

CIV5000Z: DISSERTATION FOR THE DEGREE OF A
MASTER OF SCIENCE IN CIVIL ENGINEERING

**Managing nutrient flows into the Zandvlei Estuary, Cape Town
using Sustainable Drainage Systems (SuDS)**



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Dissertation submitted in partial fulfilment of the requirements for the degree of Master of
Science in Civil Engineering (MSc (Eng))

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Abstract

The Zandvlei estuary is the only functional estuary on the False Bay coastline which forms part of the southeast boundary of the City of Cape Town (CCT) on the southwest coast of South Africa. It provides an important habitat for fledgling fish species in the area and is a popular recreational site for birdwatchers, sailors and conservation-enthusiasts.

The estuary has experienced water quality problems for many years. It drains three upstream sub-catchments which are largely comprised of conventional drainage systems which carry stormwater pollutants into the watercourses that then drain into the estuary. Of the many pollutants that flow into the estuary, elevated nutrient levels (specifically phosphorus and nitrogen) are a major concern because they can lead to eutrophication and excessive plant growth. In recent years the estuary has experienced algal blooms resulting in adverse effects on local plants, and animal and human life. Sustainable Drainage Systems (SuDS) are holistic drainage practices that address, amongst other things, water quality issues like those experienced by the Zandvlei estuary by treating the stormwater. The aim of this study was to identify areas of raised nutrient concentrations and model various SuDS measures to improve the water quality of runoff.

Surface water sampling in the catchment and a study of historical water quality data was undertaken to identify areas with higher nutrient concentrations. Phosphorus concentrations, in the form of SRP, was found to be above the eutrophic threshold in most samples, while nitrogen concentrations, in the form of TIN, were found to mostly be beneath the eutrophic threshold. The locations of higher nutrient concentrations coincided with suspected sources of raised nutrients based on a desktop study of the catchment that identified agricultural areas, golf courses that use treated effluent for irrigation, and commercial/industrial areas. Water level sensors were developed and placed in each sub-catchment to compare nutrient concentrations during storms to stream flow during storms.

A hydrological and hydraulic model was built representing two of the river systems feeding into the Zandvlei estuary – the Keyzers and Westlake River sub-catchments. The PCSWMM software package developed by CHI, which uses the SWMM engine developed by USEPA, was employed. Data regarding historical rainfall, land use, stream flow, surface cover, soil types, geology and historical temperatures were used to construct the model. The model was calibrated against historical stream flow data obtained from CCT. Various SuDS measures were modelled and grouped into three scenarios: Scenario 1 modelled local controls targeting the suspected source areas, Scenario 2 involved modelling large-scale regional controls, and Scenario 3 was a combination of the first two. The local controls modelled were swales and bioretention areas. The regional controls modelled were wetlands, primarily because the study area already contains wetland areas that are currently disconnected from the river systems. Wetlands were modelled with upper and lower limit treatment equations.

The simulated wetlands were able to reduce the SRP and TIN concentrations to below the eutrophic threshold (below 0.25 mg/ℓ and 2.5 mg/ℓ respectively) for three years out of the 8-year

simulation period. The various scenarios achieved pollutant reductions ranging from approximately 25-80%.

The CCT requires a TP reduction of 45% and a TSS reduction 80%. For TP, this was achieved throughout the simulation period when wetlands are functioning efficiently (i.e. the upper treatment limit); while for TSS, this requirement was only achieved in some years when the wetlands are functioning efficiently.

Scenario 1 showed the least pollutant removal overall, while Scenario 2 and 3 showed similar pollutant removal and flow reduction, suggesting that the effect of the proposed local controls is insignificant compared to that of the large-scale regional controls.

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Acronyms, abbreviations and chemical formulae

ARC	Agricultural Research Council
CHI	Computational Hydraulics International
CCT	City of Cape Town Metropolitan Municipality
DEM	Digital Elevation Model
<i>E. coli</i>	<i>Escherichia coli</i>
EC	Electrical Conductivity
EMC	Event Mean Concentration
GIS	Geographical Information Systems
HRT	Hydraulic Residence Time
LID	Low-Impact Development
MAP	Mean Annual Precipitation
MS	Microsoft
NH ₃	Ammonia
NH ₄ ⁺	Ammonium
NO ₂	Nitrite
NO ₃ ⁻	Nitrate
NSE	Nash-Sutcliffe Efficiency
PCSWMM	Personal Computer Stormwater Management Model
PO ₄ ³⁻	Orthophosphate
R ²	Coefficient of determination
RI	Recurrence Interval
SRP	Soluble Reactive Phosphate
SRTC	Sensitivity Radio Tuning Calibration
SuDS	Sustainable Drainage Systems
TAN	Total Ammonia Nitrogen
TIN	Total Inorganic Nitrogen
TN	Total Nitrogen

TO _x N	Total Oxidised Nitrogen
TP	Total Phosphorus
TSS	Total Suspended Solids
WQV	Water Quality Volume
ZCF	Zandvlei Catchment Forum
ZENR	Zandvlei Estuary Nature Reserve
ZPAAC	Zandvlei Protected Areas Advisory Committee

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1. Introduction

1.1 Background

Estuaries are bodies of water typically found at the juncture of rivers and the sea. As such, they have a unique mix of fresh and saline water that generally bestows great biodiversity value upon them (DEA&DP, 2018). The Zandvlei estuary – a shallow waterbody on the fringe of the Muizenberg coastline in the City of Cape Town – is no exception. Like most estuaries, Zandvlei has unique characteristics that render it essential in contributing to local ecological stability, thus affecting human, plant, and animal life.

A popular recreational site, the estuary is a haven for aquatic life, fledgling fish species and local sailors alike. However, the estuary has faced various challenges, including the effects of sedimentation, excess nutrient loads, and various other pollutants including sewage. Despite various mitigation efforts, these problems persist – critically affecting the water quality of the estuary and having subsequent social and ecological effects.

