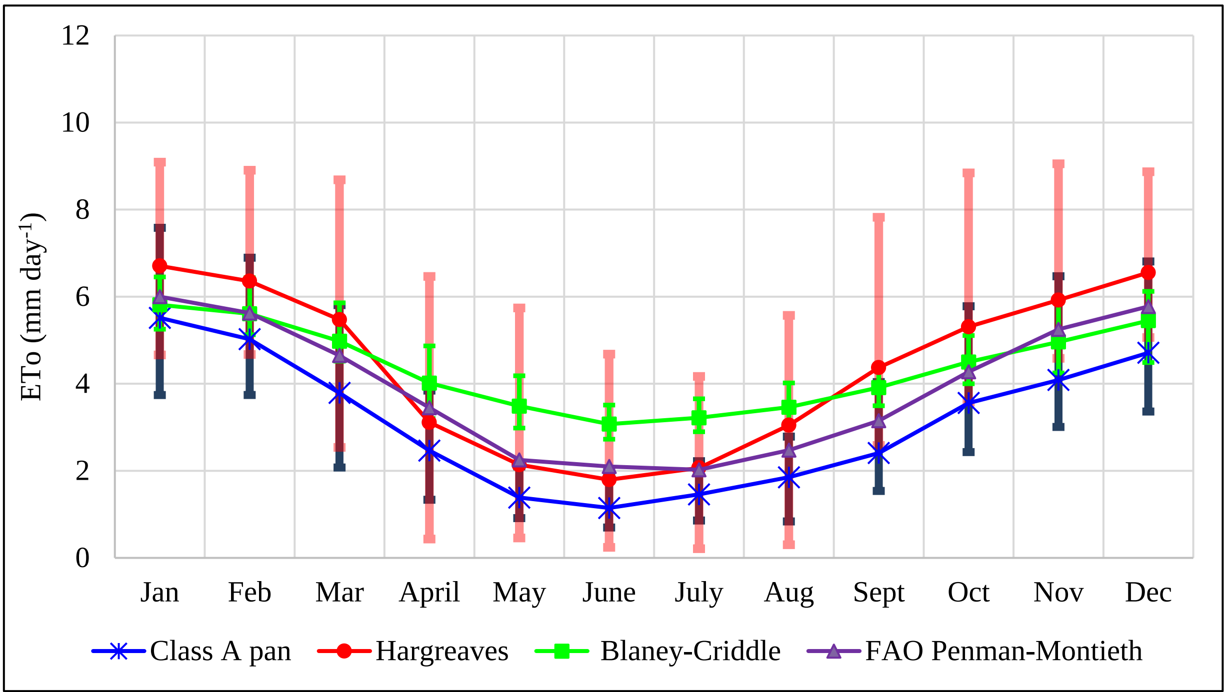

Figure 6-2: Estimated evapotranspiration values

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. The CWR was estimated using Equation 6-1 and was equal to the mean precipitation deficit from the historical rainfall data provided by the 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**
Potato	0	78	97.2	17.2	0	0	0	0	0	0	0	0
Cabbage	0	112.3	66.3	11.9	6.1	0	0	0	0	0	0	0
Small vegetables	62.3	142.3	59.3	0	0	0	0	0	0	0	0	0
Green beans	0	81.8	85.5	2.8	0	0	0	0	0	0	0	0
Tomato	0	94.7	92.6	22.8	4.1	0	0	0	0	0	0	0
Net scheme irrigation requirement												
Irrigation requirement 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**
Potato	0	0	0	0	0	0	0	18.2	48.1	108.5	158.3	41.8
Cabbage	70.1	0	0	0	0	0	0	31.9	23.1	75.5	155.7	217.5
Small vegetables	0	0	0	0	0	0	0	47.9	58.6	39.4	0	0
Green beans	0	0	0	0	0	0	0	20.6	48.6	84.8	0	0
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 per month of rainfall deficit, with the values for the wettest year (2013) and the driest year (2015), respectively, providing the minimum and maximum limits represented with range bars.

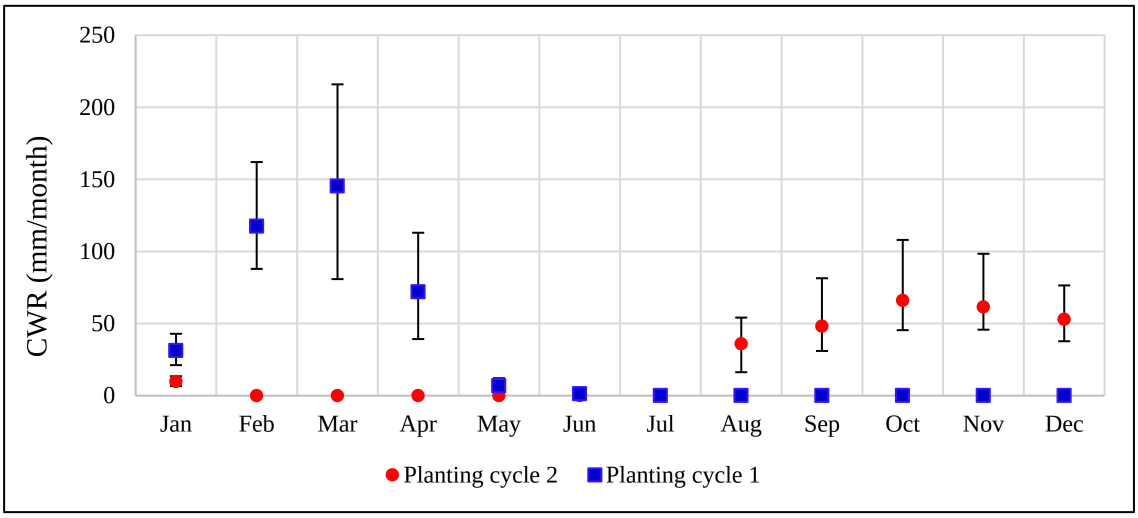

Figure 6-3: Mean monthly CWR estimates for the modelled period (2006–2015)

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 the driest year (2015), respectively, providing the minimum and maximum limits represented with range bars.

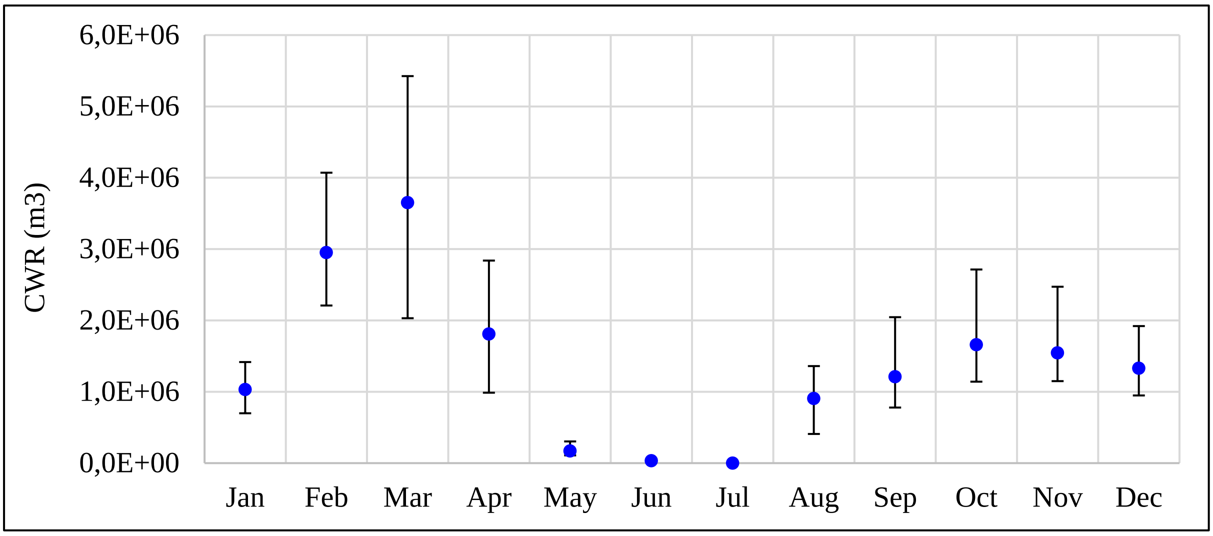

Figure 6-4: Aggregated mean monthly CWR estimates for the period (2006–2015)

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 the 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 the irrigation requirements needed to meet the 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 the driest year (2095).

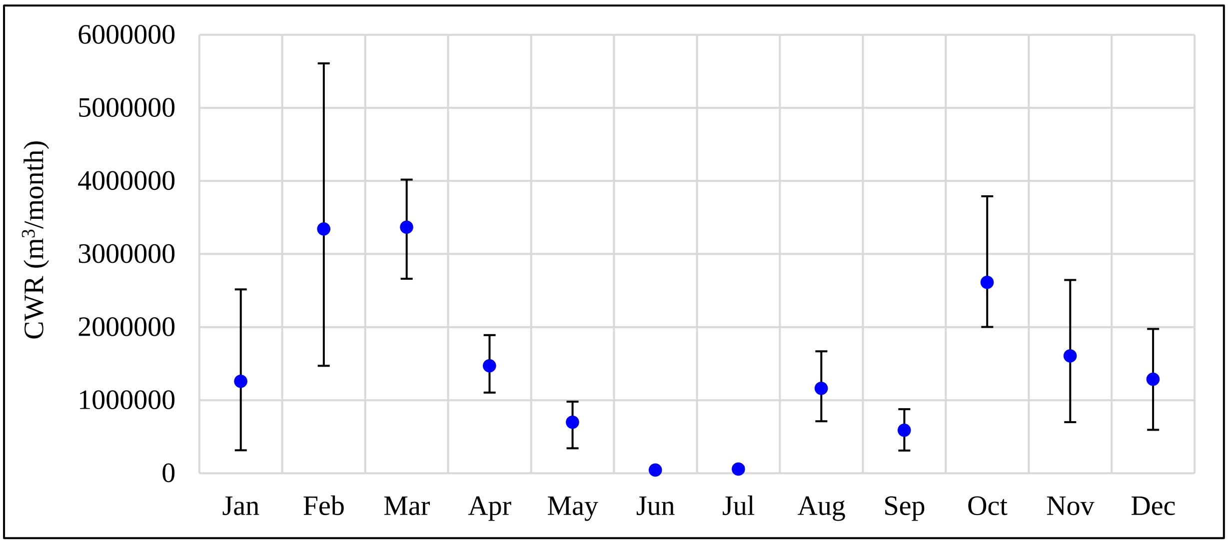

Figure 6-5: Aggregated mean monthly CWR

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 the 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 the typical annual mean agriculture water allocation to farmlands in the same regions as the study area, i.e. the 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 4,000 to 6,000 m^3 ha^{-1} yr^{-1} (Nieuwoudt et al*.,* 2008). Based on the CROPWAT estimations, the annual mean CWR value for agriculture in the study area, covering a total of 2,560 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 4,400 and 10,300 m^3 ha^{-1} yr^{-1}, respectively, because of climate change. The minimum and maximum CWR values based on 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.

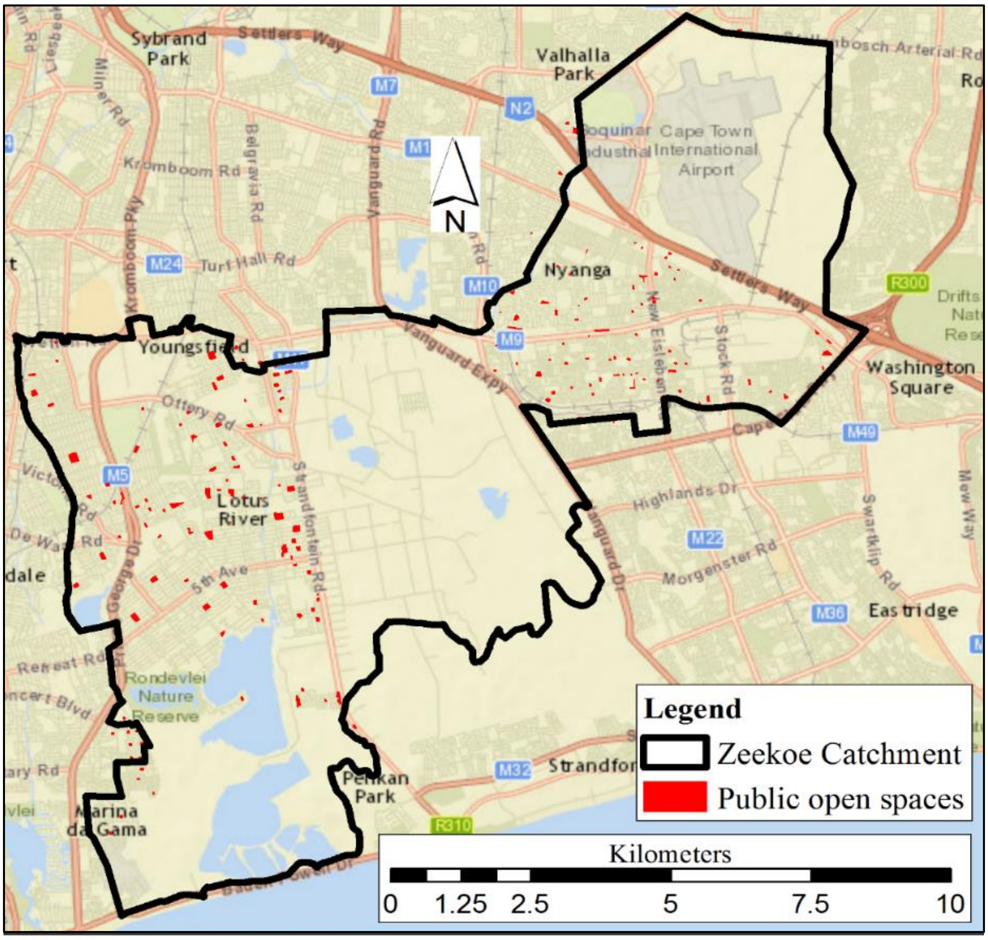

Figure 6-6: Various public open spaces in the study area (CCT, 2012)

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 the driest year (2015).

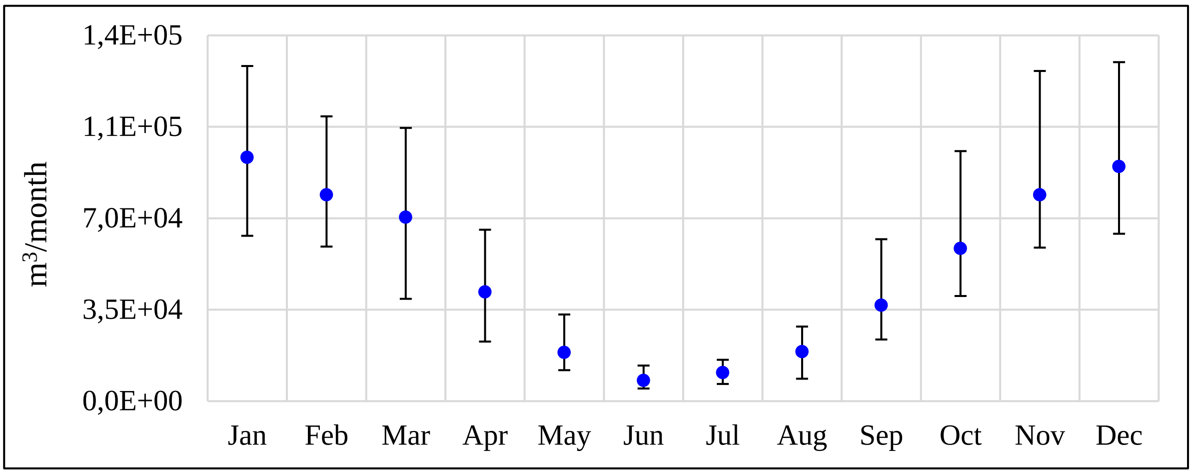

Figure 6-7: Public open spaces' monthly irrigation requirement

The mean annual CWR for the 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 the driest year (2015) were 0.93 mm^3 and 0.41 mm^3, respectively. The mean annual CWR for the 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 the 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 reuses stormwater for irrigation as an order-of-magnitude check. The mean annual irrigation demand of the 63.7-ha golf course was estimated as 2,500 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 in the study area was estimated as 3,000 m^3 ha^{-1} yr^{-1} based on the historical data (2006–2100), which was projected to increase to 4,100 m^3 ha^{-1} yr^{-1} in future, based on data from the climate-change prediction model. The estimated water demand for public open spaces was higher than the mean annual stormwater reuse 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. It is also likely that not all of the golf course site is irrigated, e.g. sand traps and tree verges would normally not require watering.

6.3 RESIDENTIAL GARDEN IRRIGATION

In the estimation of water use for residential garden irrigation, it was necessary to disaggregate the domestic water demand into specific end uses. Domestic demand can be categorised into indoor or outdoor (Jacobs and Haarhoff, 2004). Residential garden irrigation is an outdoor demand, together with other uses such as swimming pools and car washing (Jacobs and 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 (Figure 6-8). Thus, no adjustment was undertaken over the model period 2006–2015.
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: .

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

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	10,947	126	1,410	1,530	15,561
Nyanga	8,595	30	1,134	792	5,439
Crossroads	5,346	21	27	21	5,241
Philippi	25,401	183	1,659	543	36,627
Lotus River	6,591	18	1,056	591	636
Parkwood	1,248	6	717	42	444
Grassy Park	4,029	9	384	84	204
Zeekoevlei	117	3	0	0	9
Pelican Park	2,250	33	27	57	888
Wynberg	1,977	24	2,100	879	147
Wetton	723	-	57	66	12
Ottery	1,749	0	141	105	210

	Houses on a separate stand	Traditional structure	Flat/ cluster	Semi-detached house	Backyard/ informal
Royal Cape	183	-	0	3	12
Elfindale	663	-	96	24	57
Southfield	1,791	3	186	102	72
Seawinds	465	3	6	738	177
Lavender Hill	2,310	18	1,338	267	1,182
Vrygrond	2,568	39	138	321	2,172
Plumstead	4,566	27	1,671	672	144
Total	**81,519**	**543**	**12,147**	**6,837**	**69,234**

iii. The 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 size to coincide with the CCT's water use bands. The categories are <200 m^2, 200–500 m^2, 500–1,000 m^2, 1,000–1,500 m^2, 1,500–2,000 m^2 and >2,000 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 smaller 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 properties without residential garden areas, 69,329 houses in the entire study area were considered for the 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	500–1,000	1,000–1,500	1,500–2,000	>2,000
Crossroads	158	297	658	1,384	1,761	2,079
Elfindale	124	374	708	1,159	1,610	2,859
Grassy Park	161	371	667	1,211	1,764	2,317
Gugulethu	146	386	641	1,236	1,613	2,893
Lavender Hill	151	297	650	1,130	1,558	2,323
Lotus River	170	360	650	1,190	1,653	2,428
Nyanga	145	351	656	1,235	1,899	2,486
Ottery	154	377	675	1,131	1,681	2,014
Parkwood	159	430	634	1,209	1,707	2,205
Pelican Park	142	333	646	1,066	1,916	2,321
Philippi	150	317	680	1,271	1,751	2,308
Plumstead	140	386	621	1,180	1,990	2,799
Seawinds	140	315	594	1,227	1,773	2,320
Southfield	160	376	608	1,209	1,903	2,317
Wetton	154	373	633	1,131	1,681	2,014
Wynberg	139	337	665	1,225	1,651	2,403
Zeekoevlei	126	312	630	1,133	1,727	2,321

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 was assumed – as for public open spaces. The mean monthly CWR values for a residential garden in the modelled period (2006–2016) are provided in Figure 6-9, including the maximum and minimum for the wettest year (2013) and the driest year (2015). The CWR for residential garden irrigation in the 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 the driest (2015) years, respectively. The predicted CWR 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 of 8.8 mm^3 and 20.5 mm^3 for the wettest year (2092) and the driest (2095) year, respectively.