1.2 Research problem

Besides the area surrounding the estuary, three river systems feed into Zandvlei. These sub-catchments are drained via conventional drainage systems where stormwater is largely collected by catchpits alongside the roads and transported via concrete pipelines to the nearest watercourse – which eventually flow into the estuary. Stormwater collected in this way carries many pollutants into the receiving water courses. Although there are other sources of pollution (point sources, overflowing sewers, etc) this study was limited to stormwater runoff pollution.

Of the many stormwater pollutants that flow into the estuary, elevated nutrient levels (specifically phosphates and inorganic nitrogen) in the rivers are a major concern because they can lead to eutrophication (CCT, 2010). Eutrophication is characterized by excessive plant growth, which in severe cases may lead to algal blooms – resulting in adverse effects on local plant, animal and human life (Edokpayi *et al.*, 2017). In recent years the estuary has experienced algal blooms resulting in many fish and bird deaths. Although various studies on the catchment have commented on potential sources and causes of raised nutrient levels, more information is needed on how high the nutrient concentrations are. The locations of elevated nutrient levels also need to be known, so that these areas can be targeted for interventions.

1.3 Objectives

The objectives of this research project were two-fold:

- Firstly, to investigate the concentrations and origins of selected nutrients that are being transported by the Keyzers and Westlake Rivers into the Zandvlei estuary.
- Secondly, to investigate the effect that Sustainable Drainage Systems (SuDS) interventions might have on water quality in the study area (the Keyzers and Westlake sub-catchments) through the construction and use of a coupled hydrological, hydraulic and water quality model.

1.4 Method overview

The method employed in this study comprised the following steps:

- A brief desktop study on the Zandvlei catchment and the challenges it faces;
- Water quality sampling to briefly identify areas with higher pollutant concentrations;
- Data gathering for the construction of the hydrological model;
- The creation of a model that represents the current state of the catchment including estimated wash-off rates for selected pollutants; and
- The simulation of a pre-development scenario and various SuDS interventions, comparing their relative pollutant removal capabilities.

1.5 Dissertation structure

This document presents the outcome of this research project in the following chapters (supplementary information can be found in the appendices):

- Chapter 1 (this one) introduces the research problem, lists the research objectives and briefly describes the method employed.
- Chapter 2 presents a brief review on literature on the impact of development on catchments, the threat of eutrophication and relevant South African guidelines. It continues with a brief introduction to SuDS and the modelling thereof.
- Chapter 3 describes the Zandvlei catchment in terms of its river systems, the estuary that drains them, and some of its challenges.
- Chapter 4 explains the methods employed in the various components of the study.

- Chapter 5 discusses the water quality in the study area, based on historical data as well as water quality sampling undertaken as part of this study.
- Chapter 6 describes the various scenarios created in the hydrological model.
- Chapter 7 presents the findings of the modelling exercise.
- Chapter 8 draws conclusions based on the findings and offers some recommendations.

2. Literature Review

2.1 The impact of development on catchments

Urban development has resulted in increased agricultural modification, industrialization and urbanization resulting in changes to the drainage characteristics of catchments (Wu *et al.*, 2018). The altering of natural rivers to accommodate these changes – through widening, canalisation, reshaping or burial – and the increased hardening of surfaces are seen as some of the most drastic influences on river systems since the middle of the twentieth century with major consequences for the quantity and quality of surface waterbodies (Wu *et al.*, 2018).

2.1.1 The effect of urbanisation on runoff quantity

Rising rates of urbanisation have resulted in land increasingly being converted into residential, commercial and industrial areas. According to Armitage *et al.* (2013), these factors contribute to increased runoff peaks in the following ways:

- Replacing the once permeable landscape with a variety of impervious surfaces – rooftops, roadways, parking lots and other paved surfaces – reduces the amount of infiltration into the soil.
- Impermeable urban areas are typically drained by other forms of impermeable infrastructure, such as pipes and lines channels, which don't allow for infiltration. This infrastructure usually discharges runoff directly into a nearby watercourse – which has to deal with greater flow rates and volumes than prior to development.
- Clearing vegetation to make way for new developments results in the loss of its role of intercepting, slowing down and storing runoff and promoting evapotranspiration.
- The compaction that often takes place prior to development reduces the infiltration capacity of the subsoil, thus reducing the amount of runoff that is infiltrated.

It is generally accepted that urbanisation causes the interval between flood events to decrease and the amplitude of floods to rise, although there is a minority that disputes this (Goudie and Viles, 1997). Greater flood peaks are usually found in commercial and industrial land use areas where there is generally a greater proportion of impervious surfaces (Endreny, 2006). Runoff from smaller storms also increases; USEPA (2003) estimates that a typical city block can generate up to five times more runoff than a woodland area of the same size – as a direct result of impervious surfaces.

Figure 2-1 compares general runoff characteristics between natural ground cover and impervious surfaces using percentage estimates to illustrate relative differences.

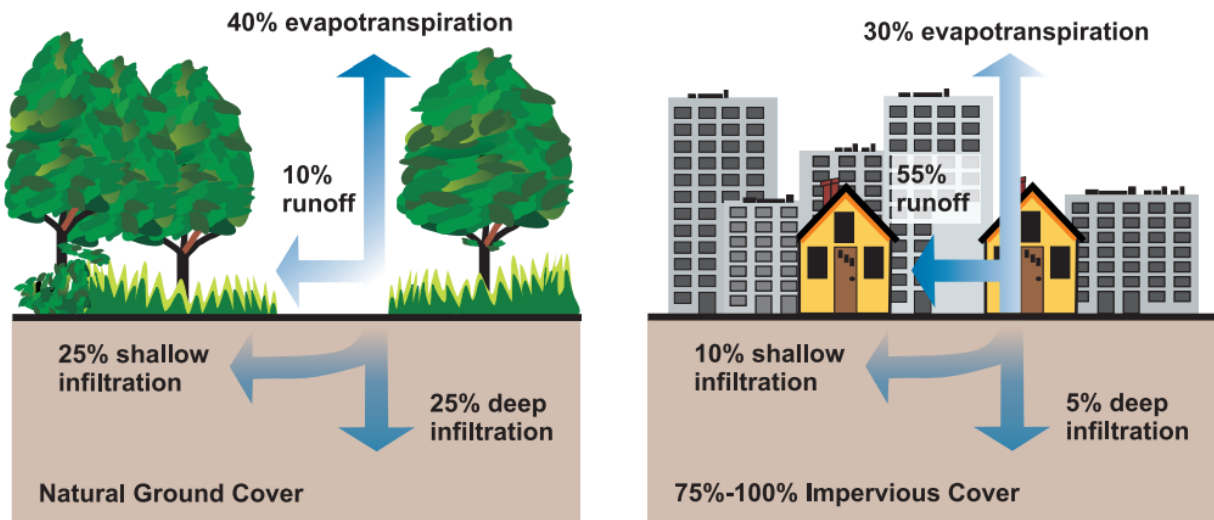


Figure 2-1: A comparison of runoff between pervious and impervious areas
(USEPA, 2003)

2.1.2 Pollutants in urban stormwater

Increasing urbanisation results in the increase of pollutants in the environment, transported by runoff (Edokpayi *et al.*, 2017). Urban stormwater carries a variety of pollutants, the most common of which are:

- **Sediments:** Topsoil washed off from paved surfaces and agricultural fields, sand from construction sites, and erosion as a result of streambed alteration are common contributors to sedimentation in urban catchments (Ashton & Bhagwan, 2001). Sedimentation is a serious problem because other pollutants, such as fertilizer compounds and heavy metals, stick to sediments and are carried into receiving waterbodies (Khatri & Tyagi, 2015).
- **Nutrients:** While nutrients (phosphorus and nitrogen) are essential to aquatic life, an excess can cause ecological imbalance. There are many sources of nutrients in urban catchments, including animal waste and fertilizers from agricultural areas, fertilizers and treated effluent runoff from golf courses and gardens, atmospheric deposition and sewage spills (Khatri & Tyagi, 2015; Edokpayi *et al.*, 2017).
- **Heavy metals:** Heavy metals are typically washed off from industrial and commercial land use areas – mainly metal roofs and structures, vehicle components, and metallic road structures (Saskon *et al.*, 2018). Runoff from agricultural areas may also contain heavy

metals such as cadmium, lead and arsenic that are occasional constituents of fertilizers and pesticides (Atafar *et al.*, 2010).

- **Oils and other organic matter:** Organic matter from vehicles such as oils, greases, brake fluid, fuel and polycyclic aromatic hydrocarbons (PAHs) enter stormwater from areas with vehicular activity (Legret & Pagotto, 1999). These pollutants are mostly washed off from vehicle repair workshops, typically found in industrial land use areas.
- **Pathogens:** One of the most serious human health problems in South African waterways is the presence of faecal pollution (Edokpayi *et al.*, 2017). *E. coli*, faecal coliforms and other pathogens are washed off from informal settlements (often as a result of inadequate sanitation and solid waste disposal in the settlement). Harmful pathogens also enter stormwater due to dysfunctional sewage systems (leaking pipes and/or broken pumps) that spill untreated sewage onto roads and into the ground (Ashton & Bhagwan, 2001). *E. coli* levels are used as indicators of faecal pollution, because they are easier to detect. High *E. coli* levels therefore imply the presence of potential presence of other pathogens.

The types of pollutants washed off into runoff is influenced by land use (Khatri & Tyagi, 2015). Ashton & Bhagwan (2001) summarise expected pollutant concentrations in stormwater runoff (Table 2-1) for residential, industrial and commercial areas based on a review of South African studies. Data ranges for agricultural land use could only be found for one area near Durban, on South Africa's east coast (Dlamini *et al.*, 2021); pollutant concentrations are based on samples taken from a river in an agricultural sub-catchment.

Table 2-1: Expected pollutant concentration ranges for various land uses in South Africa

Pollutant	Unit	Residential ¹		Industrial/ Commercial ¹	Agricultural ²
		High density	Low density		
Suspended solids	mg/ℓ	20 - 8000	1 - 2500	40 - 2000	-
Phosphates	mg/ℓ	0.2 - 6.0	0.0 - 6.0	1 - 6	0 - 8.01
Ammonia	mg/ℓ	1 - 30	0 - 21	1 - 8.2	0 - 0.93
Nitrates & nitrites	mg/ℓ	-	-	-	0 - 24.1
Total Kjeldahl Nitrogen (TKN)	mg/ℓ	2 - 103	1 - 16	1 - 74	-
Faecal coliforms	/100 mℓ	100 - 10 ⁷	0 - 10 ⁴	5000 - 750 000	-

¹ From Ashton & Bhagwan (2001) and Coleman (2001)

² From Dlamini *et al.* (2021)

2.1.3 The threat of eutrophication

The various pollutants in urban stormwater inevitably make their way into rivers and waterways through surface runoff and subsurface flow (Khatri & Tyagi, 2015). In a country like South Africa, healthy rivers are particularly important for human health since many rural residents rely on rivers as their main source of water (Edokpayi *et al.*, 2017).

One of the major threats to surface water quality in South Africa is eutrophication – which is the excessive growth of aquatic plants caused by elevated levels of nitrogen and phosphorus in surface waters (Griffin *et al.*, 2014). This excessive plant growth potentially leads to algal blooms, with devastating effects on ecosystem health including increased turbidity, reduced oxygen availability, increased sedimentation and greater levels of particulate organic matter. Extreme cases result in the death of local fish, birds and wildlife in river systems (Edokpayi *et al.*, 2017). Eutrophication is also a threat to the health of local estuaries – half of South Africa's estuaries are affected by nutrient pollution, resulting in hypoxia, fish kills and loss of ecosystem services (Adams *et al.*, 2020).