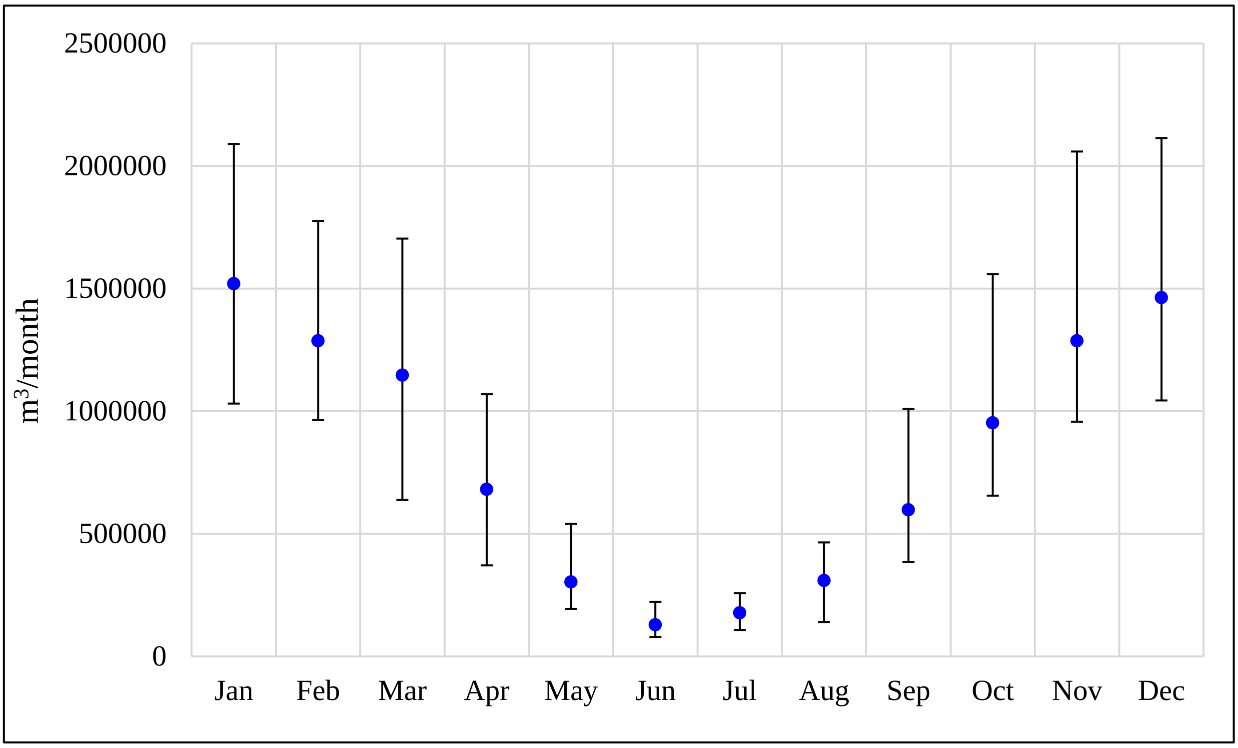

Figure 6-9: Mean monthly residential garden irrigation demand for the study area

6.4 WATER-USE ESTIMATION FOR TOILET FLUSHING

The unit water demand analysis method (Rinaudo, 2015), one of the approaches discussed in Section 2.5.4, was adopted to estimate 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 those of Van Zyl et al. (2008) and Smith (2010) have determined a typical frequency of toilet use (flushes per day) as four flushes per person per day for medium- to high-income households, and three flushes per person per day for low-income households. 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 in 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	Household size	Low-income household	Middle-income household	High-income household
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 for non-potable water demands, including the irrigation of urban agriculture, residential gardens and public open spaces, and for toilet flushing. The potential use of stormwater was considered as a supplementary source to existing resources (i.e. water from dams) and was 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. the 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, 14 and 14 mm^3, respectively. The mean monthly demand volumes over the period are presented in Figure 6-10.

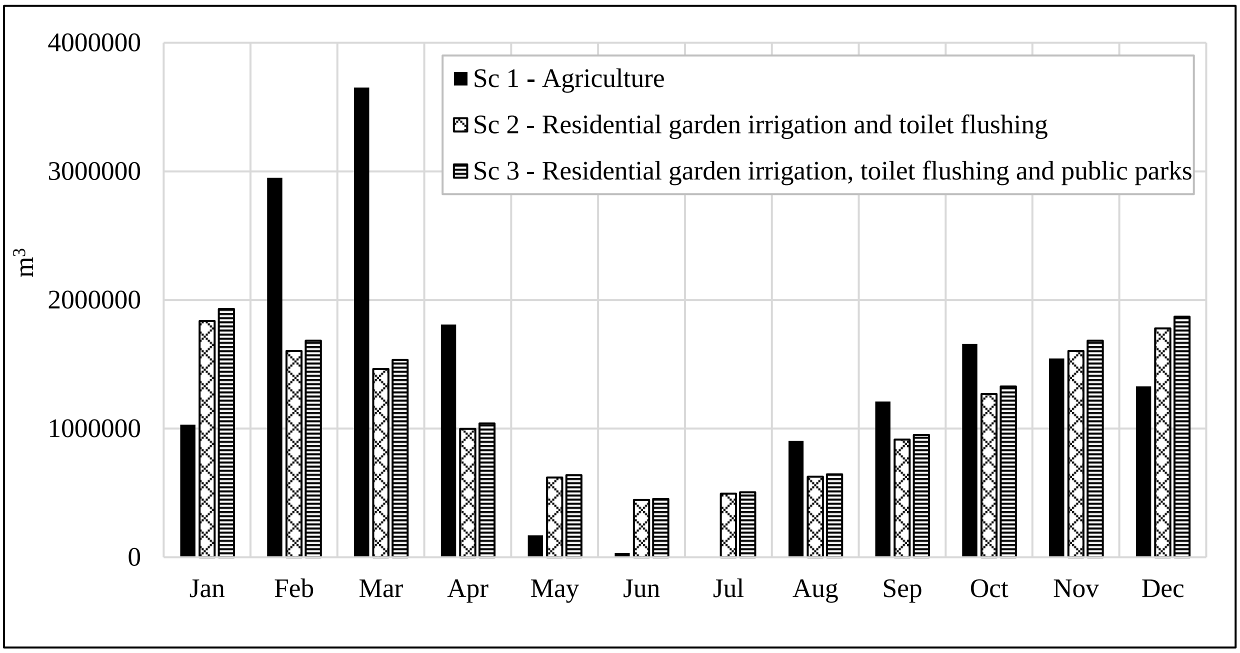

Figure 6-10: Mean monthly estimated demand volumes

6.6.2 Assessment of storage for stormwater harvesting and supply

A volumetric reliability 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 and 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 6-4 (Mitchell et al., 2007) as discussed in Section 0. The total storage capacity in the stormwater ponds was estimated at 1 mm^3 (Table 4-9:). This capacity was assessed against the need for storage capacity required for SWH to determine reliability and adequacy using Equation 6-3 and 6-4. The capacity of the vleis (Zeekoevlei and Rondevlei) (

Figure 3-10:) of some 5 mm^3 and 1 mm^3, respectively, was also assessed against the need to balance 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 non-potable water

The ideal storage required in the study area to account for the mismatch between the availability of stormwater and the various demand scenarios was estimated in a stepwise manner with the capacity provided in the vleis. The simulation was based on the YAS model (Equation 6-4) with RTC on the stormwater ponds and vleis (Zeekoevlei and Rondevlei). The results, presented in Figure 6-11, show that SWH with 1 mm^3 balancing storage (the current capacity available in the stormwater ponds) was only adequate to supply 44, 60 and 58% for the 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).

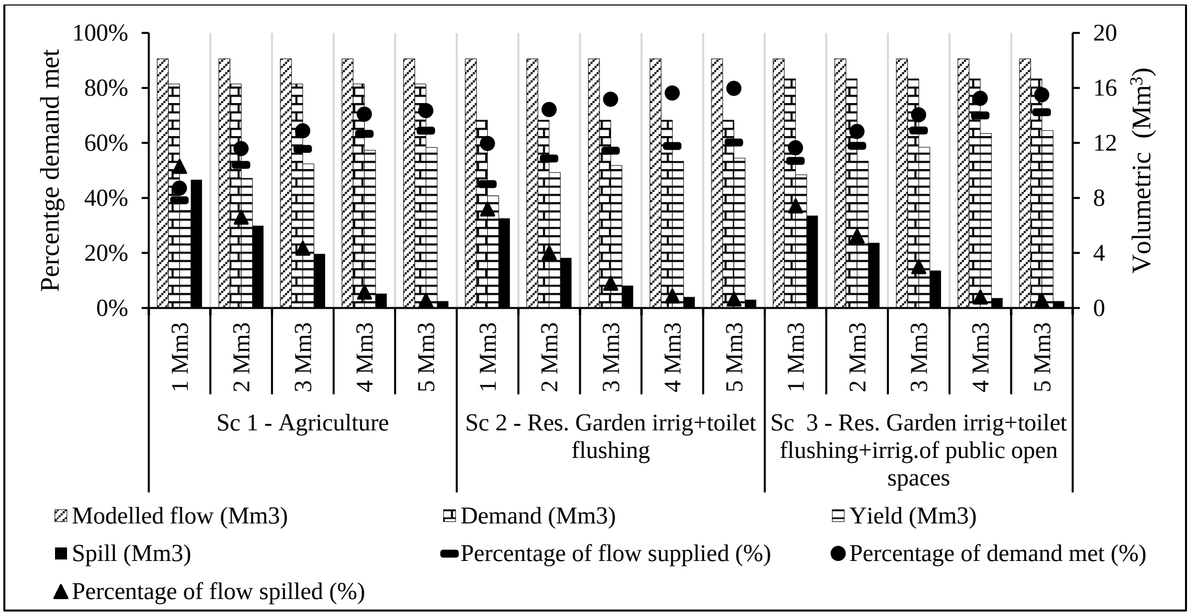

Figure 6-11: Assessment of storage for the various demand scenarios

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 the application of RTC, and physical expansion was unlikely due to land limitations typical in urban areas, enlarging the vleis (Zeekoevlei and Rondevlei) to provide additional storage seems to be the most promising option. However, the vleis are currently used for other purposes, such as recreation – including sailing – and maintaining the ecology, which requires a permanent pool of water. The ecological sensitivity and recreational activities in the vleis were the driver for the consideration of alternative storage options, such as groundwater through managed aquifer recharge and recovery.

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, as indicated in Figure 3-11. A similar assessment as that discussed in Section 0 was undertaken with SWH to supply potable water. The results with various storage and pump capacities, and corresponding yield and spillage are presented in Figure 6-12.

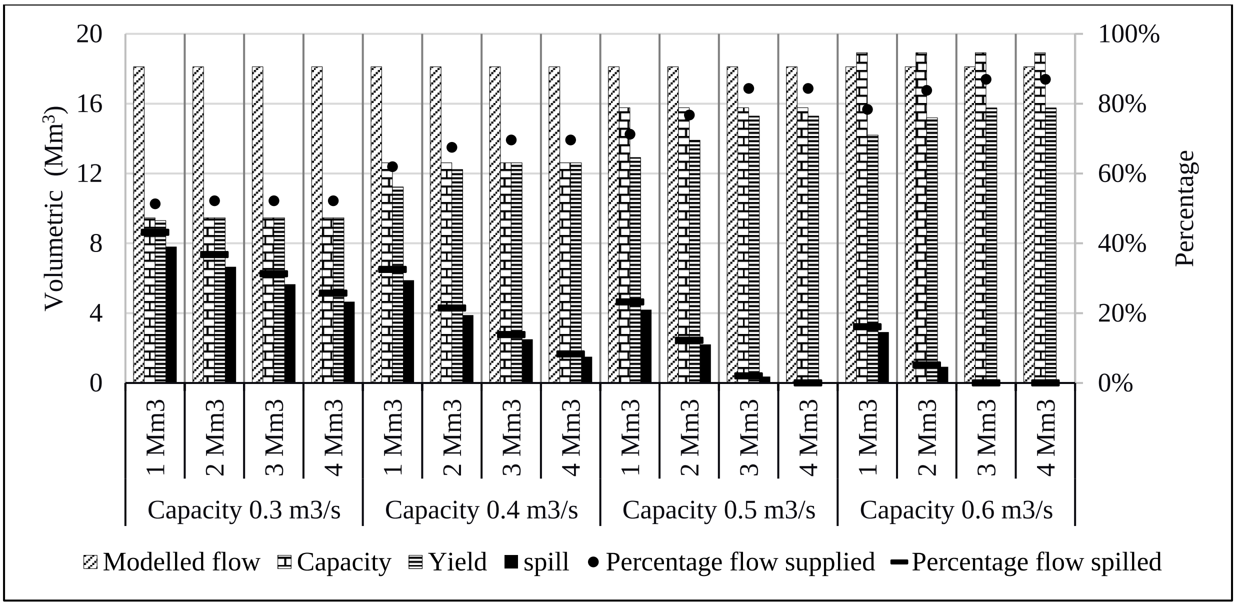

Figure 6-12: Selection of pump capacity for stormwater transmission

It was determined that, for potable water that is required all year round, the yield was not that sensitive to changes in storage volume (Figure 6-12), but rather linked 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 one with a 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, while a trial design (in Appendix) indicates the need for a DN 450 PN 20 pipe and two online 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.

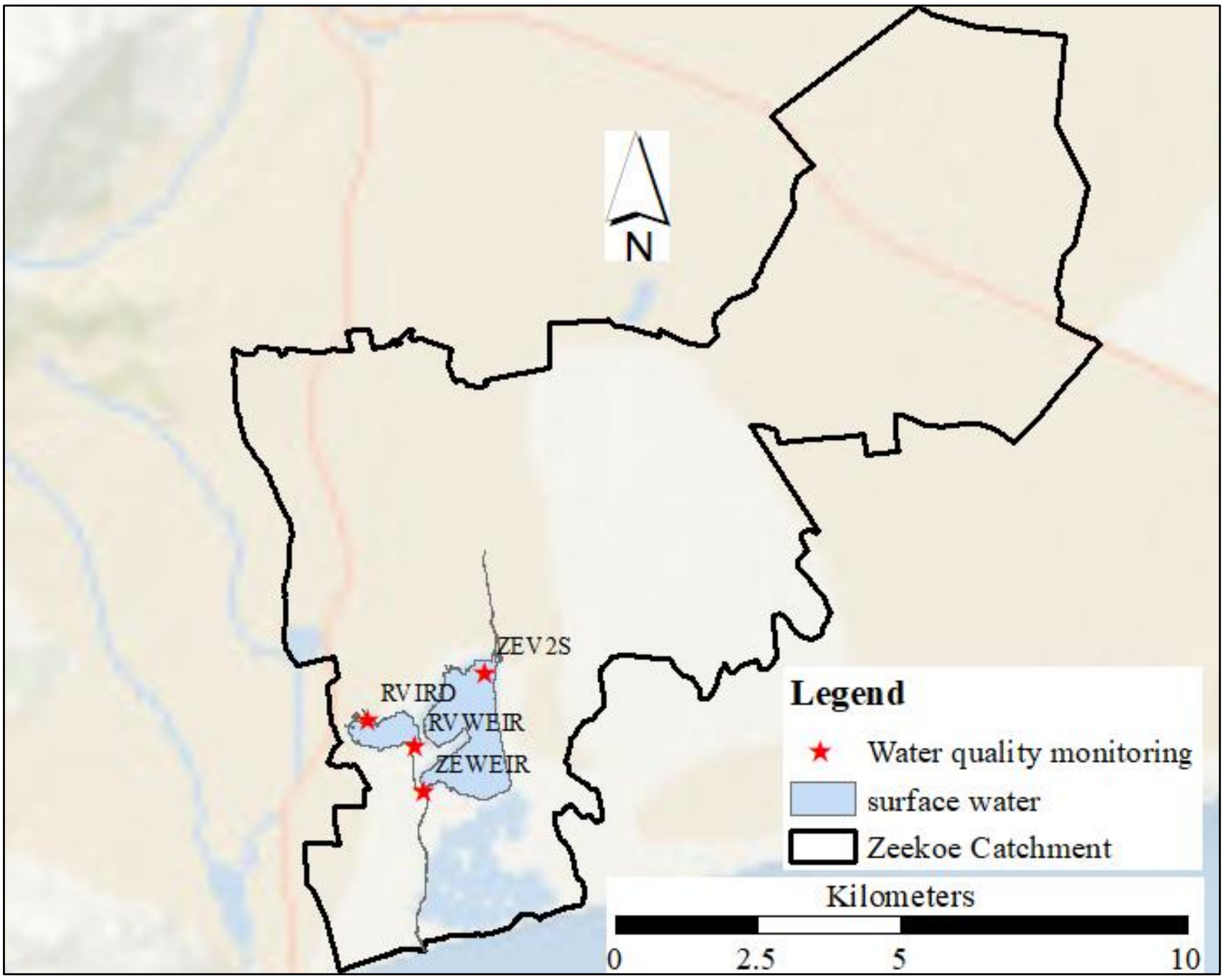

Figure 6-13: Water quality monitoring sites in Zeekoevlei and Rondevlei (after CCT, 2015)

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 Table 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 – 1,000	Arsenic mg/ℓ	Health	0.1–2
Conductivity (at 25 °C mS/m)	Aesthetic	40–540	Cadmium mg/ℓ	Health	0.01–0.05
Total dissolved solids (mg/ℓ)	Aesthetic	260–3,500	Chromium mg/ℓ	Health	0.1–1
pH level		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 25 °C mS/m)	Aesthetic	≤ 170	Cadmium µg/ℓ	Chronic	≤ 3
Total dissolved solids (mg/ℓ)	Aesthetic	≤ 1,200	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 the 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 (Table 6-7) and National Drinking Water Standards (Table 6-8), 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.