2.2 South African water quality standards

The South African Water Quality Guidelines for Aquatic Ecosystems (DWAF, 1996) describe eutrophic conditions in waterbodies in relation to inorganic phosphorus and inorganic nitrogen concentrations. Inorganic phosphorus in rivers is typically represented by dissolved inorganic phosphorus, largely made up of orthophosphates (Watson *et al.*, 2018). Orthophosphates are the only form of soluble inorganic phosphorus that are directly utilizable by aquatic organisms – thus are often referred to as soluble reactive phosphate (SRP) (DWAF, 1996). Total inorganic nitrogen (TIN) represents the nitrogen contribution of ammonia and ammonium ($\text{NH}_3\text{-N}$) and nitrites and nitrates ($\text{NO}_x\text{-N}$).

Eutrophic conditions are expected when the mean summer concentration of inorganic phosphorus is between 0.025-0.25 mg/ℓ or the mean summer concentration of inorganic nitrogen is between 2.5-10 mg/ℓ. Values exceeding the given ranges result in hypertrophic conditions. The various ranges and their resultant conditions are shown in Table 2-2.

Table 2-2: Target water quality ranges (DWAF, 1996)

Mean summer concentration (mg/l)		Resultant conditions	Typical effects
Inorganic P	Inorganic N		
< 0.005	< 0.5	Oligotrophic	Moderate levels of species diversity; low productivity systems with rapid nutrient cycling.
0.005-0.025	0.5-2.5	Mesotrophic	High levels of species diversity; productive systems; nuisance growth of aquatic plants and blooms of blue-green algae, but seldom toxic.
0.025-0.25	2.5-10	Eutrophic	Low levels of species diversity; highly productive systems, nuisance growth of aquatic plants and blooms of blue-green algae, which may include species toxic to man, livestock and wildlife.
> 0.25	> 10	Hypertrophic	Very low levels of species diversity; very highly productive systems; nuisance growth of aquatic plants and blooms of blue-green algae, often including species toxic to man, livestock and wildlife.

2.3 Managing nutrient levels: Sustainable Drainage Systems

Traditional methods of urban stormwater management are based on the principle of collecting and transporting runoff away from urban areas typically using concrete conduits; however, this method is increasingly being hailed as inappropriate to deal with the water quality problems resulting from increasing urbanisation (Khurelbaatar *et al.*, 2021). The Sustainable Drainage Systems (SuDS) approach has been developed as an alternative to conventional stormwater management.

2.3.1 Introduction

SuDS include a variety of practices which primarily involve slowing down and reducing the quantity of stormwater runoff from a developed area with the main goals of reducing downstream flood risk and reducing the risk of that runoff polluting downstream waterways (Woods Ballard *et al.*, 2015). Unlike traditional conventional stormwater management practices – which simply convey stormwater into underground pipelines that transport it away from the urban area – SuDS make use of a variety of mechanisms such as: harvesting, infiltration, slowing, storing and treating runoff on site and preferably above ground (Woods Ballard *et al.*, 2015). This way, the runoff becomes a visible feature of the built environment where it can add amenity and biodiversity value.

Woods Ballard *et al.* (2015) describe the underlying objectives of SuDS as: i) effectively managing stormwater runoff quantity and quality, while ii) promoting any associated amenity and biodiversity value. Figure 2-2 shows these objectives as a hierarchy where achieving the

lower levels has greater priority than the higher levels, although each successive level leads to a more enhanced and sustainable system (Armitage *et al.*, 2013).

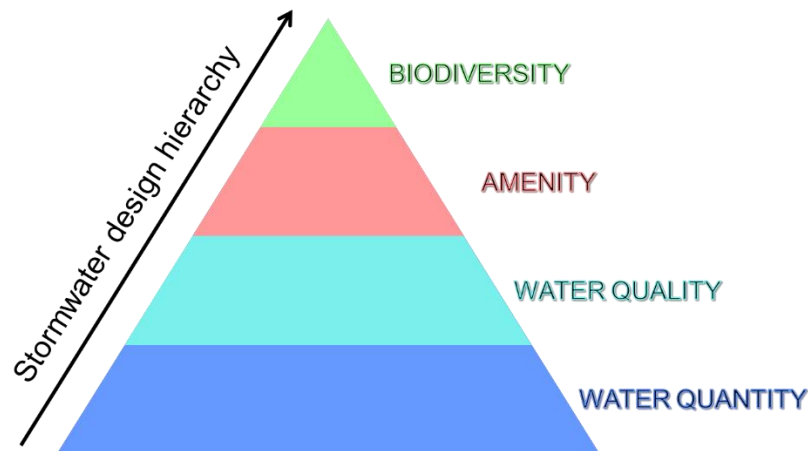


Figure 2-2: The SuDS design hierarchy (after Armitage *et al.*, 2013)

Appropriately designed SuDS offer many wide-ranging benefits; Woods Ballard *et al.* (2015) describe how the promotion of infiltration increases groundwater recharge. SuDS also provide ‘ecosystem services’ – benefits to an ecosystem society – by boosting biodiversity, enhancing the liveability of urban environments, decreasing ambient temperatures and encouraging urban agriculture (Vincent *et al.*, 2017).

2.3.2 Classification of SuDS measures

There are a variety of interventions that fall under the broad umbrella of sustainable drainage; Woods-Ballard *et al.* (2007) and Armitage *et al.* (2013) classify interventions based on the scale at which they will most likely be used. SuDS measures are grouped under the following categories:

- Good housekeeping: Measures to prevent the release of pollutants into the environment and thus prevent the spread of these pollutants via stormwater.
- Source controls: SuDS measures that can be implemented as close to the source as possible, typically within a site boundary.
- Local controls: Measures that are usually instituted at a neighbourhood or town level, generally in public areas such as parks and along road reserves.
- Regional controls: Large-scale interventions such as detention ponds, usually built on municipal land.

Various interventions associated with the groups listed above are presented in Figure 2-3. SuDS interventions are not intended to function as stand-alone processes but rather as a series of sequential processes known as a treatment or management train. Any number of these interventions can be implemented at any scale deemed necessary. Arranging treatment trains as a combination of various unit processes not only allows SuDS solutions to be tailored to the requirements of each site, but also creates a more resilient drainage system with multiple ‘lines of defence’ (Armitage *et al.*, 2014). The following sections explore selected local and regional controls in more detail. Good housekeeping and source controls are not discussed in further detail as these types of SuDS were not used in the modelling exercise (Section 4.5.1).