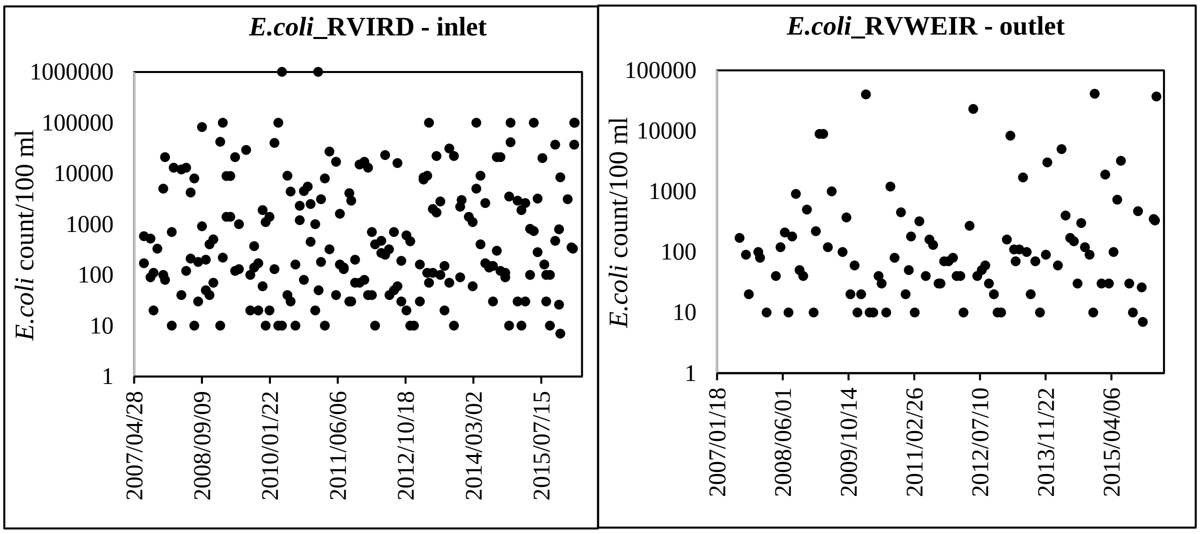

Figure 6-14: *E. coli* time series measured at Rondevlei (after CCT, 2017)

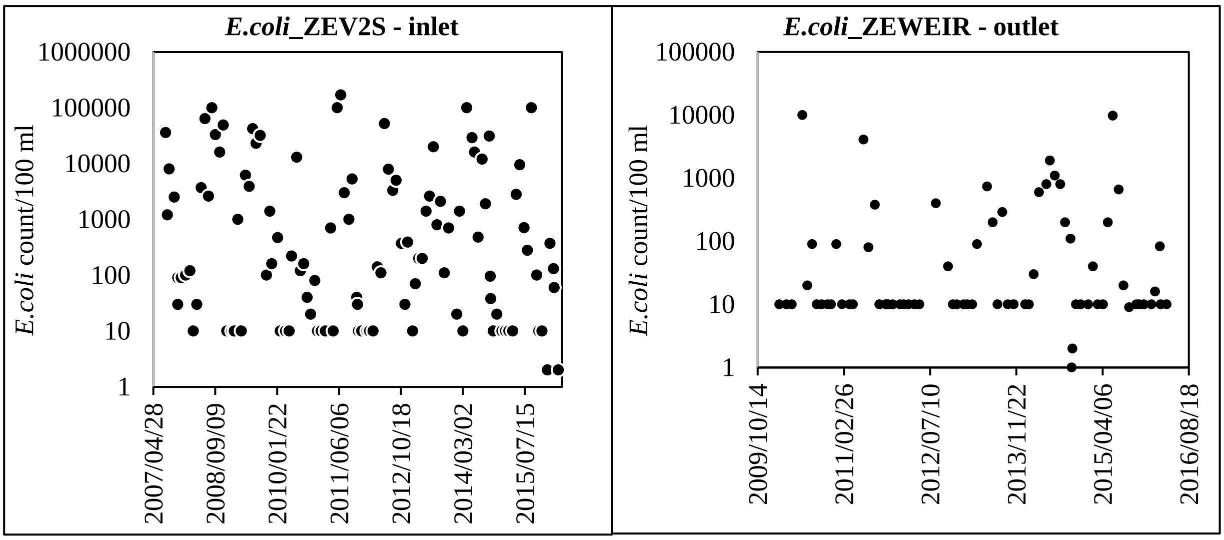

Figure 6-15: *E. coli* time-series measured at Zeekoevlei (after CCT, 2017)

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 tests for several nutrients. The results show relatively low levels of nutrients, as presented in Figure 6-16.

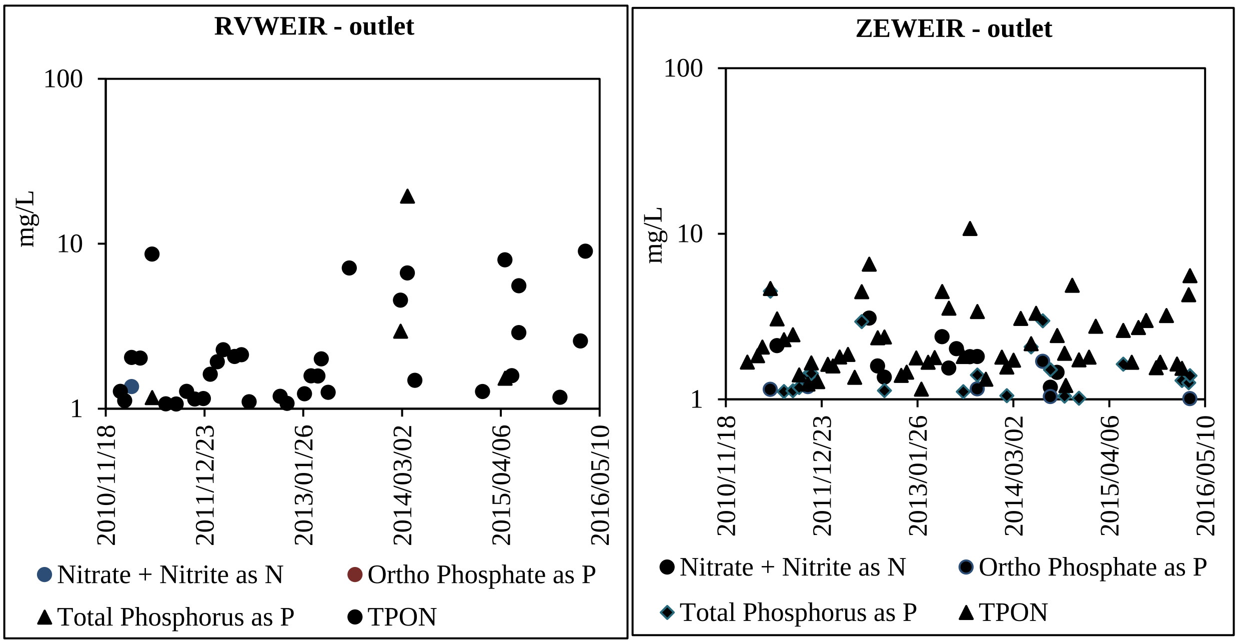

Figure 6-16: Nutrient time series measured at the outlet of the vleis

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 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 electrical conductivity (EC) (DWA, 1996), typically estimated with Equation 6-4.

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

where TDS = Total dissolved solids (mg/ℓ); EC = Electrical conductivity (mS/m)

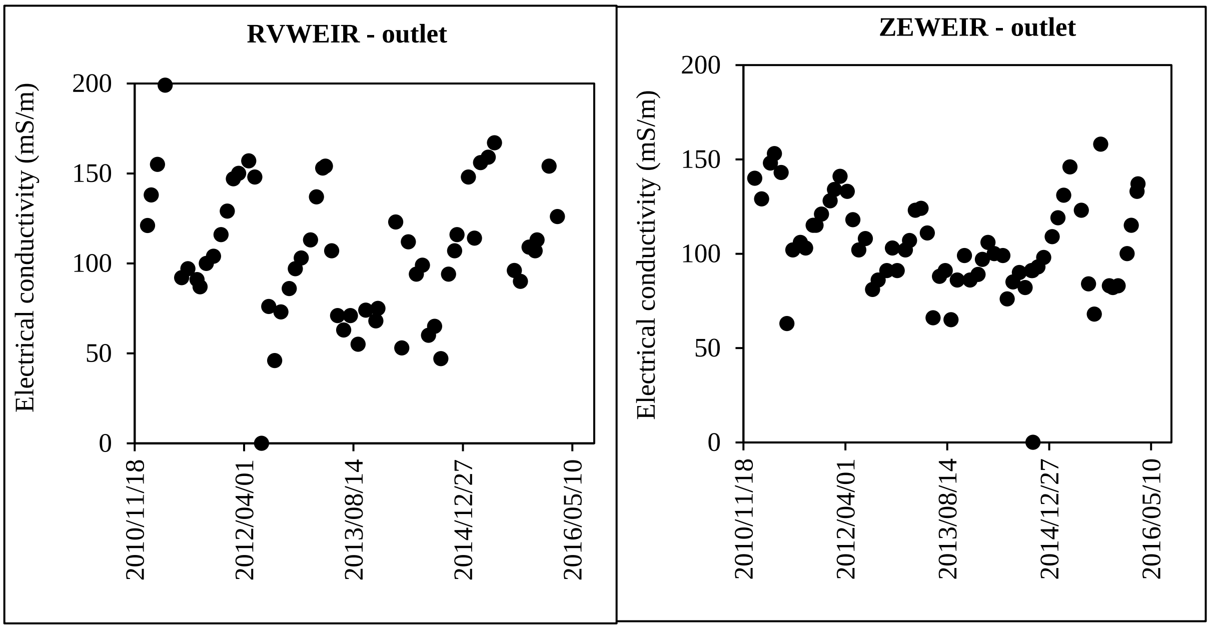

Figure 6-17: Total dissolved solids and electrical conductivity at the outlet of the vleis

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

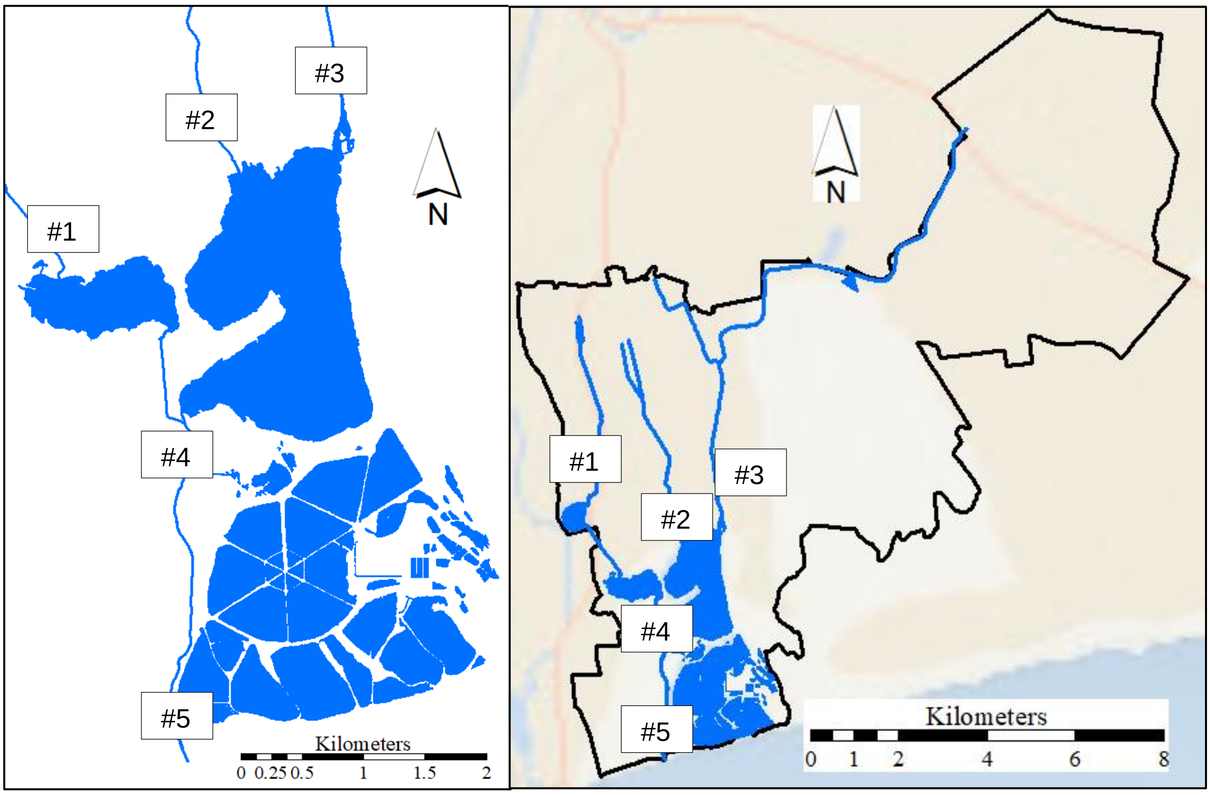

Figure 6-18: Water quality test sites for total dissolved solids and electrical conductivity

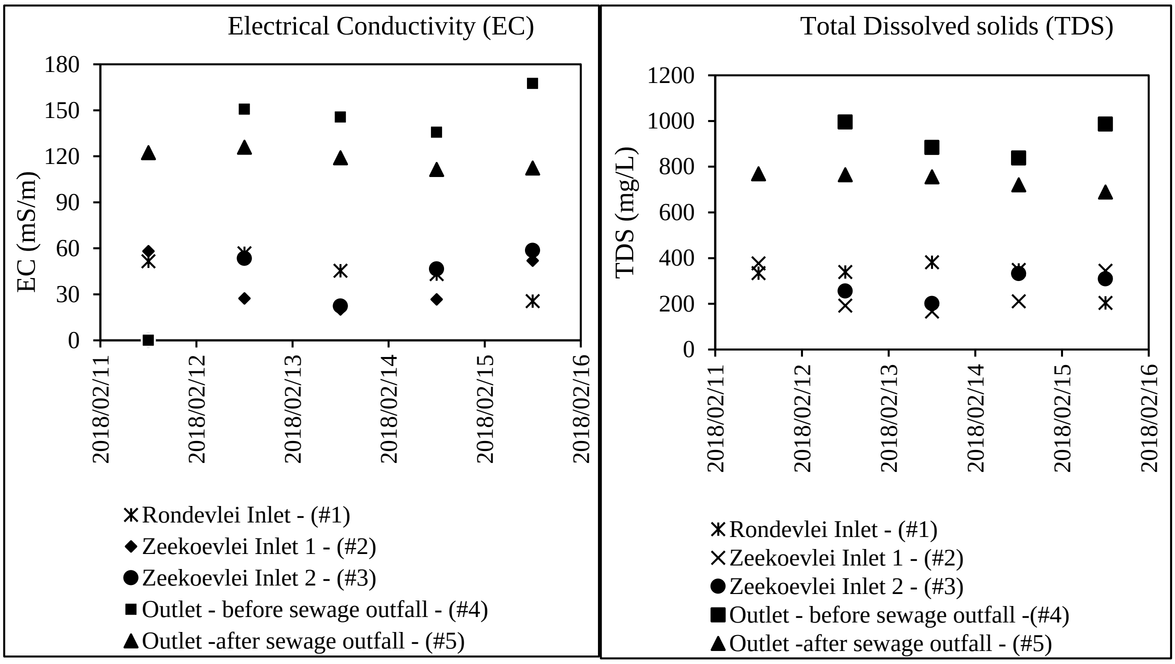

Figure 6-19: Total dissolved solids and electrical conductivity at the vleis

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 to 7.1 with a correlation coefficient of 0.41 to 0.94.

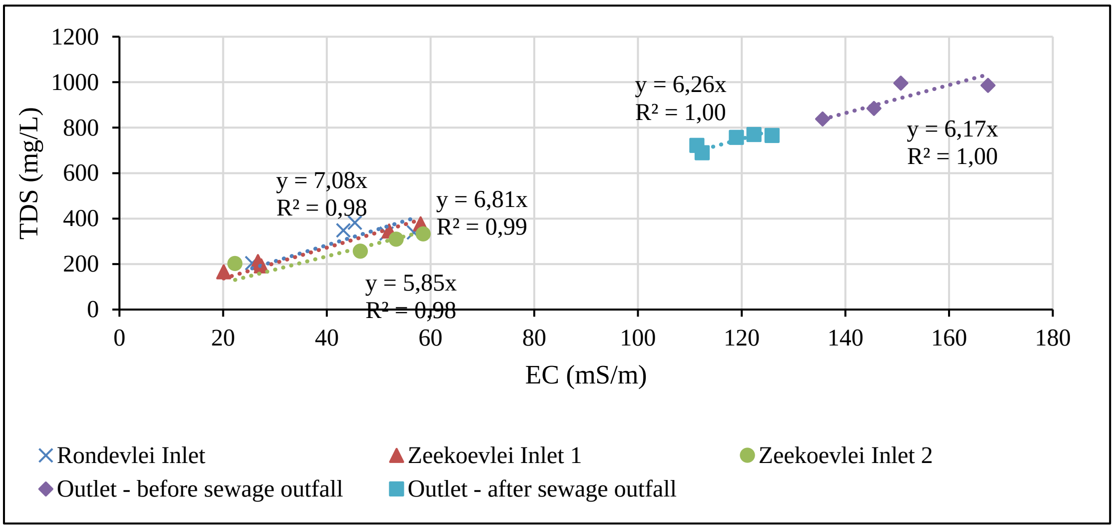

Figure 6-20: Linear relationships for total dissolved solids and electrical conductivity

No specific treatment is proposed for TDS and EC as the measured values were relatively low compared with the South African Water Quality Guidelines for Irrigation in Table 6-7: and the National Drinking Water Standards in Table 6-8: .

6.7.5 Heavy metals

Stormwater samples were collected from 20 to 24 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 the Swiss Tropical and Public Health Institute. 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 June	<3	<1	<7	<7	<5	<3	<1	<7	<7	<5
21 June	<3	<1	13	<7	<5	22	<1	12	<7	12
22 June	<3	<1	<7	<7	<5	<3	<1	<7	<7	22
23 June	<3	<1	<7	<7	<5	<3	<1	<7	<7	<5
24 June	7	<1	<7	<7	<5	13	<1	<7	<7	<5

*Combined outlet from Zeekoevlei, Rondevlei and the Cape Flats WWTW

With the grab samples indicating a relatively low concentration of heavy metals in the vleis compared with the South African Water Quality Guidelines for Irrigation in Table 6-7: and the National Drinking Water Standards in Table 6-8: , 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 processes.

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's 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 (the 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 water to one of the existing WTPs for blending and treatment, it is likely that no specific treatment would be required.
- **For potable water uses (irrespective of source):** Full conventional treatment with sand/ultra-filtration and final disinfection with UV 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 of being supplied with stormwater. The non-potable demands included the irrigation of urban agriculture, public open spaces and 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 4,100 mm^3 and 9,600 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 the 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 4,000–6,000 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-estimate evapotranspiration values compared to the Class A pan.

The mean annual water requirement for the 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 3,000 m^3 ha^{-1} yr^{-1}, based on the historical data (2006–2100), which was projected to increase to 4,000 m^3 ha^{-1} yr^{-1} in future, based on data from the climate change prediction models. In the study area, there is a 63.7-ha golf course that already harvests and reuses stormwater for irrigation. On mean, around 2,500 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 public open gardens was also higher than the mean annual stormwater reuse 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 of 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 the driest year (2015) were 7 and 15 mm^3, respectively. The mean annual CWR 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 the driest year (2095) were 9 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 the temporal mismatch between stormwater and demand. RTC on ponds provided storage equal to 5.5% of the mean annual flow volume to meet 44–67% of demand and 37–51% of spill. RTC on both ponds and vleis provided storage equal to 22% of the mean annual flow volume to meet 70–79% of demand and 4–11% of 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's capacity for stormwater abstraction, treatment and supply. The suitable plant capacity was determined in an optimisation process to maximise harvested stormwater volumes and minimise 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 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 been significantly constrained. The associated costs are determined and discussed in Chapter 7. In the economic analysis (Chapter 7), capital, operation and maintenance cost estimates are provided for stormwater treatment to potable water standards. Corresponding seawater desalination is included for comparison purposes and assessment.

CHAPTER 7: ECONOMIC ANALYSIS

In this chapter, the viability of stormwater harvesting 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. Unit costs in R/kℓ were estimated and compared with the indicative costs of competing water sources presented in the City of Cape Town's 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 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 rates, 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 the discount rate determined from the South African government's 10-year bond expressed as real values, i.e. adjusted for inflation (Van Vuuren and Van Dijk, 2006).

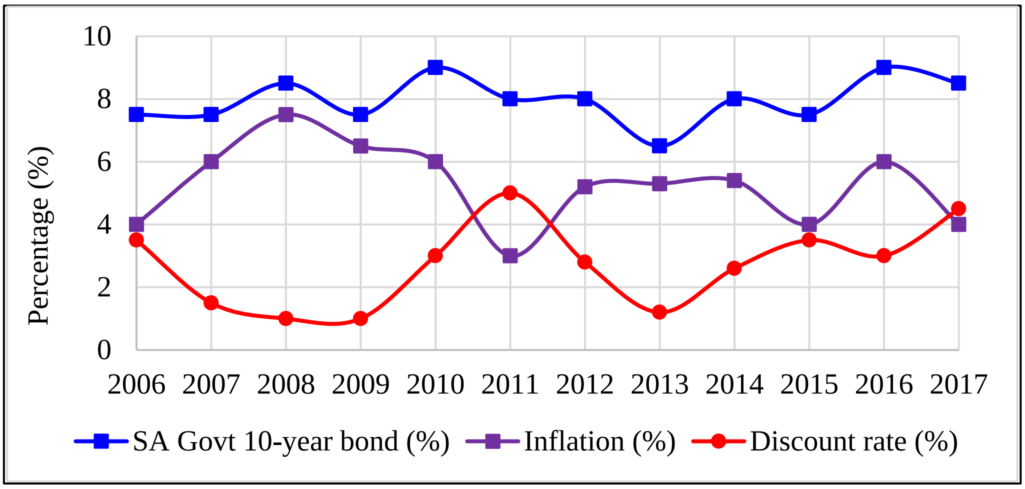

Figure 7-1: Discount rate over a 10-year period (after StatSA, 2018)

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. using the difference between the mean 10-year government bond and the inflation rate). To account for the potential uncertainty associated with the selected discount rate, the analysis was repeated with 4.5% (i.e. the maximum difference between the 10-year government bond and the inflation rate). The expected life expectancy in Equation 7-1 to account for the frequency of the 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 were selected (Mackenzie, 2010) and used in the economic analysis.

Table 7-1: The 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 or replace	15–25	25+
Fixed facilities	Expensive to enlarge or replace	20–25	50+
Equipment	Easy to refurbish or 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); Dillon et al. (2010); McArdle et al. (2011); Gerrity et al. (2014); USEPA (2016) and Blersch and Du Plessis (2017) are summarised in Table 7-2:.

7.3 CAPITAL COSTS

Capital cost estimates were derived from the Department of Water Affairs (DWA)'s 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 per year. The discount rate and the design period were estimated as discussed in Section 0. The resulting estimated capital costs for the 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 35 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 – 3,200 m^3	4,653	4,653	4,653	4,653
	Supply reservoir in steel – 1,588 m^3	2,534	2,534	2,534	2,534
Transmission in decentralised system	Pump (0.45 m^3/s)	2,958	2,958	2,958	2,958
	16.9 Km DN400 PN 20	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 external reservoirs, the cost of the transmission pipeline was estimated using Equation 7-2 and 7-3 (Bester et al., 2010).

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

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

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)
26.9 km DN400 PN 20 – steel pipeline	45,312
3 No. pump stations	8,875
Total	**54,187**

7.4 OPERATION AND MAINTENANCE COSTS

Various approaches for the estimation of operation and maintenance costs are possible, including as a percentage of investment costs, or based on the past performance of similar utilities as costs per unit volume of water produced (Boshoff et al., 2009).

A combination of both these options was 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 were calculated 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. Meldrum et al., 2013 and Pabi et al., 2013). The estimation of the unit energy costs was based on the CCT's electricity tariffs for 2017, as shown in Table 7-5:.

Table 7-5: Electricity tariffs for 2017 (CCT, 2017)

Large power user	**Time of use**	**Units**	**Low voltage (500–1,000 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 (>1,000 kWh/ month)**	**Small power user 2 (<1,000 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
Poly-aluminium chloride	0.00384	95.8

Maintenance typically comprises 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. Central Public Health and Environmental Engineering Organisation (CPHEEO) (2005), Van Zyl (2014) and the infrastructure asset management guideline of South Africa's 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.

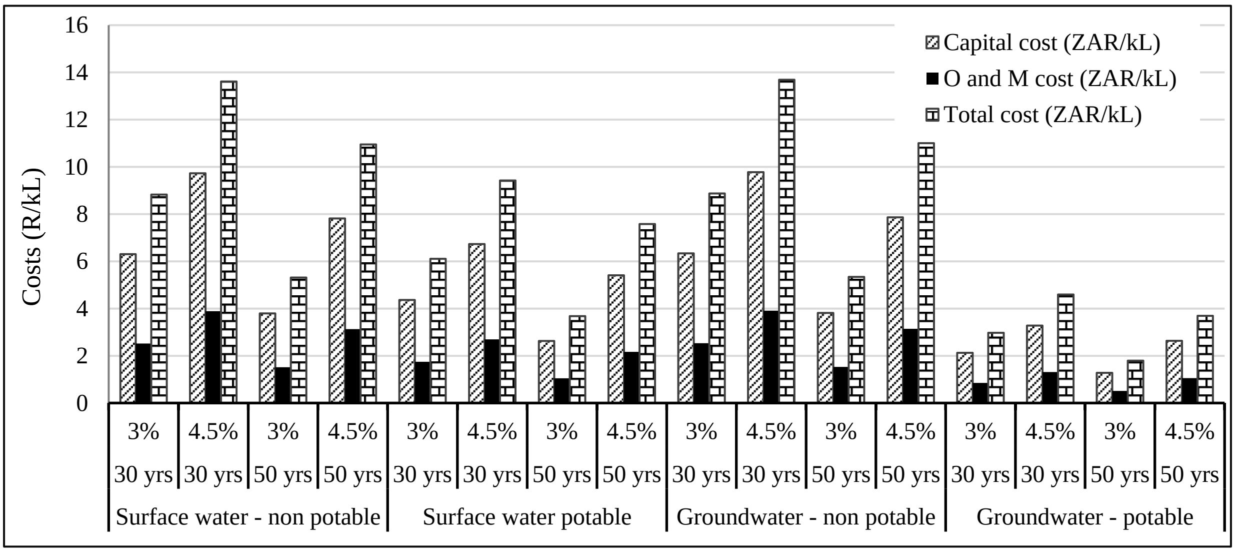

Figure 7-2: Costs for supply from various systems per kℓ

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

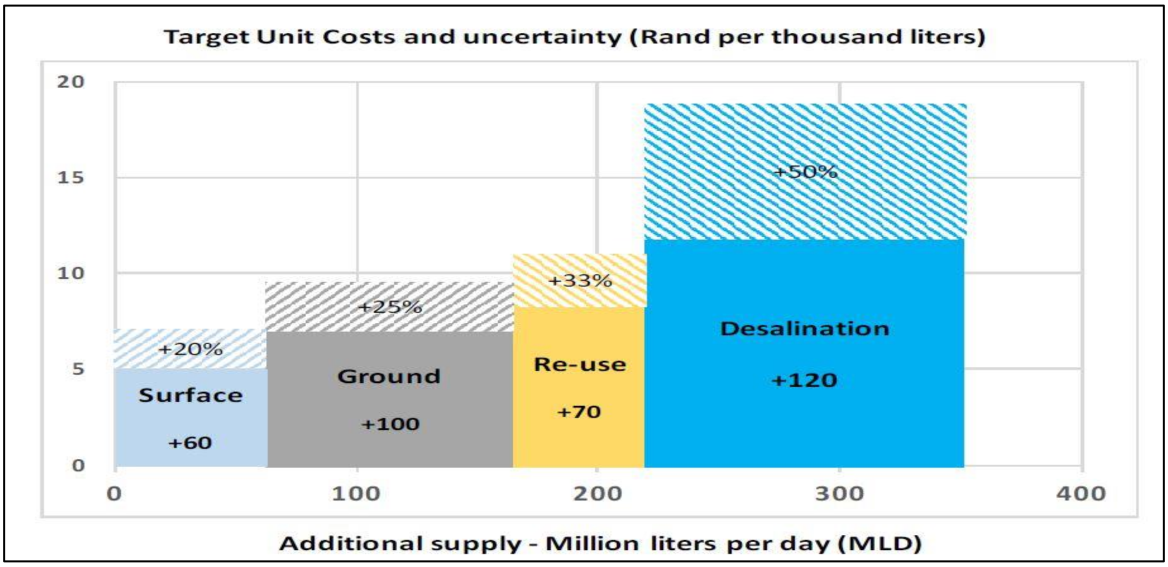

Figure 7-3: Costs of water from proposed systems in Cape Town per kℓ (CCT, 2018)

The CCT's Water Outlook Report of 2018 indicates 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), large-scale desalination (R12–R19/kℓ) and small-scale desalination (R35/kL). From this comparison, it appears that SWH is competitive with these alternatives, with MAR&R

combined with potable water demand using the CCT's existing water reticulation system being the most cost-effective approach.

CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

This chapter provides a concluding overview of the areas investigated and a summary of the findings, as well as the study's contribution and recommendations. Section 0 presents an overview of the areas investigated, highlighting the proposed stormwater harvesting options (i.e. from the surface and groundwater storage) and water demand (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's 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 were investigated with a focus on available storage in the study area, i.e. stormwater ponds and vleis (with storage enhancement using RTC), and the 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 the extended detention of water to provide the balancing storage required for SWH. RTC was also essential for flood control through pre-emptive draining before storm events.
ii. The 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. An 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. An investigation of the full treatment of stormwater – both locally and at a remote existing WTP – to determine the opportunity for supply to meet the potable water demand.
v. An economic analysis 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, and 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 between the availability of the resource and the demand. Large storage areas were necessary, in particular, for non-potable water uses, e.g. the 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 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 between the 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 the irrigation of public open spaces). The storage in the ponds was only able to supply 44, 60 and 58% of the demands for 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 between the availability of stormwater and the 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 of 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). Minimal increase in demand was met, as well as the reduction of spillage with a capacity of 5 mm^3. In general, it was determined that the stormwater supply to non-potable demand was sensitive to balancing storage, and required a capacity of 20–30% of stormwater volume to maximise meeting the demand and minimise loss through spillage.

The 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 on meeting demand, and the optimisation of the SWH system was based on the plant's 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. An 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 to 200 mm with an increase in the 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 consisting mainly of residential (formal and informal) and light industrial land uses. The study area also comprised 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 planned developments and a hypothetical increase in imperviousness. An assessment with a mean imperviousness of 50% (allowing for the developments planned 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 of 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 features suitable for MAR&R from groundwater storage, i.e. the availability of a large unconfined aquifer with a depth ranging between 20 and 55 m, pervious soils (sandy soils) and reasonably flat terrain (a 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 having abstraction rates greater than 5 ℓ/s (CCT, 2005). Some studies, such as those of 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 a 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. The deliberate recharge of the 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 would range from 3.5–8.1 ℓ/s from 140 boreholes in the 89 km^2 catchment.

8.1.3 Volumetric assessment 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 to 25 mm^3 for the driest (2015) and the wettest year (2013). 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 downstream surface storage, where SWH is undertaken, would reduce from 18 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. in Sc 1, Sc 2 and Sc 3 of 16, 13 and 14 mm^3, respectively. On the other hand, the enhanced infiltration would augment the groundwater resource from about 20 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 the adoption of MAR&R, the mean annual surface water volume and natural groundwater infiltration would be about 18 and 20 mm^3, respectively. Volumetrically, both resources are sufficient separately to supply the identified non-potable demands. Thus, the use of both surface water and a natural groundwater resource would be sensible for non-potable demands. However, it was noted that the 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 were largely marshland and not linked directly to the ocean. The land use change would significantly reduce infiltration, thus resulting in the groundwater resource being lost. The 10 mm^3 natural groundwater resource would no longer be adequate to supply the 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 significantly large and would require equally large storage, e.g. coastal reservoirs for the 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 the provision of 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 poor stormwater quality. Based on monthly grab samples of stormwater, the risks associated with poor water quality were significant in the study area as, in some cases, *E. coli* levels were greater than 1 x 10^6 counts per 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 treating 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. the irrigation of urban agriculture, residential gardens and public open spaces, were mainly in summer, thus mismatched with the availability of the stormwater resource, which was largely available in winter (

Figure 3-15:). In the yield-demand analysis, the impact from toilet flushing (an indoor water use) with regard to enhancing the performance of volumetric reliability and supply efficiency was negligible. The toilet flushing demand is not sufficient to account for much of the available stormwater 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 volumetric (an adequate yield to meet demand), supply should be for demands that are available throughout the year, e.g. potable water use. Clearly, the optimal use of stormwater requires a shift in use to potable water uses. Treatment to potable standards would also eliminate potential public health risks from cross-connections. It was also determined that 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 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 is through the existing potable water system. In the case of SWH from groundwater storage, it was determined that abstraction from boreholes 400 m from the ponds and a travel time of 300 days would allow for a reduction in pollution associated with *E. coli* to values less than 10 counts per 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 and shanty towns), that generate wastewater, and litter discharges into the drainage channels, particularly in the upper reaches of the catchment. The CCT's monthly grab samples of stormwater quality showed that the drainage system in the study area was 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 benefitting 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 the die-off of pathogens in the water to values less than 10 *E. coli* counts per 100 mℓ of pond water. 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 attempted to contribute to the debate on alternative water resources by considering the possibilities of SWH from surface and groundwater storage for supply to potable and non-potable demands at a catchment scale. The factors that were determined to be important and needed to be considered for the efficacious application of an SWH system included the availability of storage (surface or groundwater), the catchment characteristics (terrain, soil types, level of development, population density) and the 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 found that, in the Zeekoe catchment, the following applied:

- SWH is a viable water resource volumetrically (with sufficient quantity to meet a significant portion of water demand) and economically (it is 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. the irrigation of agriculture, residential gardens and public open spaces in areas such as Cape Town with rainfall limited to the winter period, storage in the range of 20–30% of the mean annual modelled stormwater volume would be required to balance the mismatch between the 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 the 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 an unconfined aquifer, need to be present. In the selection of the groundwater abstraction rate and distance of boreholes from ponds, the study confirmed that at least a one-year residence period should be allowed to provide for a reduction in *E. coli* to values less than 10 counts per 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 per 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 to 200 mm and an increase in mean daily temperature of 3–5 ºC 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 an 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 reservoirs 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. Besides the relief on existing water resources by such an alternative water source, additional benefits, e.g. stormwater quality improvement, were identified.

8.3 RECOMMENDATIONS FOR FURTHER RESEARCH

This research mainly focused on the prospects for stormwater harvesting in Cape Town through the identification and assessment of suitable storage reservoirs 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. These areas are recommended for future research as follows:

- In the case of non-potable water demand, a detailed investigation would be required to determine the perception and community acceptance of stormwater as a resource. The respondents and their reactions need to be categorised according to demographics (e.g. level of education, income and age group), preferred uses and under what conditions the resources would be accepted or considered (e.g. water scarcity and restrictions, high tariffs).
- A comprehensive study of SWH, considering all the catchments in the 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, commerce and institutions to determine if there are significant needs in the rainy seasons that might make it unnecessary to treat the water to a potable level of supply.

- Whether cost savings that might be achieved through the joint installation of a dual reticulation system in a greenfield 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 an SWH infrastructure by local residents and the willingness to bear the associated maintenance costs and management requirements.
- A pilot study is required to determine the suitability of bio-retention 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.
- A detailed exploration of additional benefits, including amenity values, such as increasing property values, biodiversity preservation and cooling to minimise the “urban heat” effect.

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APPENDIX 1: CONFERENCE PRESENTATIONS

Some of the work in the report have been presented at international conferences as listed below

OKEDI J and **ARMITAGE NP** (2018). Benefits of real-time control for catchment scale stormwater harvesting in Cape Town, South Africa. In: *11th IWA/IAHR International Urban Drainage Modelling Conference,* Palermo, Italy, 23–26 September 2018.

OKEDI J, ARMITAGE NP, CARDEN KJ and MAHINTHAKUMAR K (2017). An investigation into the potential for catchment scale stormwater harvesting in Cape Town, South Africa. *14th International Conference on Urban Drainage*, Prague, Czech Republic, 10–15 September 2017.

OKEDI J, ARMITAGE NP and CARDEN KJ (2017). Stormwater as a water resource – a case of Zeekoe catchment in Cape Town, South Africa. *6th World Sustainability Forum*, Cape Town, South Africa, 27–28 January 2017.

FISHER-JEFFES LN, **ARMITAGE NP**, CARDEN KJ, WINTER K and **OKEDI J** (2016). Addressing water scarcity in South Africa through the use of LID. *The EWRI International Low Impact Development Conference,* Portland, ME, USA, 20–23 September 2016.

OKEDI J, ALASTAIR R and **ARMITAGE NP** (2015). Towards improving the performance of Cape Town's stormwater ponds using a SuDS approach. *10th IWA/IAHR International Urban Drainage Modelling Conference,* 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

The provision of the data is subject to the User providing the South African Weather Service (SAWS) with a detailed and complete disclosure, in writing and in line with the requirements of clauses 1.1 to 2.4 (below), of the purpose for which the specified data is to be used. The statement is to be attached to this document as Schedule 1.

1 **Should the User intend using the specified data for commercial gain then the disclosure should include the following**:

1.1 the commercial nature of the project/funded research project in connection with which the User intends to use the specified data;

1.2 the names and fields of expertise of any participants in the project/funded research project for which the specified data is intended; and

1.3 the projected commercial gains to the User as a result of the intended use of the specified data for the project/funded research project.

2 **Should the User intend using the specified data for the purposes of conducting research, then the disclosure should include the following**:

2.1 the title of the research paper or project for which the specified data is to be used;

2.2 the details of the institution and supervisory body or person(s) under the auspices of which the research is to be undertaken;

2.3 an undertaking to supply SAWS with a copy of the final results of the research in printed and/or electronic format; and

2.4 the assurance that no commercial gain will be received from the outcome from the research.

If the specified data is used in research with disclosure being provided in accordance with paragraph 2 and the User is given the opportunity to receive financial benefit from the research following the publication of the results, then additional disclosure in terms of paragraph 1 is required.

The condition of this disclosure statement is applicable to the purpose and data requirements of the transaction recorded in Schedule 1 on page 2. This statement is effective from June 2014.

Bolepi House, 442 Rigel Avenue South, Erasmusrand, 0181 Private Bag X097, Pretoria, 0001 Tel: + 27 (0) 12 367 6000

Weatherline: 083 123 0500 www.weathersa.co.za sms line: *120*555# (Dial)

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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.

APPENDIX 4: HISTORICAL DATA AGGREGATED TO MONTHLY VALUES

APPENDIX 4A: HISTORICAL 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 4B: HISTORICAL DATA AGGREGATED TO MONTHLY VALUES: RONDEVLEI

Mean monthly precipitation (mm/month) – Rondevlei Station													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1996	1	20	24	24	49	202	97	113	113	66	59	34	802
1997	7	3	1	54	89	185	30	139	32	19	58	11	629
1998	11	0	12	42	181	60	110	41	35	24	45	38	597
1999	0	0	0	64	34	117	52	141	86	0	18	5	517
2000	11	0	15	9	62	71	109	60	77	10	3	6	432
2001	15	6	2	43	140	53	289	183	70	52	12	6	872
2002	77	17	9	32	103	85	93	94	25	27	25	0	586
2003	5	7	46	15	40	14	34	179	53	38	4	37	473
2004	10	3	39	70	12	95	94	107	44	103	1	9	585
2005	21	4	6	161	100	130	52	170	43	30	31	0	748
2006	0	20	2	30	130	148	90	109	77	40	34	19	698
2007	11	54	19	104	120	168	172	187	32	53	34	12	965
2008	20	34	9	9	151	137	244	138	208	20	42	14	1027
2009	10	8	4	23	133	148	120	111	53	31	76	3	720
2010	2	5	6	15	167	94	43	43	23	28	24	24	474
2011	6	2	9	34	78	127	24	63	40	19	14	8	424
2012	23	4	21	67	45	93	132	228	58	31	11	3	716
2013	26	42	49	79	99	186	59	242	73	41	110	4	1010
2014	29	3	52	37	88	165	187	153	20	5	16	2	756
2015	9	2	1	9	25	111	121	32	29	8	43	10	398

APPENDIX 4C: HISTORICAL MEAN MONTHLY MAXIMUM TEMPERATURE VALUES: AIRPORT

	Mean monthly maximum temperature (^0C) – 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: HISTORICAL MEAN MONTHLY MINIMUM TEMPERATURE VALUES: AIRPORT

	Mean monthly minimum temperature (⁰C) – 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: HISTORICAL MEAN MONTHLY PERCENTAGE HUMIDITY VALUES: AIRPORT

	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

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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.