2.3.3 Local controls

This section lists various local SuDS controls and how they function. More information on each of these controls is provided in Appendix G.

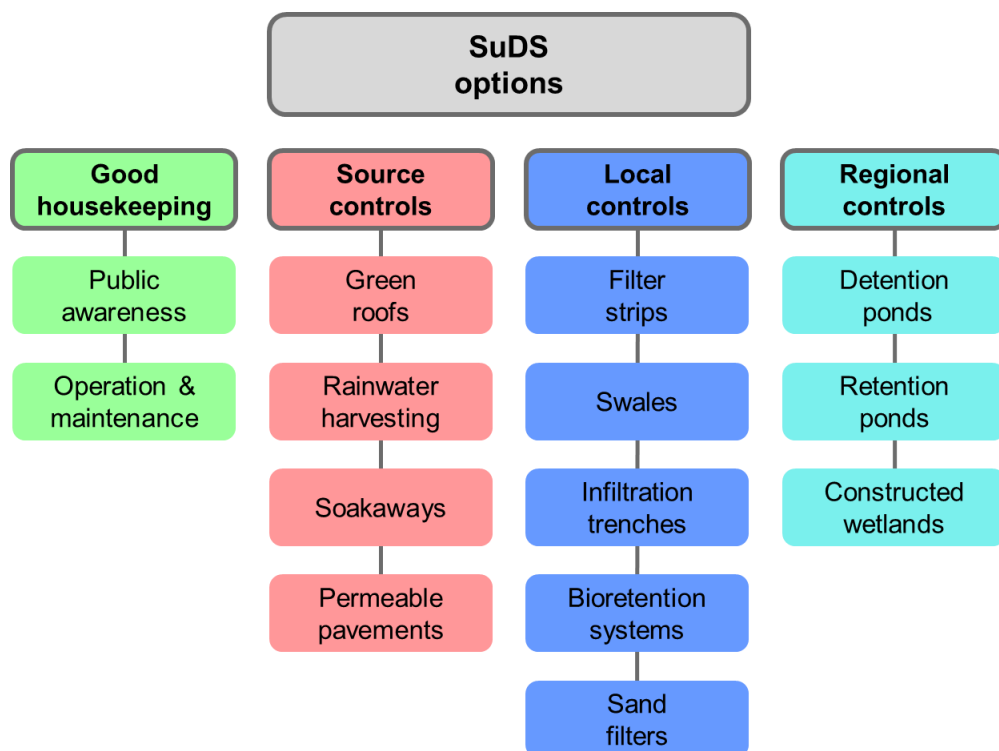


Figure 2-3: Grouping of SuDS options (after Armitage *et al.*, 2014)

2.3.3.1 Filter strips

A filter strip is an area of grass that is used to manage shallow stormwater runoff through various filtration processes.

The main functions of a filter strip are dispersing runoff and removing pollutants and sediments from stormwater. The grass or vegetation intercepts the stormwater flow and spreads it out, which can aid in the attenuation of flood peaks (Armitage *et al.*, 2013). Pollutant removal is largely accomplished by the vegetation in the strip, which removes sediment and other settleable solids such as hydrocarbons. Soluble nutrients, on the other hand, are not easily removed by the vegetation; they either pass through the filter strip or infiltrate into the soil, where they can be bound to organic matter and then removed by biological processes (Debo & Reese, 2003).

Because of their filtering ability, filter strips are frequently used as a pre-treatment device at the initial stages of treatment trains, often followed by bio-retention areas, infiltration trenches and swales. Common settings are along stream banks as vegetated buffer strips, and downstream of agricultural land and paved areas where they defend against sheet flows (Debo & Reese, 2003).



Figure 2-4: Filter strip draining the adjacent paved parking area (CORE, 2020)

2.3.3.2 Sand filters

A typical sand filter setup consists of two linked sections: a sedimentation chamber, in which suspended particles and heavy metal are separated from stormwater, and a filtration chamber, where sand and/or other filtration media remove smaller particulate pollutants. These chambers can be above or below ground. After water passes through the second chamber, it can either enter

the conveyance system via a perforated pipe or percolate into the surrounding soil (Woods Ballard *et al.*, 2015).

Sand filters are primarily for water quality improvement; hydrocarbons, sediment and other particulate pollutants are effectively removed from stormwater runoff through filtration by the sand bed and microbial action within the filtration media. They are often used to treat first flush runoff from adjacent impervious areas. Additionally, filtered effluent can contribute to groundwater recharge.

Because sand filters are generally located below ground, they can be accommodated on most sites. They are mostly used in impervious areas where land for other SuDS options is limited, in arid regions where high evaporation rates make measures with open water unsuitable, in areas with fine soils and low infiltration rates, and to protect groundwater resources (Field & Sullivan, 2003).

2.3.3.3 Swales

Swales are shallow channels usually with sloped vegetated sides. They are typically grass-lined but may sometimes have alternative linings such as gabion-lined “dry” swales, depending on site-specific requirements (Field & Sullivan, 2003).

Swales transport stormwater runoff further downstream, while infiltrating and treating the water. The vegetation filters larger pollutant particles and sediment, while dissolved pollutants are removed via infiltration and biofiltration (Debo & Reese, 2003). Similar to filter strips, dissolved pollutants that are trapped between soil particles and other organic matter are removed via infiltration. Infiltration also contributes to reduced runoff volumes and delayed peak flows.

According to Armitage *et al.* (2013), swales are often used as an alternative to road medians, roadside kerbs and gutters – typically in low density residential areas. They are sometimes used in conjunction with bio-retention systems in a treatment train. Like filter strips, swales provide a pre-treatment function for runoff (Melbourne Water, 2005). In flatter areas, they can even function as detention basins (Section 2.3.4.1).