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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 BC and CRANE RG (2006). Consensus between GCM climate change projections with empirical downscaling: Precipitation downscaling over South Africa. International Journal of Climatology **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

	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 (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 (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)

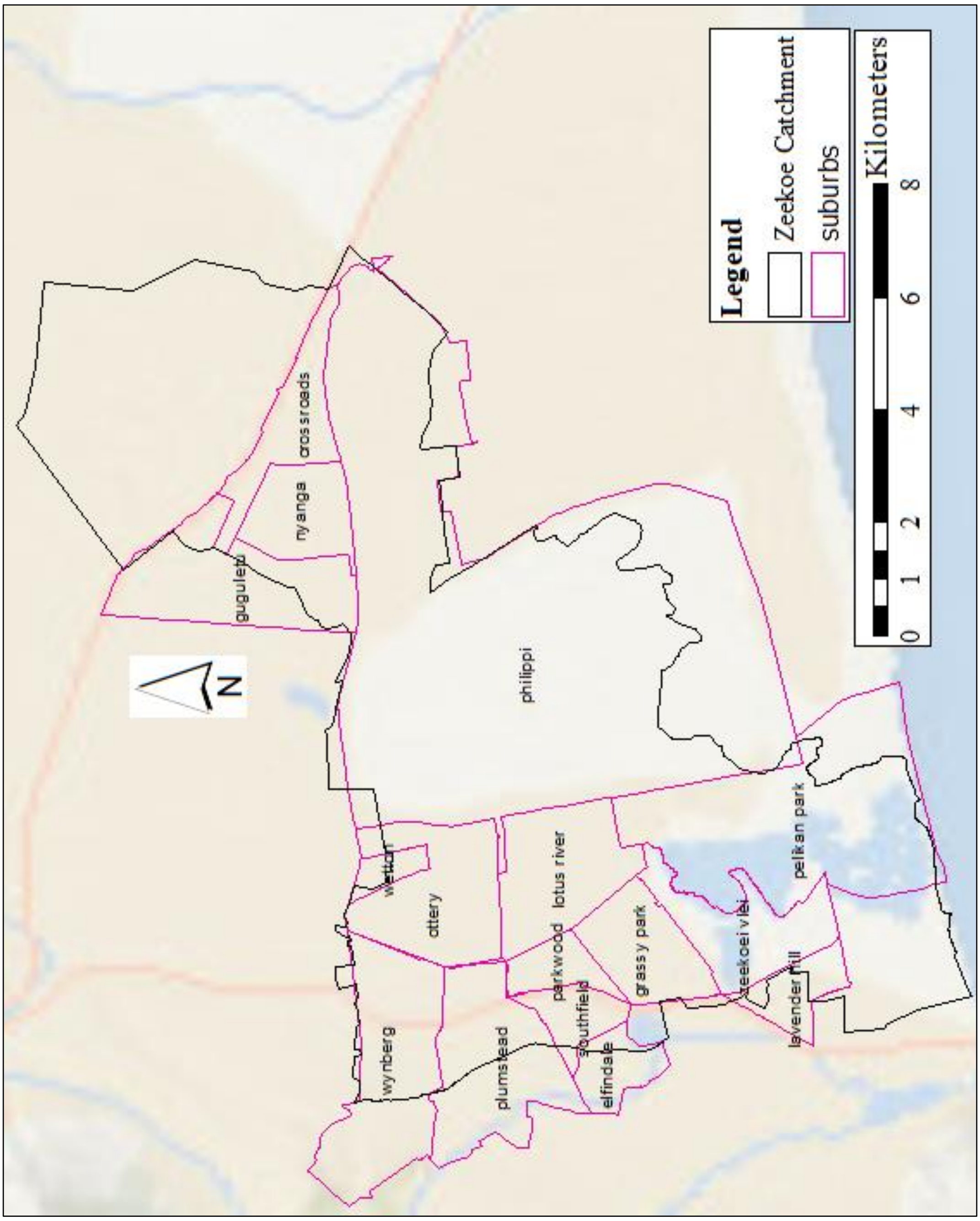
Climate change prediction models - mean monthly minimum temperature 2090 - 2100 (ºC) – Airport Station									
	HadGEM2-CC-rcp45	HadGEM2-CC-rcp85	inmcm4-rcp45	inmcm4-rcp85	IPSL-CM5A-MR-rcp45	IPSL-CM5A-MR-rcp85	IPSL-CM5B-LR-rcp45	IPSL-CM5B-LR-rcp85	MIROC5-rcp45
Jan	15.9	18.4	17.3	18.6	18.4	20.6	16.4	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

W12/05/3799

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;

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Making progress possible. Together.

14/12/2015
MARIO CARELSE Recommended ☒/Not Recommended ☐ Date
IMS SPECIALIST: WATER AND SANITATION

COMMENTS: ____________________

14 12 2015
JACO DE BRUYN Recommended ☒/Not Recommended ☐ Date
HEAD: INTEGRATED PLANNING,
STRATEGY AND INFORMATION MANAGEMENT
(DATA CUSTODIAN)

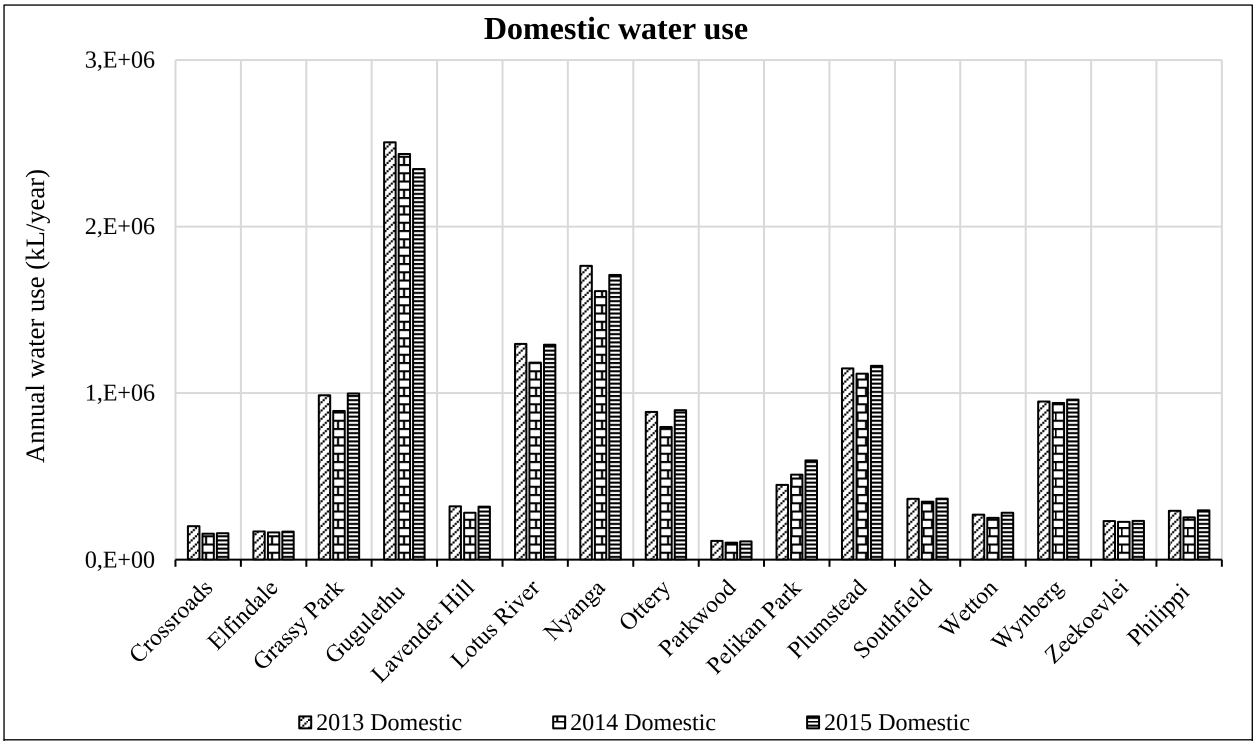
COMMENTS: ____________________

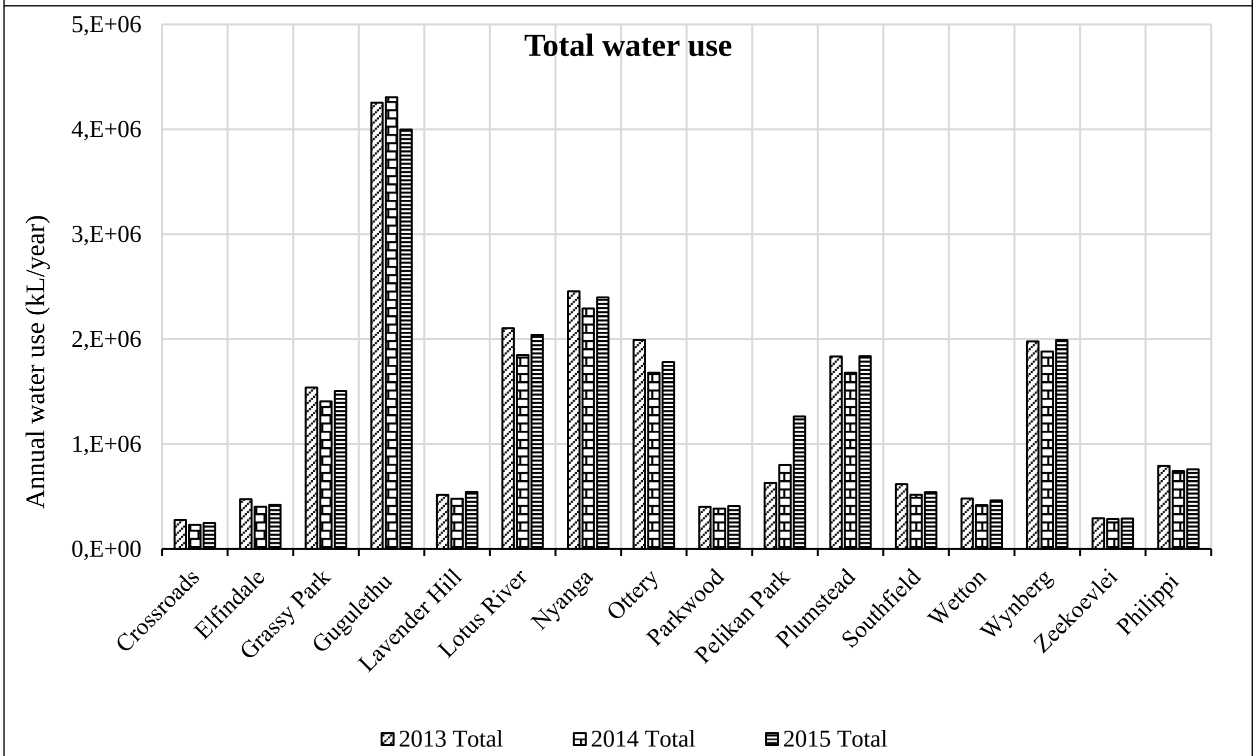
DR GISELA KAISER Approved ☒/Not Approved ☐ Date
EXECUTIVE DIRECTOR: UTILITY SERVICES

COMMENTS: Subject to conditions as listed

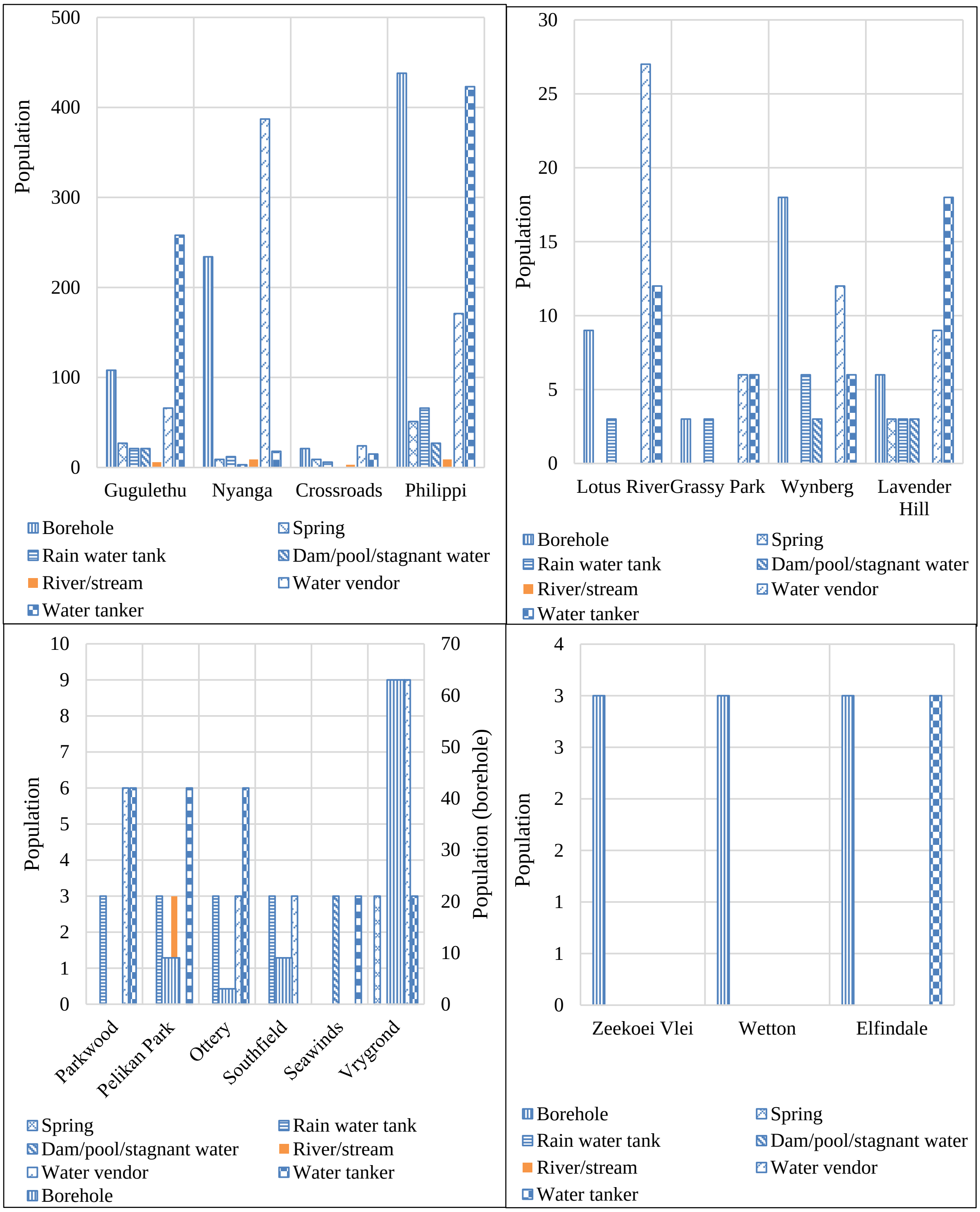
OFFICE OF THE EXECUTIVE DIRECTOR
UTILITY SERVICES
RECEIVED
28 DEC 2015
SIGNATURE ....

APPENDIX 6D: ANNUAL DOMESTIC AND TOTAL WATER USE PER SUBURB FROM RETICULATION SYSTEM

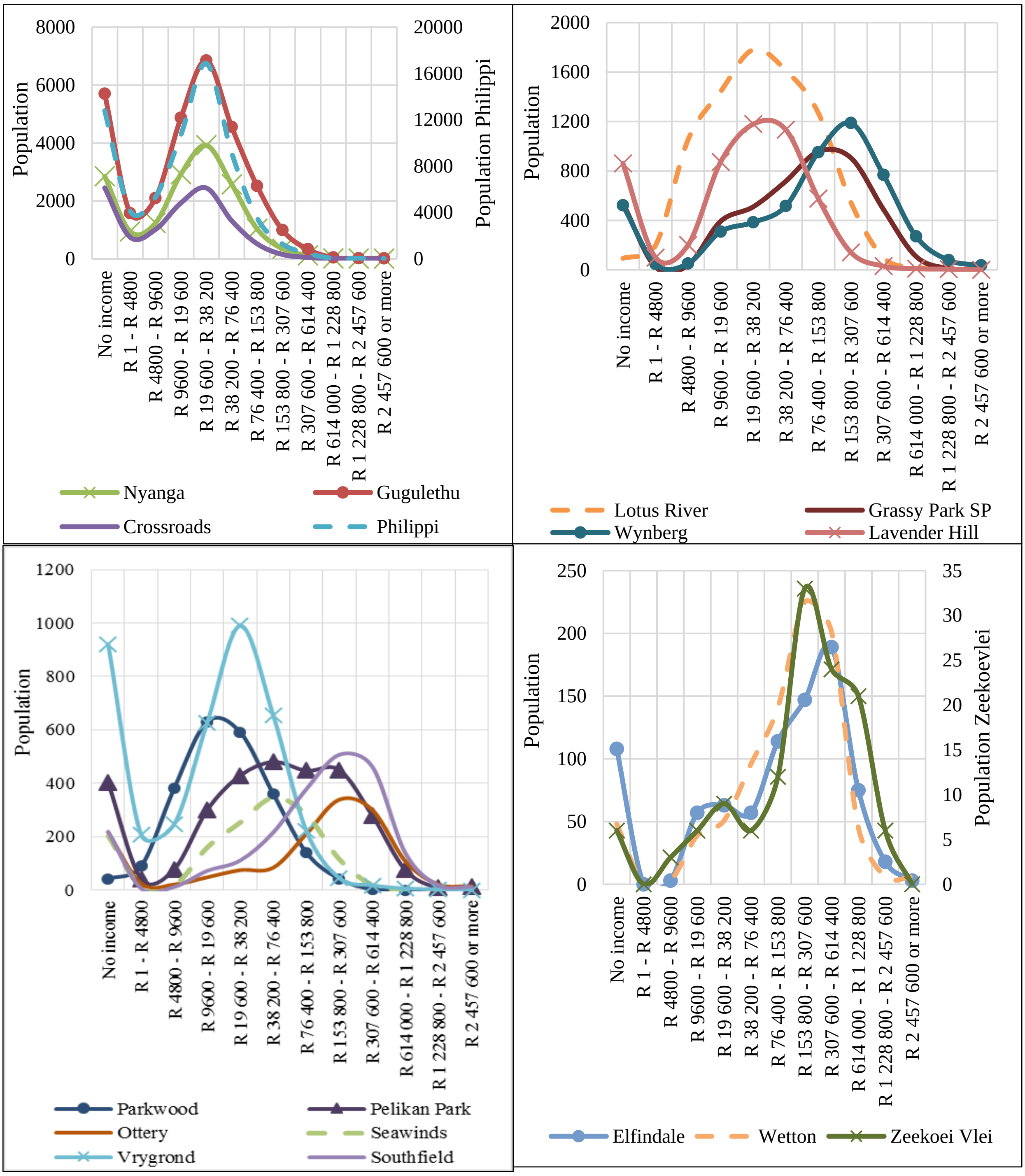




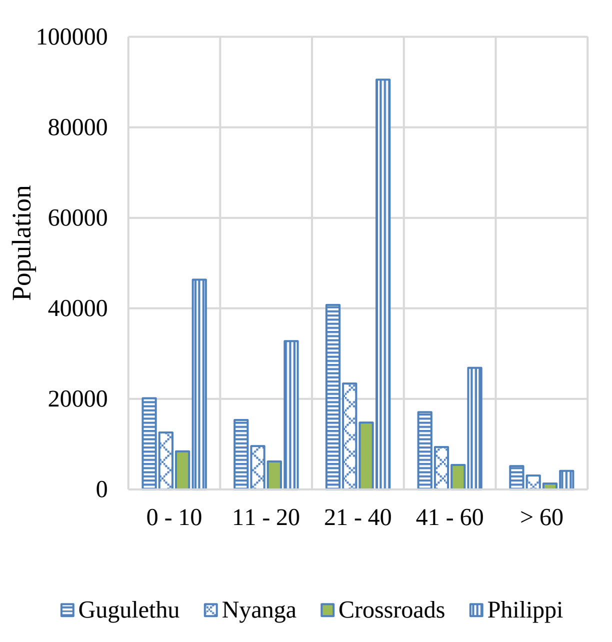
APPENDIX 6E: HOUSEHOLD ALTERNATIVE WATER SOURCE PER SUBURB (STATSSA, 2011)

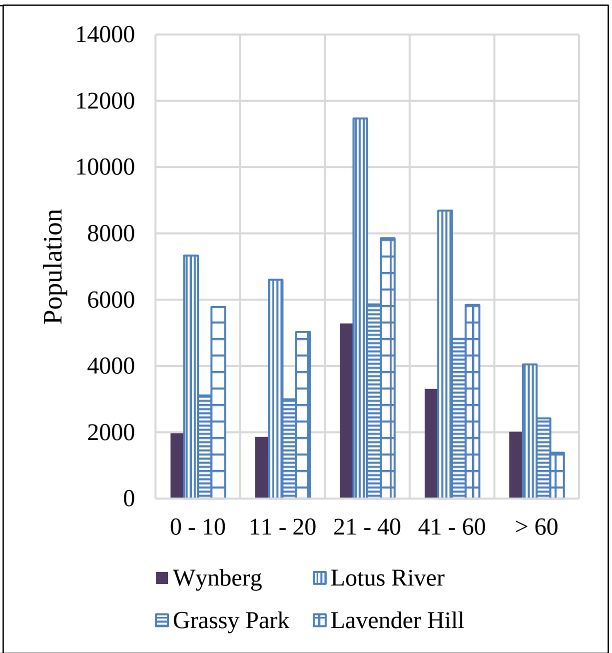


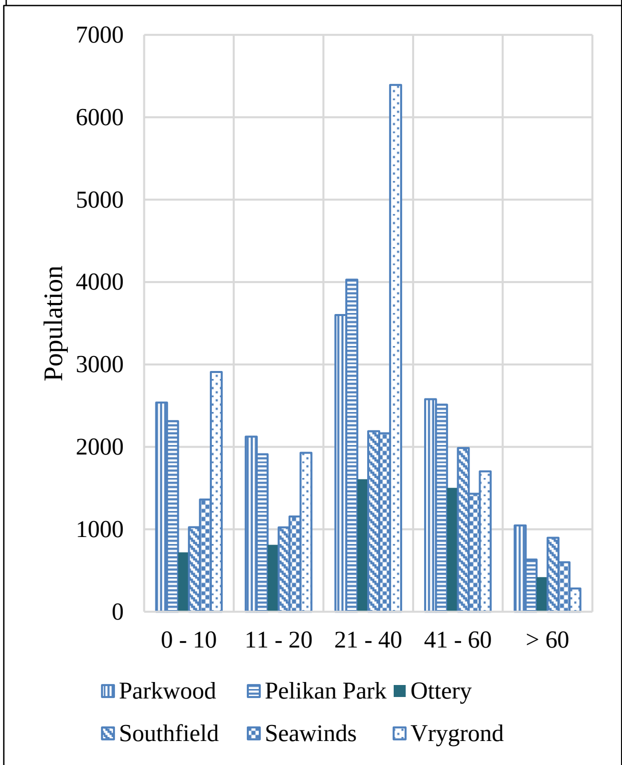
APPENDIX 6F: HOUSEHOLD INCOME GROUPS PER SUBURB IN THE STUDY AREA (STATSSA, 2011)

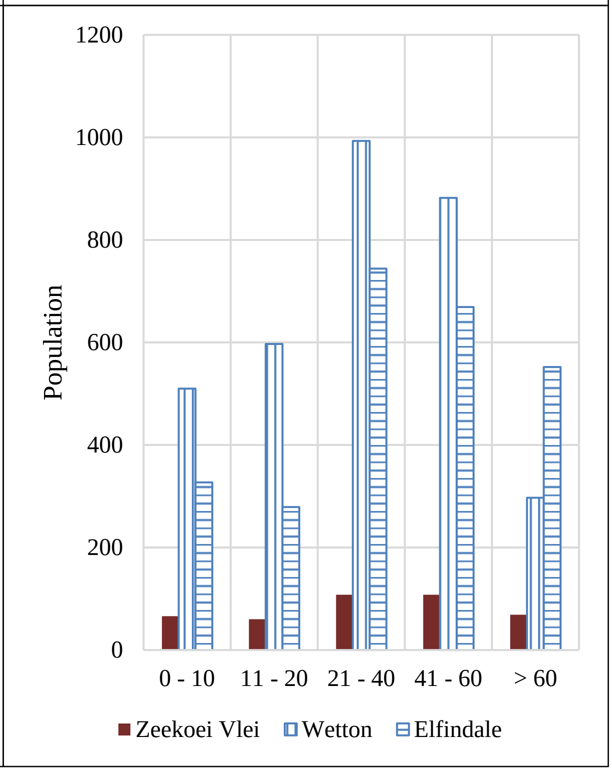


APPENDIX 6G: HOUSEHOLD AGE GROUPS PER SUBURB IN THE STUDY AREA (STATSSA, 2011)

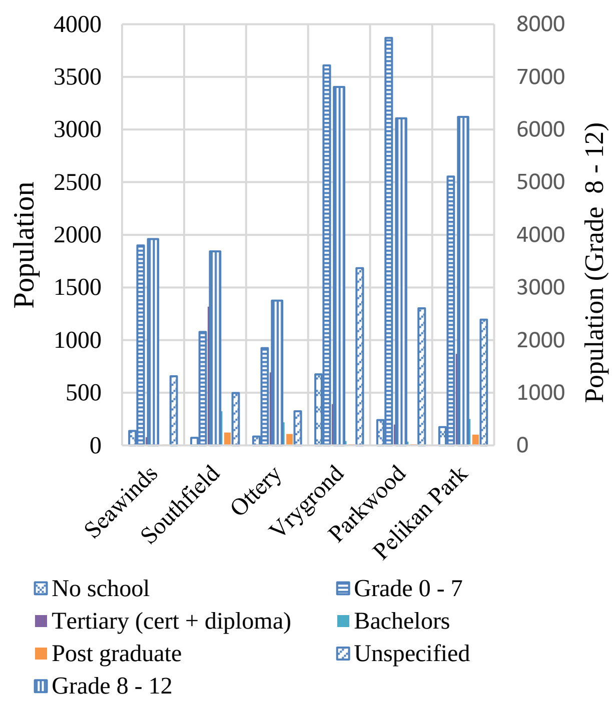


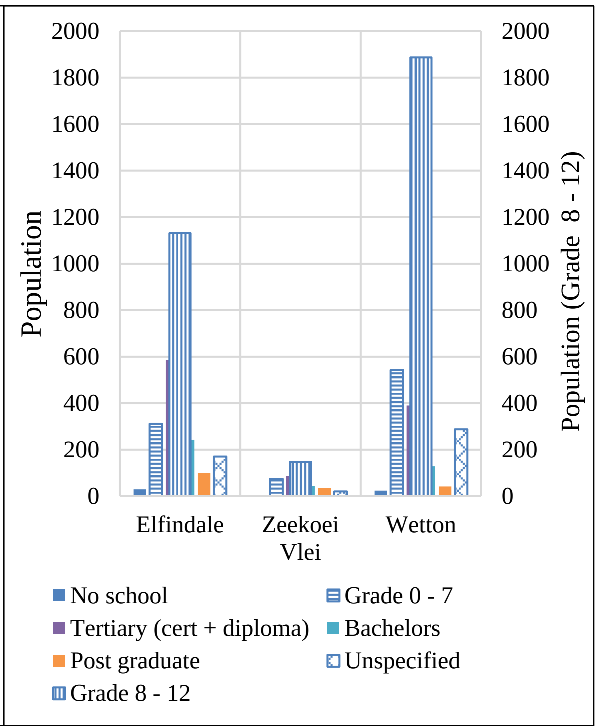


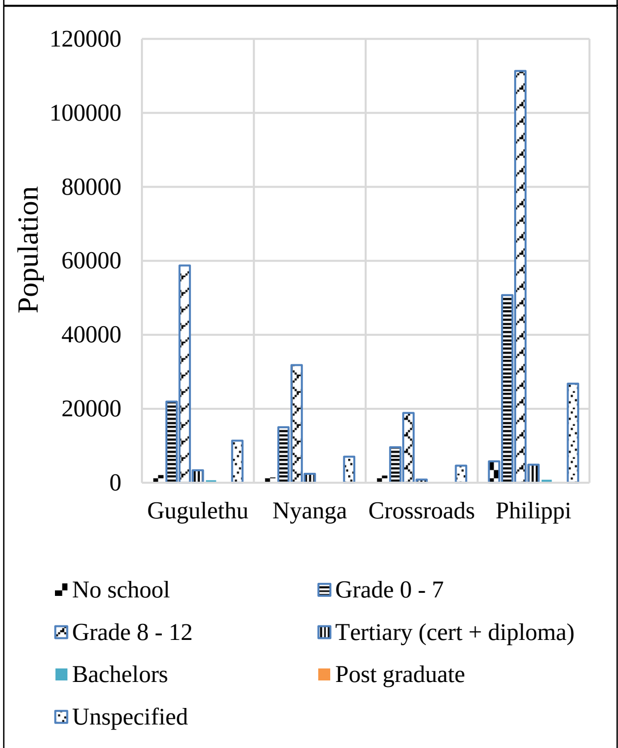


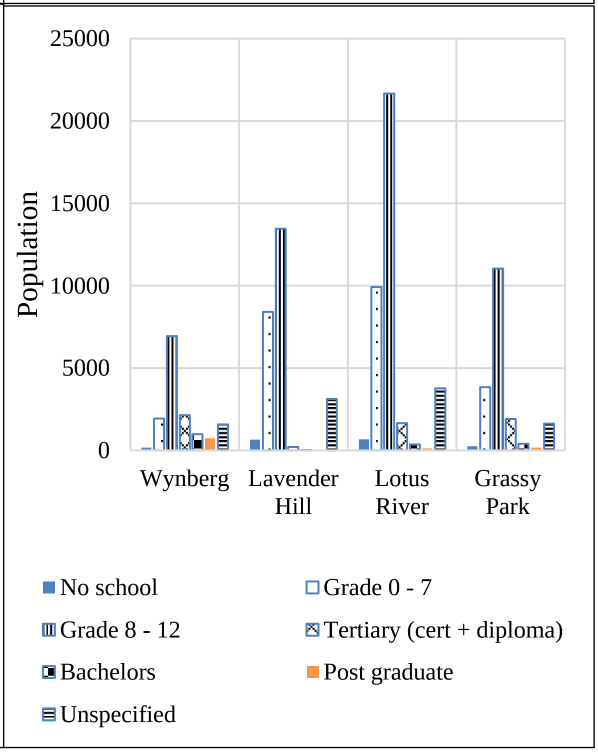


APPENDIX 6H: LEVEL OF EDUCATION PER SUBURB (STATSSA, 2011)









APPENDIX 7: INNOVATIONS IN NATURE-BASED STORMWATER MANAGEMENT

The 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 may be abundant, but in a form that is not easily usable by human, for example sea water and frozen water at the poles (IWMI, 2000). Seawater makes up 97% of the total global water. A further 2.25% is trapped in glaciers and ice. This leaves 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 are 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 *w*ater-sensitive urban design (WSUD) in Australia, low-impact development (LID) in the USA 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 WSUD, LID and SuDS, are linked to a philosophy where a holistic water cycle management approach aims to minimise the net outflow of stormwater from a given catchment (Fletcher et al., 2014). The principles common to these environmentally sensitive approaches are summarised in Table A-1.

Table A-1: 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 or 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 environmentally sensitive approaches compared to other water balances are shown in Figure A-1.

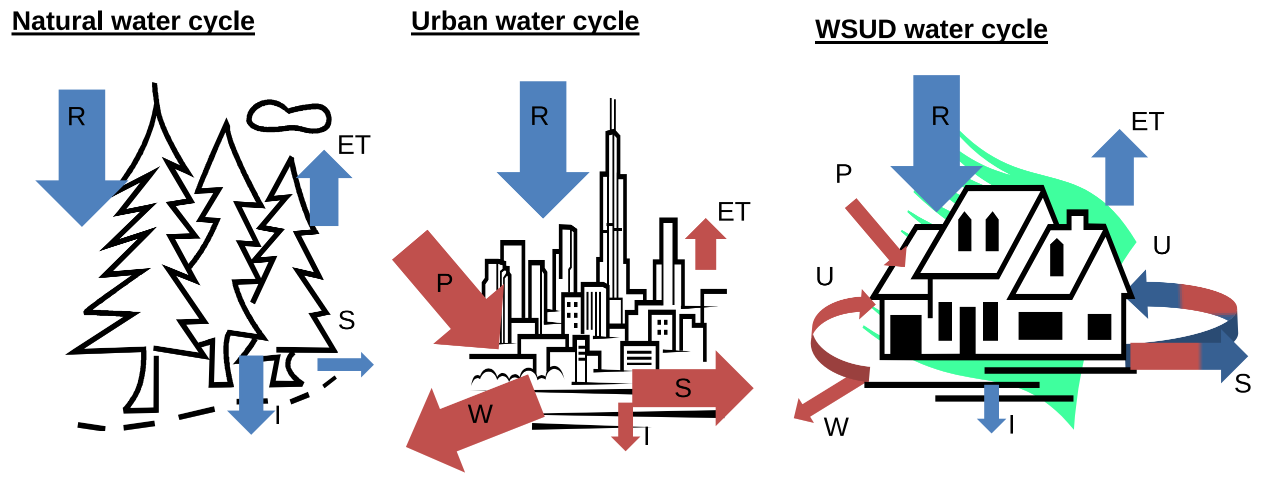


Figure A-1: The concept of water-sensitive urban design (Hoban and Wong, 2006)

For the seasonal availability of stormwater in regions such as Cape Town, balancing storage is required to enhance the reliability of the water supply from the option. There are various storage options for the stormwater harvesting systems discussed in the 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 the 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 the literature is briefly given 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 temporarily store rainwater that runs off roofs or properties (Begum et al., 2008; Hatt et al., 2006) 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 are 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 wetlands, can provide at least four types of benefits: the management of water quantity, the improvement of water quality, the provision of an 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). The 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 on other activities such as 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 (Armitage et al., 2013; Woods-Ballard et al., 2007). Detention basins are typical in conventional stormwater management due to the available storage capacity (no permanent pool of water) as the focus of flood control (Armitage et al., 2013). Detention ponds are typically a vast expanse of depressions on land. Some detention pond designs include 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 the aesthetic value and affluence of the area (Woods-Ballard et al., 2007). 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 an 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 that provides some level of stormwater quality improvement in addition to peak flow attenuation from storm events to mitigate flood risk downstream of the ponds (Armitage et al., 2013; Debo and Reese, 2003; Mays 2001; Woods-Ballard et al., 2007). 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 (Armitage et al., 2013; Stahre, 2006). Retention ponds are not common as they provide limited flood control measures, 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% (Dinovo, 1995; USEPA, 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 by litter and solid waste, potential breeding ground for mosquitos 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).

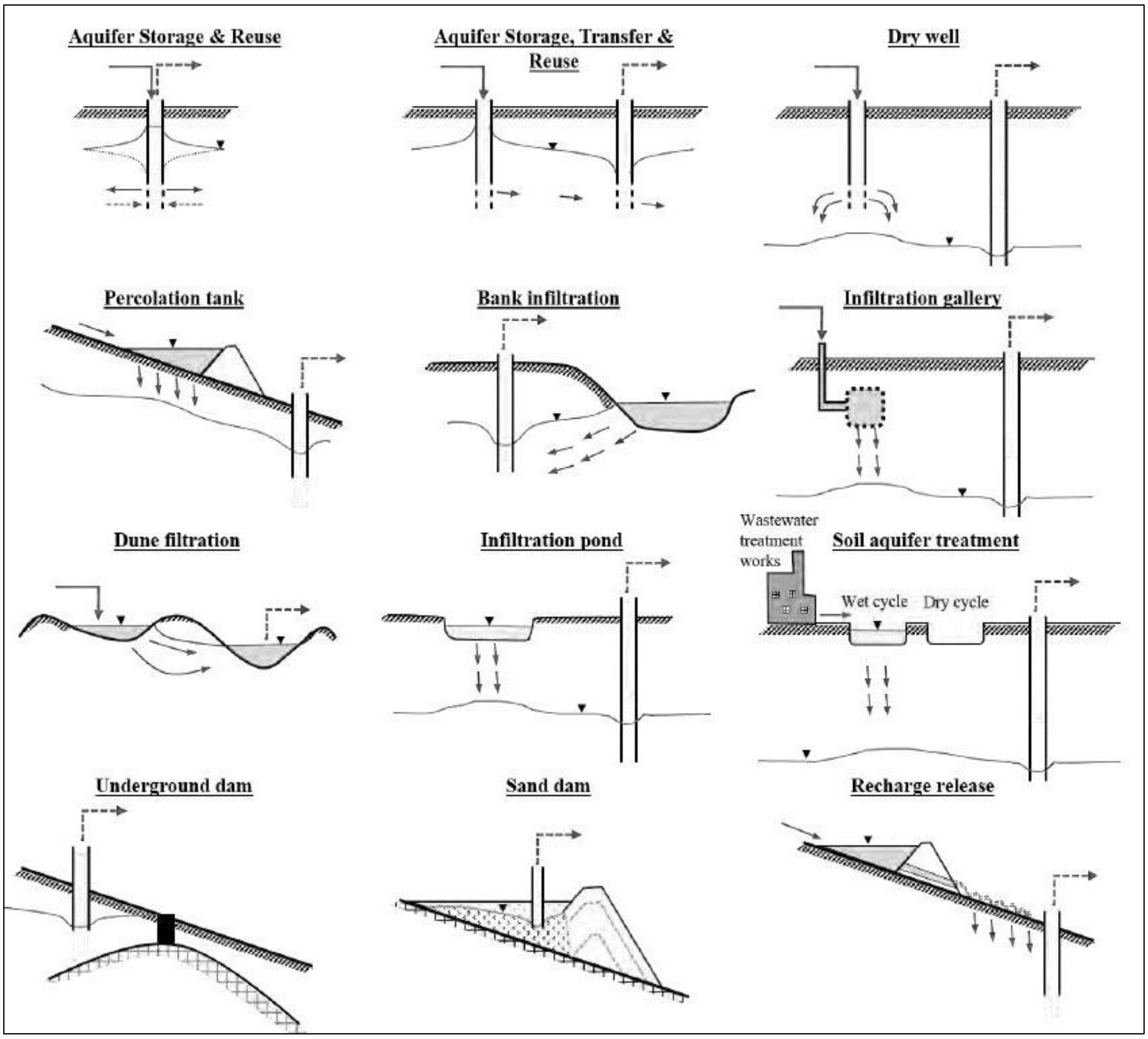
A constructed wetland is typically characterised by marshy, shallow water that is partially or completely covered in aquatic vegetation and provides more stormwater quality improvement than peak flow attenuations from storm events to mitigate flood risk downstream of the ponds (Armitage et al., 2013; Woods-Ballard et al., 2007). 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 and their 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 measures, 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 nutrients and the removal of some pathogens (Armitage et al., 2013; Field and Sullivan, 2003; Parkinson and Mark, 2005).