Figure 2-5: A shallow swale in a housing estate in Malmö, Sweden (Wilson *et al.*, 2010)

2.3.3.4 Infiltration trenches

Infiltration trenches are shallow excavations lined with a geotextile and filled with large granular material, such as rock or a synthetic void-forming material. The fill is typically covered with a second geotextile overlaid by a layer of soil.

The primary function of an infiltration trench is infiltration and temporary storage – stormwater enters the trench and is stored in the voids between the granular material. This water then gradually percolates through to the surrounding soil where it can contribute to groundwater recharge. This mechanism decreases the frequency and extent of flooding. Infiltration trenches are also effective in the removal of various pollutants such as sediment, coliform bacteria, organic matter and metals (Field & Sullivan, 2003). Their pollutant removal ability can be enhanced with an appropriately designed pre-treatment system, a sediment trap or by including washed aggregate and layering the subsoil with organic matter and topsoil (Taylor, 2003).

Infiltration trenches are used in a variety of settings; they are most effective when placed alongside roads, footpaths, parking lots and other impervious areas (Woods Ballard *et al.*, 2015). They are commonly implemented in residential areas.



Figure 2-6: An infiltration trench in Warwickshire, UK (Stott, 2010)

2.3.3.5 Bio-retention areas

A bioretention area is a shallow landscaped depression designed to capture and temporarily store runoff on the surface, before filtering through vegetation and soils. The runoff is then either collected in an underdrain or infiltrated (Woods Ballard *et al.*, 2015). They usually comprise small stormwater management devices: grassed strips, ponding areas, sand beds, mulch layers and various types of plants.

Bio-retention areas serve multiple functions. The vegetation and ponding areas intercept runoff, prompting infiltration and sedimentation. The vegetation, mulch and soil remove nutrients, heavy metals, pathogens and various suspended solids through filtration and biological uptake (Endicott & Walker, 2003). They reduce peak flows, improve the quality of stormwater entering watercourses further downstream and contribute to groundwater recharge.

Bio-retention areas are well-suited to a variety of sites: between residential plots, in parks, alongside landscaped impervious areas or parking lots and adjoining roadways.



Figure 2-7: A bio-retention system in Portland, USA (EmilyBlueGreen, 2013)

2.3.4 Regional controls

2.3.4.1 Detention ponds

A detention pond, also called a detention basin, is an empty surface depression/excavated area that can hold water for short durations (typically 24-48 hours) following a storm event. These basins are usually grass-lined but can also be sand or concrete-lined depending on the soil and land use conditions.

The main function of detention ponds is to temporarily store stormwater to reduce flood peaks. Depending on the design specifications, the collected stormwater can either be infiltrated or conveyed to a watercourse downstream. Through detention and sedimentation, ponds achieve the additional objectives of regulating the flow in downstream watercourses and removing insoluble pollutants (Armitage *et al.*, 2013).

Detention ponds are found in a variety of locations: on construction sites for temporary sediment control, alongside streams for flood control and adjacent to parking lots and other impervious surfaces. Because they can be designed to serve multiple functions, they are also found in recreational areas like parks and often serve as sports fields when dry. However, care should be taken to ensure that pathogens do not remain after the water has infiltrated.



Figure 2-8: Partially filled detention basin during a storm event in Fairfax County, USA (Famartin, 2020)

2.3.4.2 Retention ponds

A retention pond is similar to a detention pond, except that it holds water throughout the year. It can be formed by using an existing natural depression, excavating a depression or creating embankments – usually in areas with high water tables. Where the water table is not high enough, the pond would require a lining to hold the water.

As the name implies, retention ponds retain runoff from storm events, thereby reducing peak flows and typically discharge water through a weir outlet structure. In the pond, runoff mixes with the existing water and undergoes a variety of pollutant removal processes: sedimentation, filtration, infiltration, chemical processes and biological uptake. The ponds are usually designed so that runoff is retained for long enough periods to achieve a relatively high level of pollutant removal (Woods Ballard *et al.*, 2015). Over time, the water is gradually released via the outlet.

Retention ponds are implemented in a variety of settings: along streams, in commercial and residential areas. Because they can add amenity and biodiversity value, they are well-suited to recreational areas – with the necessary safety precautions. They are also found on golf courses, in public open spaces and alongside parking and other impervious areas.



Figure 2-9: Retention pond in Madison, USA (Coyle, 2016)

2.3.4.3 Constructed wetlands

Wetlands are shallow waterbodies partly or completely covered in aquatic vegetation. Their unique plant characteristics and marshy areas attract a range of birds and wildlife, offering a sanctuary for endangered species. A constructed wetland is a man-made system designed to replicate the processes that occur in a wetland.

Wetlands provide both an attenuation and pollutant removal function but are generally designed to be more effective at the latter. With the range of vegetation and organisms in the wetland, they can be very efficient at the removal of particulates, dissolved nutrients and noxious substances such as heavy metals. They achieve this via a variety of processes: sedimentation, fine particle filtration and biological nutrient and pathogen removal (Field & Sullivan, 2003). Since longer residence times (time spent traveling through) in the macrophyte zone generally results in greater pollutant removal, wetlands are often designed to encourage meandering flow and thus increased residence times.

Constructed wetlands, sometimes combined with detention or retention basins, are typically found serving catchments larger than 10 ha, in open public spaces, downstream of areas that release water of poor quality or as part of a wastewater treatment works (Endicott & Walker, 2003). They can also be found in housing estates and commercial developments, where they add aesthetic, amenity and biodiversity value.



Figure 2-10: A constructed wetland in China (Haron, 2020)

2.3.5 Treatment capacities

The various SuDS measures vary in their pollutant removal potential (Table 2-3). Removal is expressed as a percentage range of total phosphorus (TP), total nitrogen (TN) and total suspended solids (TSS) removed. Treatment capacities vary because the different SuDS measures have different treatment mechanisms; furthermore, how well SuDS are managed and maintained contribute to how efficiently they remove pollutants.