APPENDIX 8: STORMWATER HARVESTING FROM GROUNDWATER STORAGE

APPENDIX 8A: OVERVIEW OF STORMWATER HARVESTING FROM GROUNDWATER STORAGE

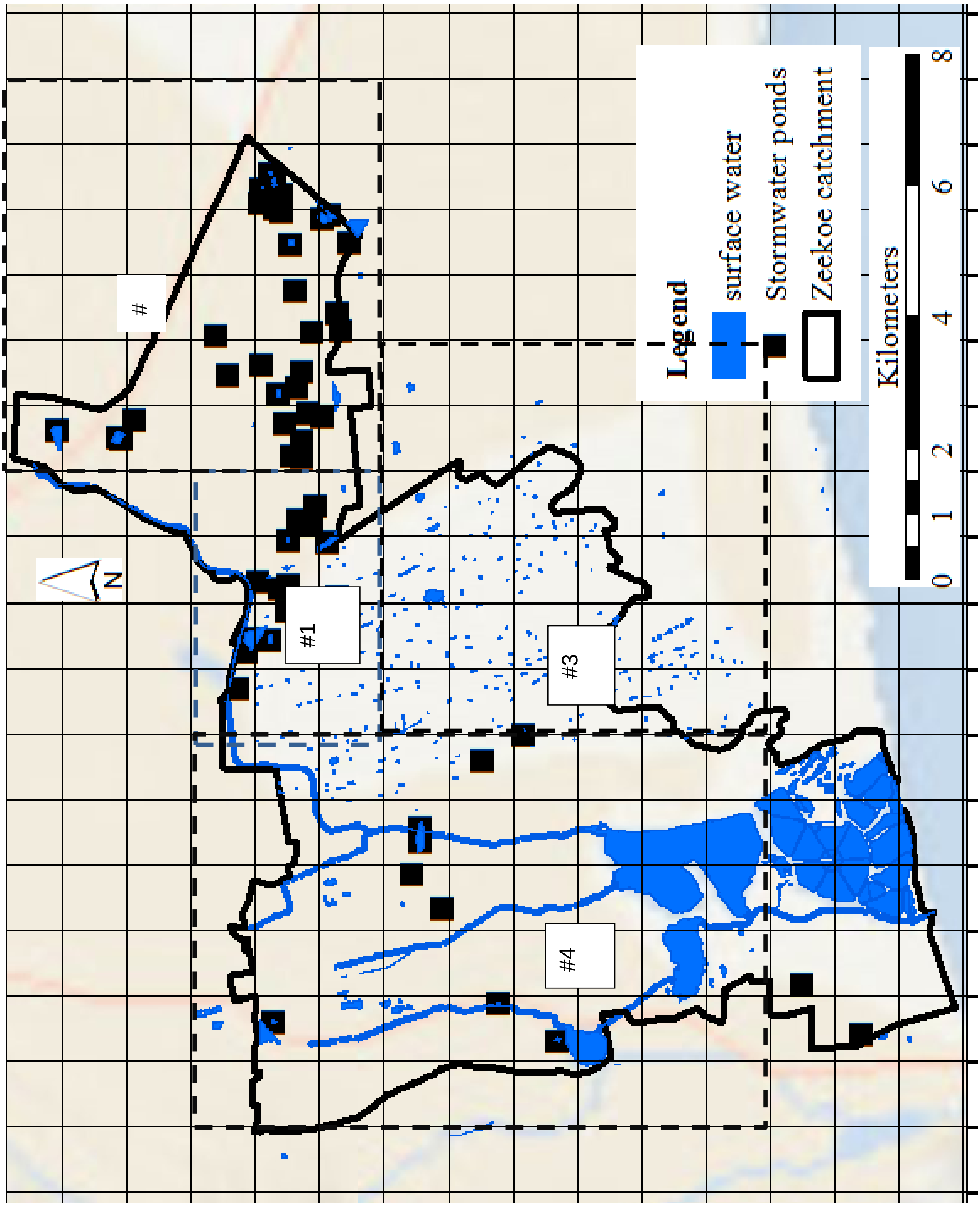
Managed aquifer recharge and recovery (MAR&R) is a process where surface water, e.g. stormwater or wastewater, is temporarily stored in excavated depressions in the earth's surface with a specific configuration to allow for the recharging of groundwater aquifers for future use or for environmental benefits (Dillon et al., 2010; Fisher-Jeffes, 2015; Wu et al., 2012).



The recharge of the groundwater aquifer can be accomplished through the direct injection of surface water to underground dams through various MAR&R approaches as shown (Dillon et al., 2010; Fisher-Jeffes, 2015; Wu et al., 2012). The main aim of the transfer of surface water to groundwater aquifers is to make use of the 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., 2010; 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. the 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 8B: A DISCRETISED SECTION OF ZEEKOE CATCHMENT FOR GROUNDWATER MODELLING IN MATLAB (AFTER CCT, 2015)



APPENDIX 8C: 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 8C: 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);
```

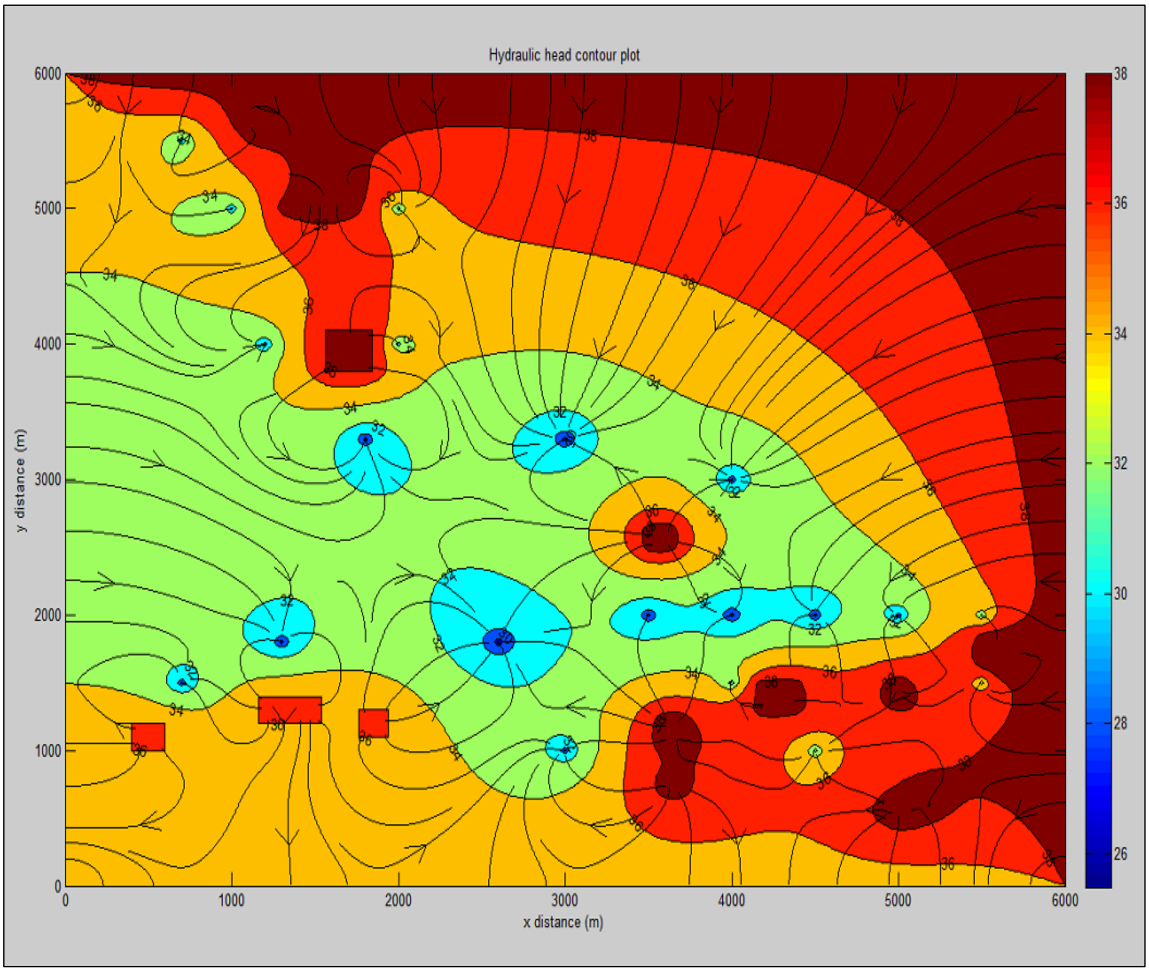
```
dt=T/nt;
% calculate velocity from head written by flow code
[vx,vy]=velocity(dx,dy,nx,ny, porosity);
% solve for concentration and output solution at isol time-steps
[csol] = gwtrans2d_solve(Lx, Ly, bctype, bcval, T, nx, ny, nt, isol, vx, vy, ...
   alphaL, alphaT, Dm, R, lambda);
% plot solution 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
```

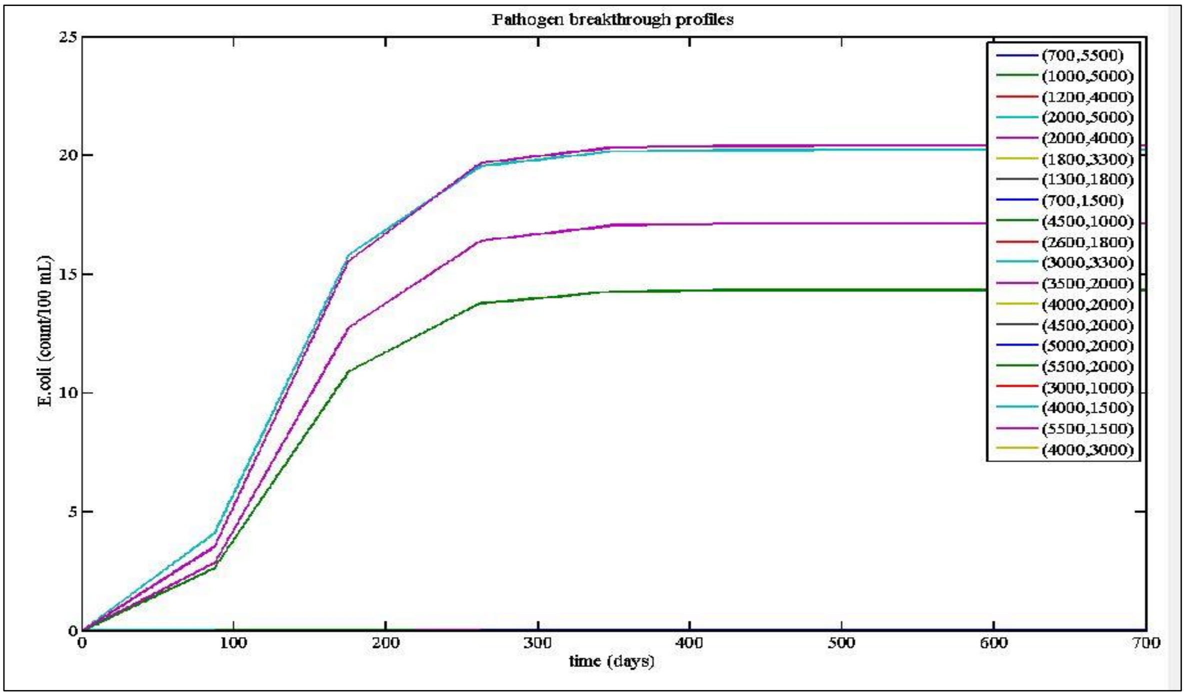
```
        nodal_dirich.type(i,j)=1;
        nodal_dirich.val(i,j)=hvalue;
    end
end
return

% 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)');
```

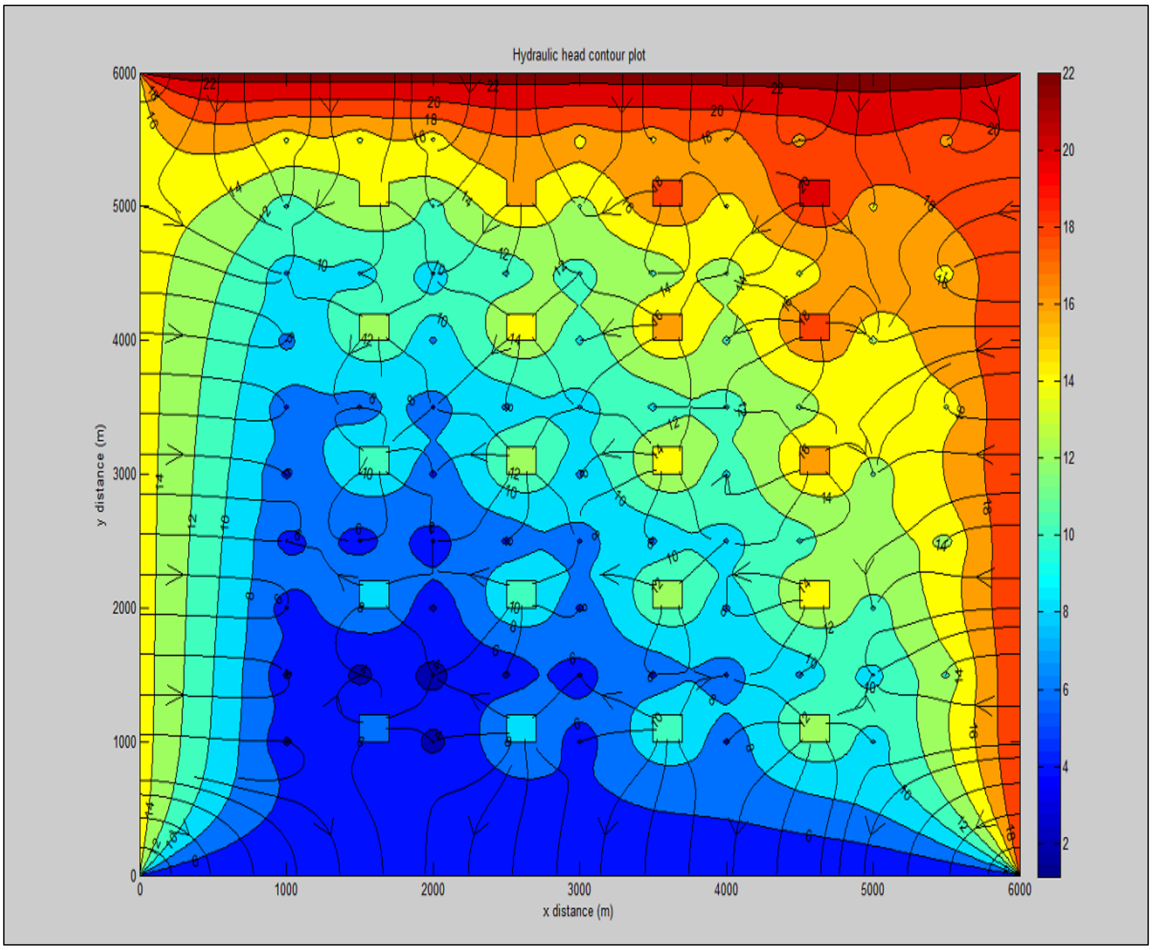
```
ylabel('concentration (mg/L)');

title(['Concentration profile at (',num2str(xp),',',num2str(yp),') ']);

end
```

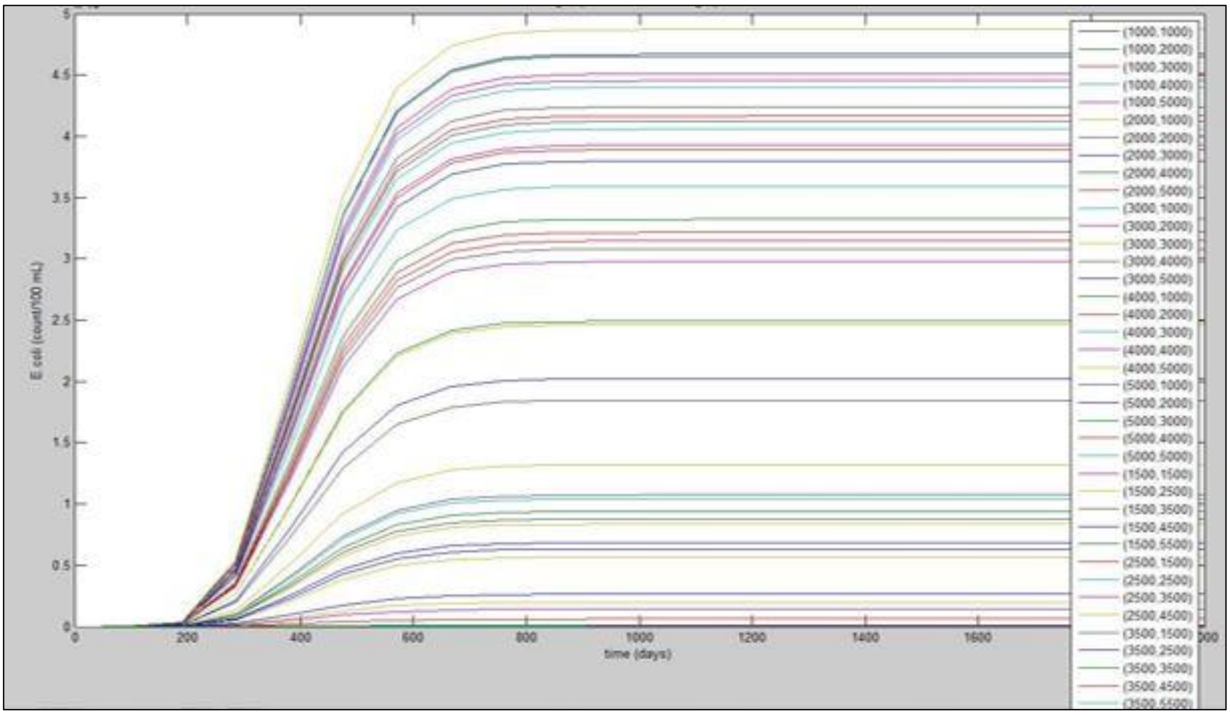
APPENDIX 8C: LOCATION OF BOREHOLES AND BREAKTHROUGH CURVES FOR SECTION #2



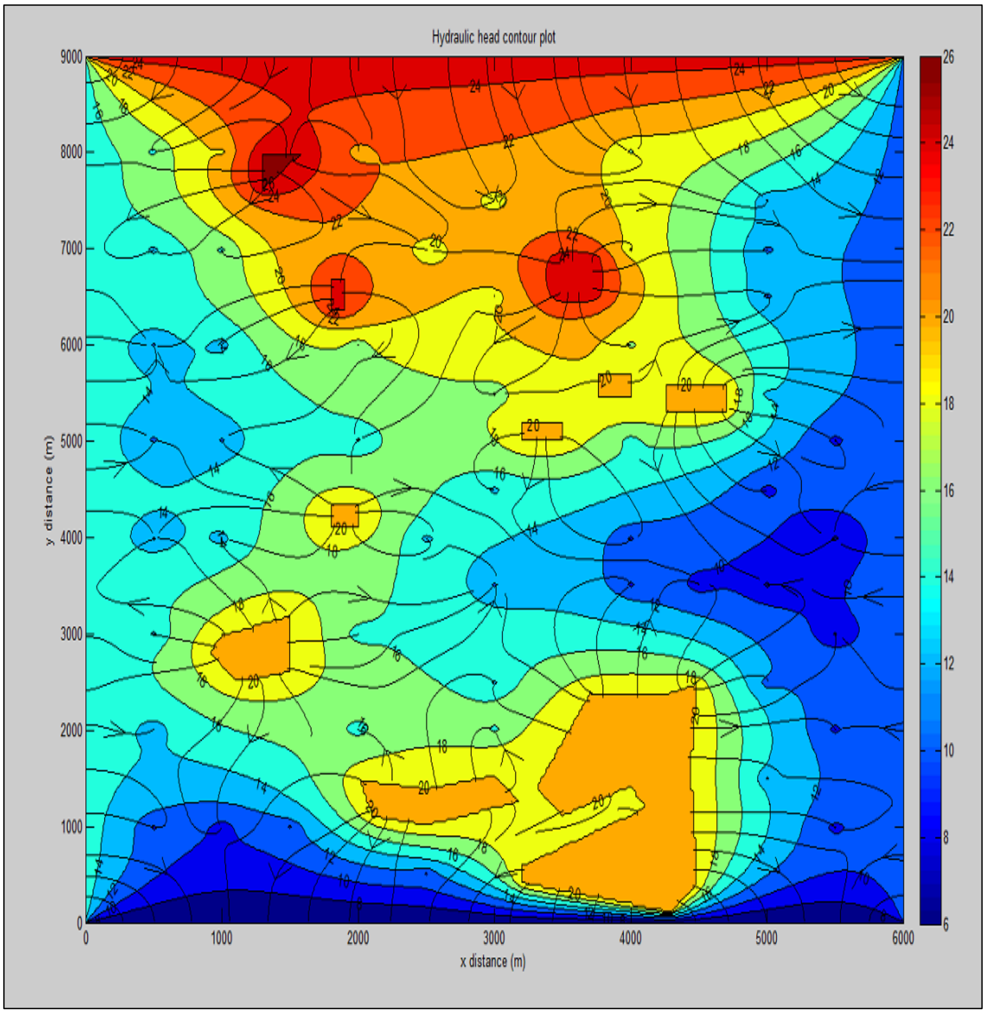


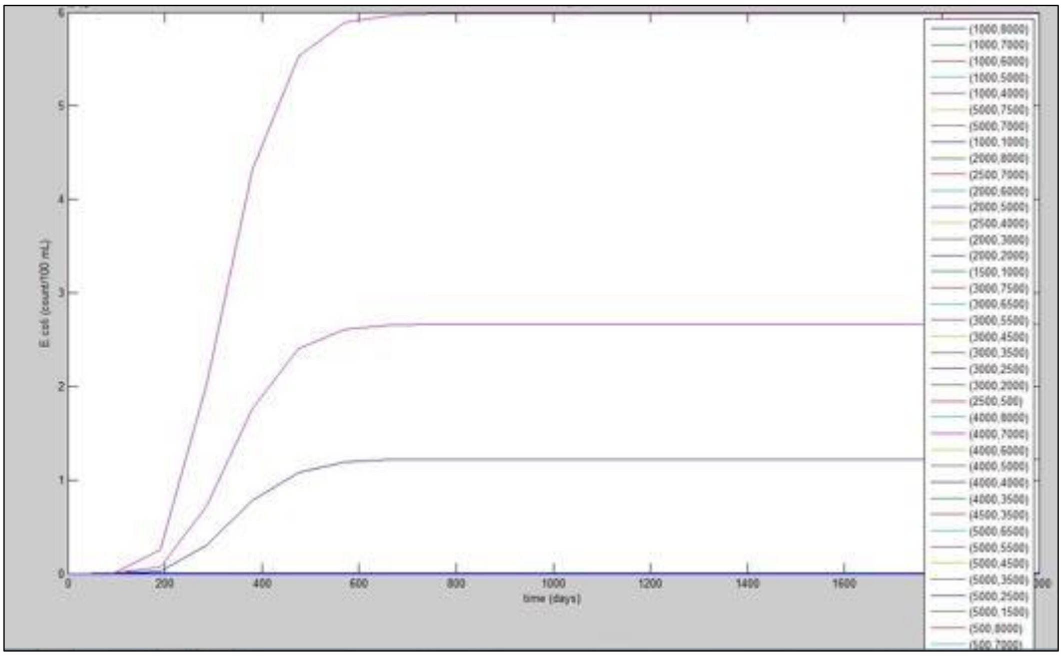
APPENDIX 8D: LOCATION OF BOREHOLES AND BREAKTHROUGH CURVES FOR SECTION #3





APPENDIX 8E: LOCATION OF BOREHOLES AND BREAKTHROUGH CURVES FOR SECTION #4

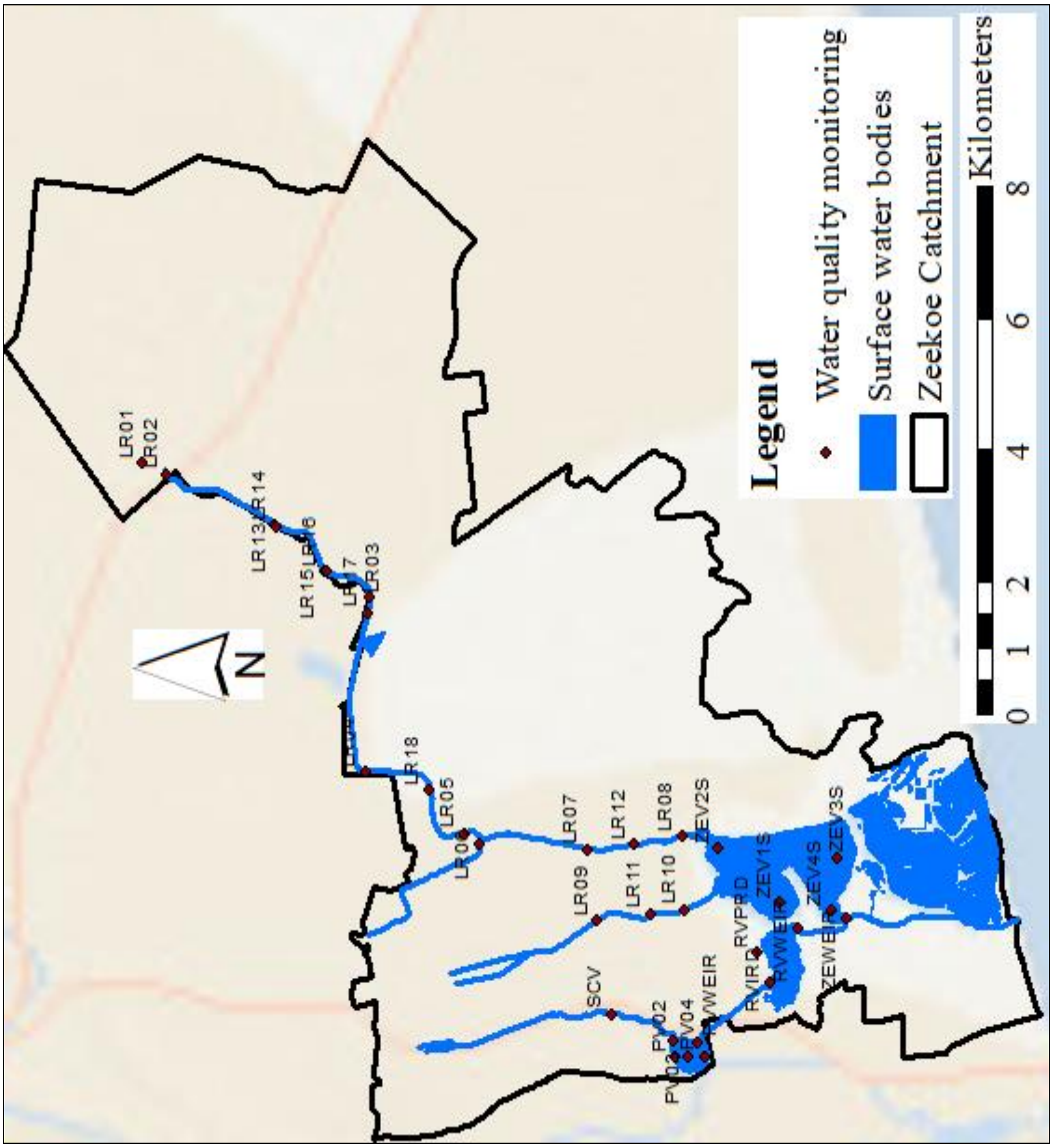




APPENDIX 9: WATER QUALITY MONITORING AND PARAMETERS MEASURED IN THE STUDY AREA

Water quality monitoring is undertaken in the study area for various parameters on a monthly basis at several locations.

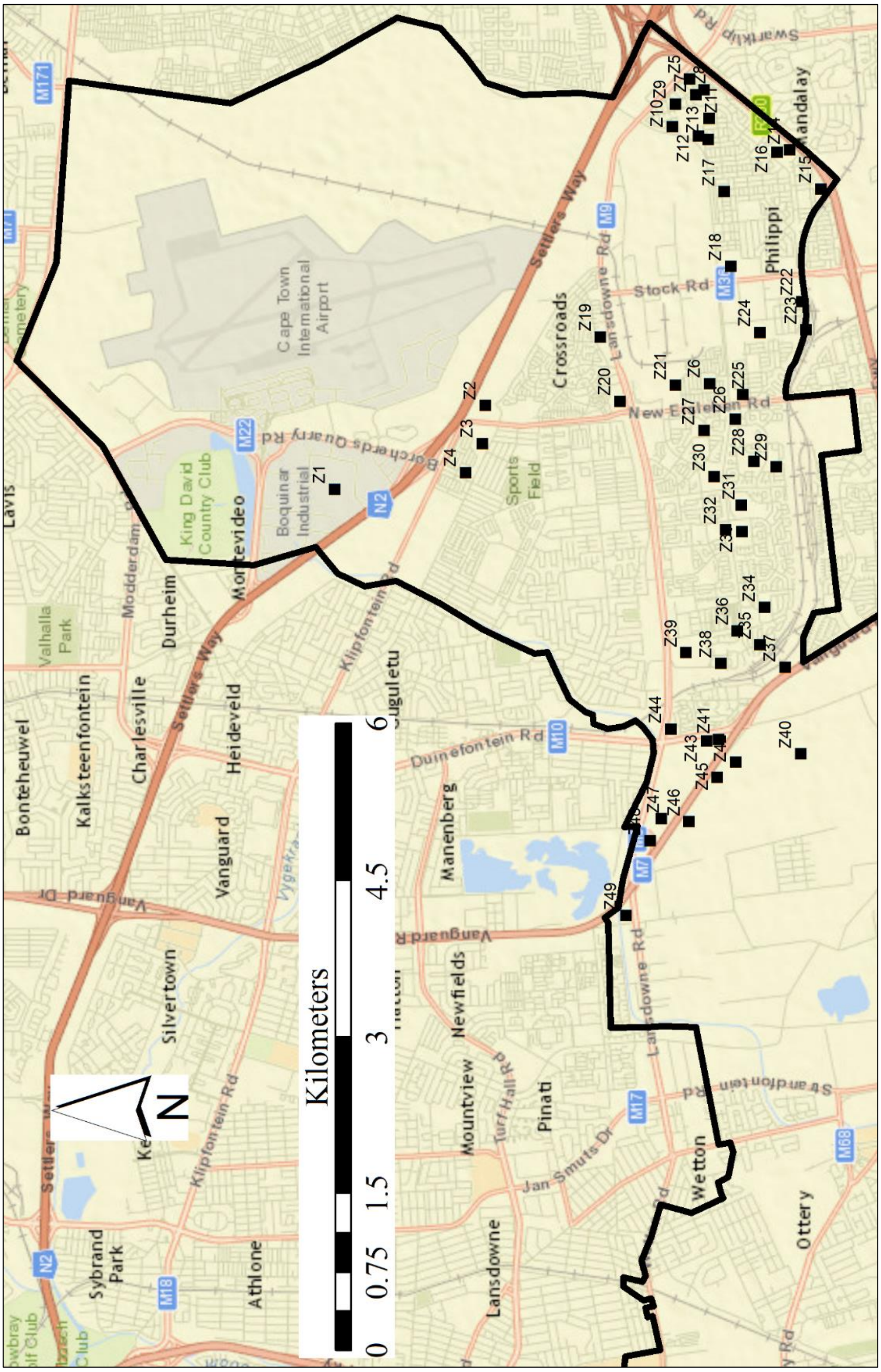
APPENDIX 9A: WATER QUALITY MONITORING STATIONS IN THE STUDY AREA

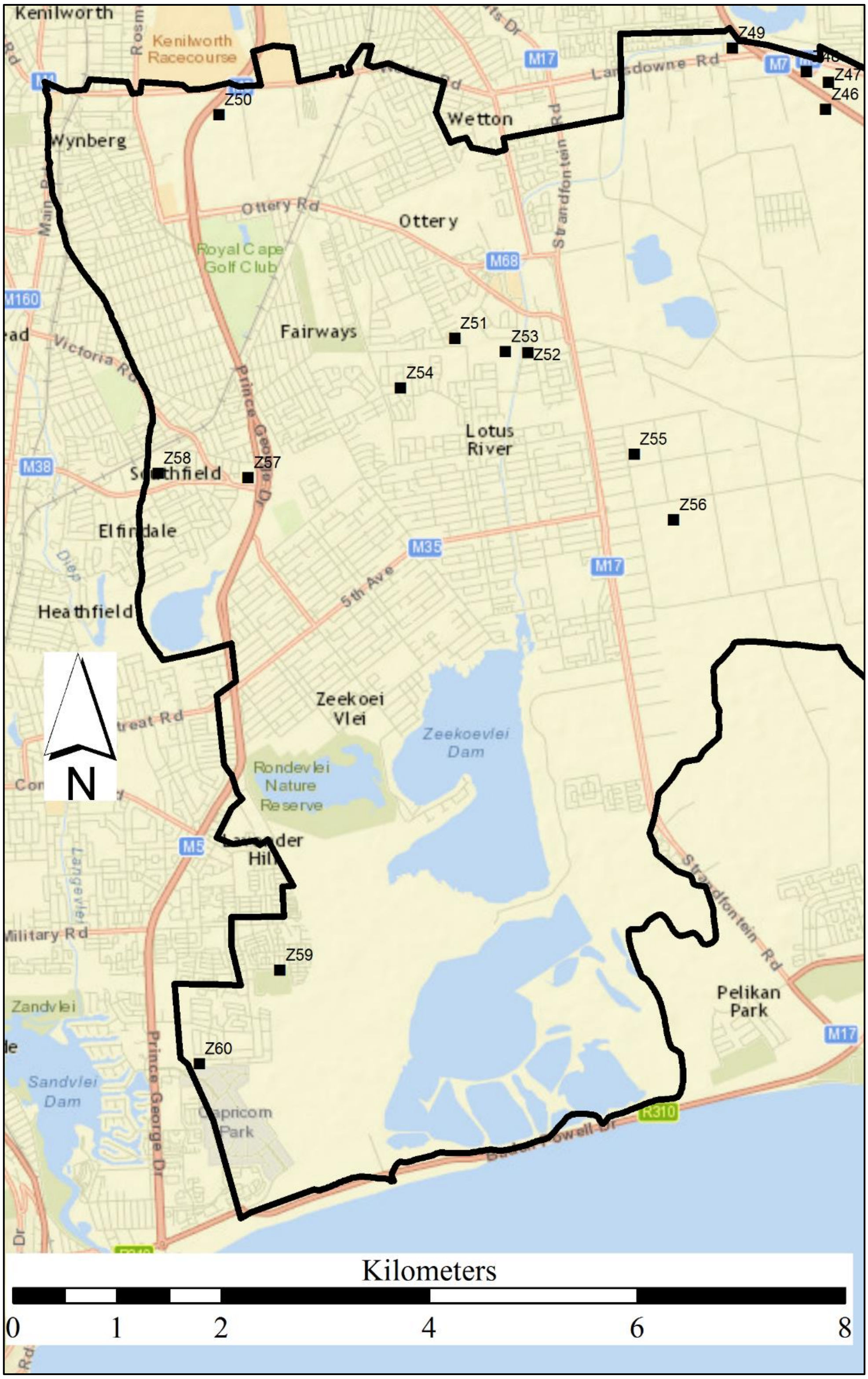


APPENDIX 9B: MONITORED WATER QUALITY PARAMETERS

	Monitoring	Description	Code
Big Lotus	Chemical or bacteriological	Lotus River on Airport approach road opposite Borcherd's Quarry final effluent ponds	LR01
		Lotus River on Settler's Way (N2) about 500 m 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 or bacteriological	Lotus River at corner of Duinefontein and Lansdowne roads	LR03
		Lotus River at Lansdowne Road	LR04
	Bacteriological	Lotus River at Springfield Road Turfhill Estate	LR18
	Chemical or 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 or 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 approximately 200 m from the weir	ZEV4S
Little Lotus	Chemical or 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 or algalogical	Princessvlei – vlei inlet	PV01
		Princessvlei – centre	PV03
		Princessvlei – north	PV02
		Princessvlei – south	PV04
		Princessvlei near outlet weir	PVWEIR
	Chemical or bacteriological	Southfield Canal at Victoria Road	SCV
Rondevlei	Chemical or bacteriological	Italian Road canal leading to Rondevlei	RVIRD
	Chemical, bacteriological or algalogical	Rondevlei Weir	RVWEIR
	Chemical or bacteriological	Perth Road canal leading to Rondevlei	RVPRD
Zeekoevlei	Chemical, bacteriological or algalogical	Vlei sample at Zeekoevlei – inlet	ZEV2S
		Vlei sample at Zeekoevlei – centre	ZEV1S
		Vlei sample at Zeekoevlei weir – outlet	ZEWEIR

APPENDIX 10: LOCATION OF STORMWATER PONDS IN THE STUDY AREA




Kenilworth
Kenilworth Racecourse
Wynberg
Wetton
Lansdowne Rd
Ottery Rd
Ottery
Royal Cape Golf Club
Fairways
Victoria Rd
Lotus River
Southfield
Elfindale
Heathfield
5th Ave
Zeekoei Vlei
Zeekoevlei Dam
Rondevlei Nature Reserve
Military Rd
Zandvlei
Sandvlei Dam
Capricorn Park
Prince George Dr
Strandfontein Rd
Pelikan Park
Baden Powell Dr
M17
M68
M35
M38
M5
M160
M7
R310
Z46
Z47
Z49
Z50
Z51
Z52
Z53
Z54
Z55
Z56
Z57
Z58
Z59
Z60
N
Kilometers
0
1
2
4
6
8

APPENDIX 10A: 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"
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"

ID	Pond type	Road	Suburb	Erf No.	Latitude	Longitude
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"
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"

ID	Pond type	Road	Suburb	Erf No.	Latitude	Longitude
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"
Z61	Detention	Madeira Drive	Muizenberg	00-160998	-34° 5' 36.56"	18° 29' 0.69"