Table 2-3: Typical SuDS pollutant removal capacities (Armitage *et al.*, 2013)

SuDS intervention	TP removal (%)	TN removal (%)	TSS removal (%)
Swales	25-80	30-90	60-90
Infiltration trenches	60-80	25-60	70-80
Bio-retention areas	50-60	40-50	50-80
Detention ponds	20-70	20-60	45-90
Retention ponds	30-50	30-50	75-90
Constructed wetlands	30-40	30-60	80-90
Extended detention shallow wetlands	30-40	50-60	60-70

2.3.6 SuDS selection

Any site requiring drainage will have its own needs and constraints; the various SuDS measures have their own advantages and limitations too, and these will have to be considered when selecting which SuDS to implement. Appropriate treatment trains (Section 2.3.2) are selected based on seven core criteria (Wilson *et al.*, 2004, as cited by Armitage *et al.*, 2013):

- i) Current and future land use
- ii) Site characteristics and utilisation requirements
- iii) Catchment characteristics
- iv) Stormwater runoff quantity (flood volume and peak flow)
- v) Stormwater quality
- vi) Amenity requirements
- vii) Biodiversity requirements

2.4 Summary

The impact of development has increased surface runoff quantity and associated flow volumes and peak flows. Rising urbanisation has also augmented nutrient levels in surface runoff – particularly nitrogen and phosphorus. Traditional pipe network-based drainage infrastructure is designed to efficiently route stormwater away from urban areas without taking the quality of runoff into consideration. These factors have caused the health of surface waterbodies to deteriorate – which could have far-reaching effects on human and aquatic life.

SuDS offer an alternative approach to stormwater drainage where systems promote infiltration and pollutant removal, giving rise to reduced flow volumes and peak flows, greater infiltration, groundwater recharge and improved surface and groundwater quality. Conventional drainage typically provides no amenity or biodiversity value and tends to cause deterioration of water quality in receiving waterbodies. The processes used in SuDS address water quality problems, and when appropriately landscaped also provide amenity and preserve biodiversity.

3. The Zandvlei Catchment

3.1 Overview

The Zandvlei catchment (Figure 3-1) covers approximately 87 km² (8700 ha), stretching from Wynberg Hill in the north, Cecilia Ridge and Constantiaberg along its western edge to the False Bay coastline and Muizenberg Mountain bordering the south. The eastern boundary follows the M5 roadway and passes through the suburbs of Plumstead, Retreat and Coniston Park (Hutchings *et al.*, 2016).

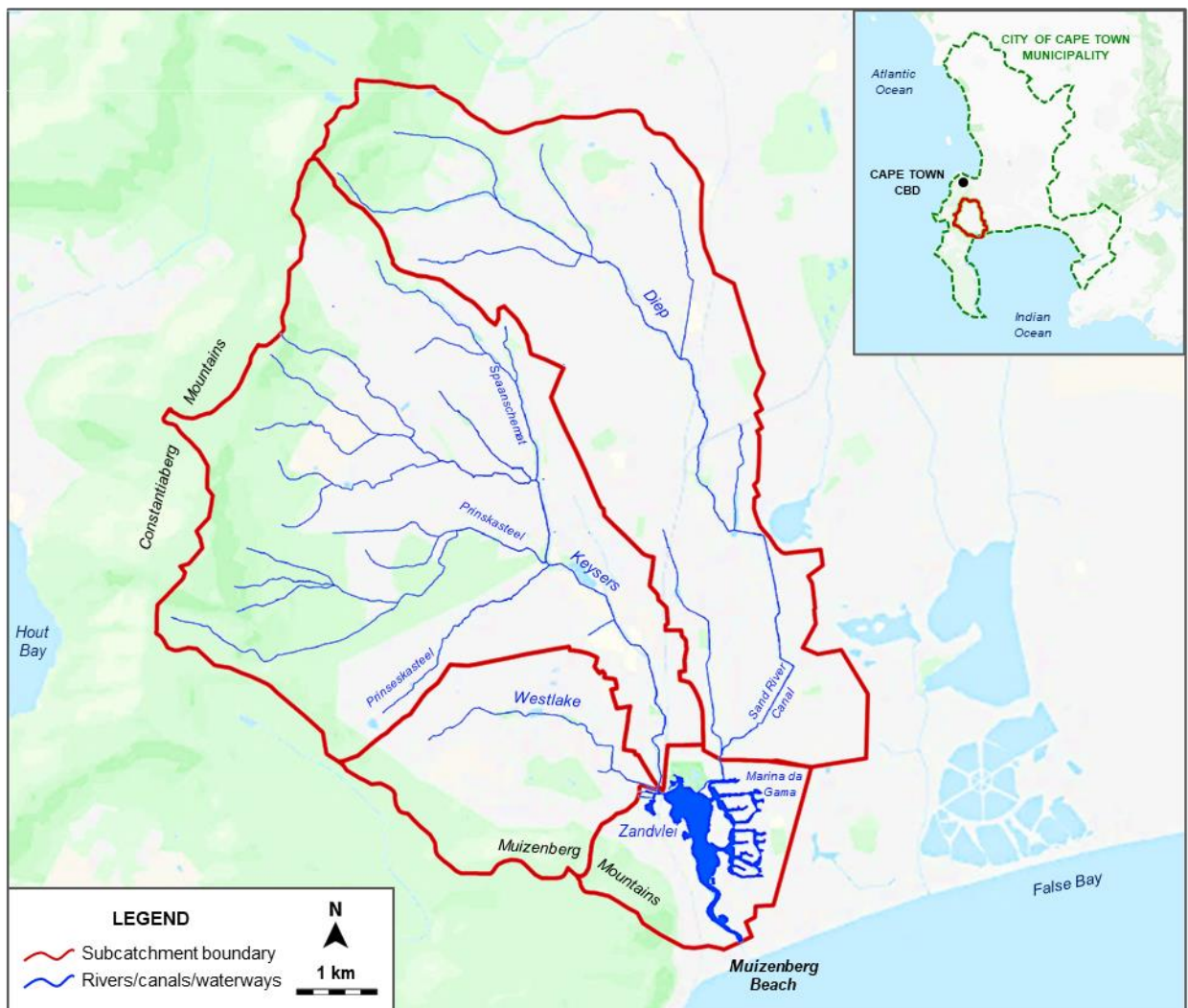


Figure 3-1: Map of the Zandvlei catchment (Google Maps, 2021)

The geology in the catchment includes the Malmesbury Group, Cape Granite Suite, Table Mountain Group and various Quaternary Age deposits (CCT, 2010). The oldest geological layer in the catchment is the Malmesbury Group, which is approximately 600 million years old (CCT, 2010). It occurs as a small outcrop in the north-west corner of the catchment and consists mainly of shale and quartzite. Cape Granites intrude into the Malmesbury Group and occur in the western half of the catchment. Along the mountainous western rim of the catchment, the Cape Granites are overlain by Table Mountain Group Sandstones. The mostly flat eastern half of the catchment comprises mainly of the Springfontyn and Witzand Formations, which are Quaternary Age deposits – approximately 1.8 million years old (CCT, 2010). The dune sands of the Witzand Formation occurs in the south-eastern portion of the catchment, extending to the False Bay coastline (Hutchings *et al.*, 2016).

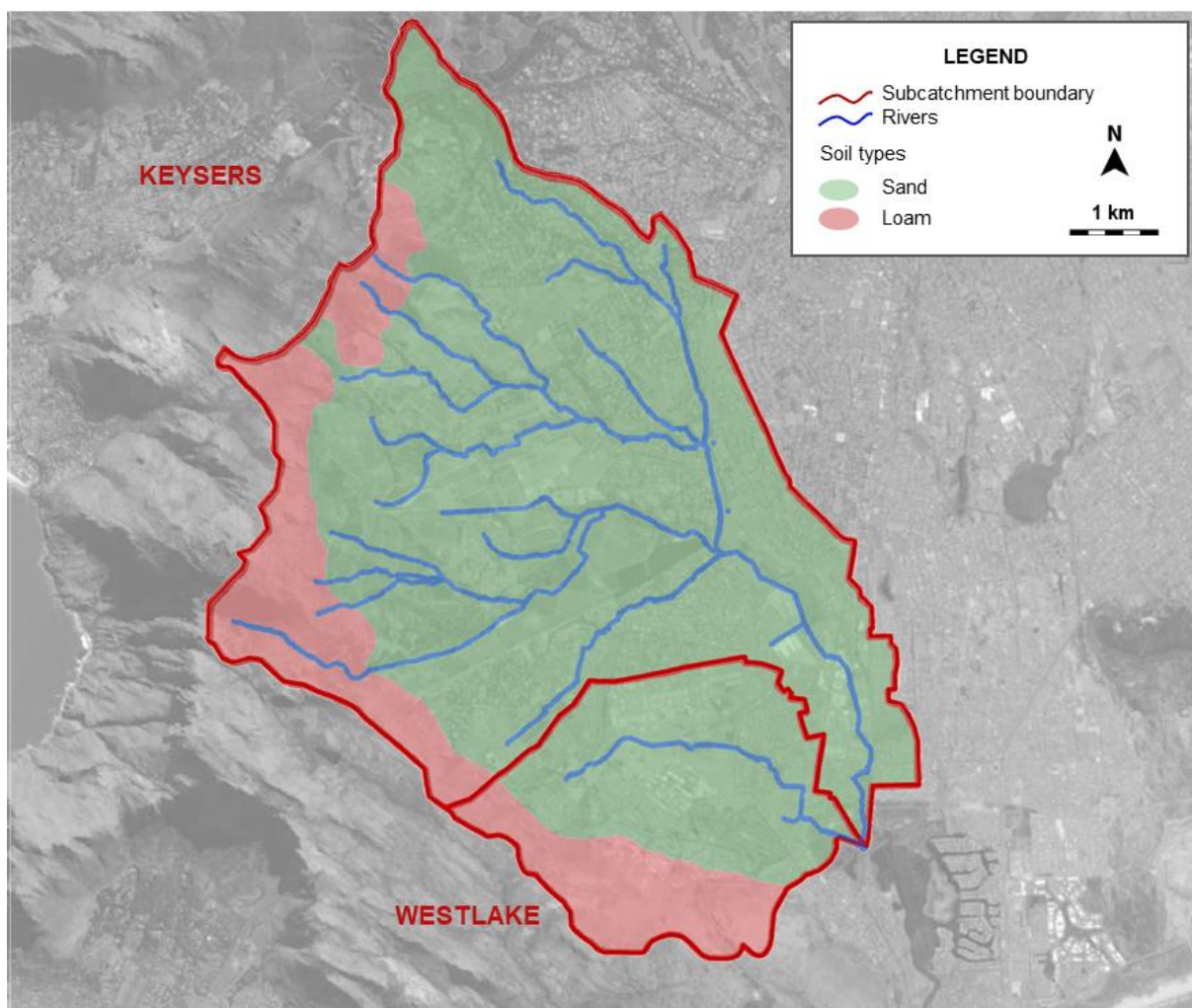


Figure 3-2: Soil types in the study area

The Zandvlei catchment is situated in a Mediterranean climatic region and receives most of its rainfall during the winter months (between May and September). Rainfall in the catchment varies considerably – the MAP ranges from 1200 mm along the western mountainous region to 690 mm around the estuary mouth. The region experiences hot, windy summers with a mean annual evaporation rate of 1400 mm (CCT, 2010).

The catchment derives its name from the relatively small (just under 1 km²) estuary at its southern outlet which drains three river systems of varying size: the Diep/Sand River, the Keyzers River and the Westlake River. The catchment has highly varied land use including agriculture, forestry, residential, industrial and commercial, amongst others (DEA&DP, 2018).

Various stakeholder groups are involved in planning and monitoring within the catchment, such as the Zandvlei Catchment Forum (ZCF), the Zandvlei Protected Areas Advisory Committee (ZPAAC), the Zandvlei Trust and others. These collectives represent various local governance structures as well as active citizen groups and convene multiple times a year to discuss and take action on catchment-related issues.

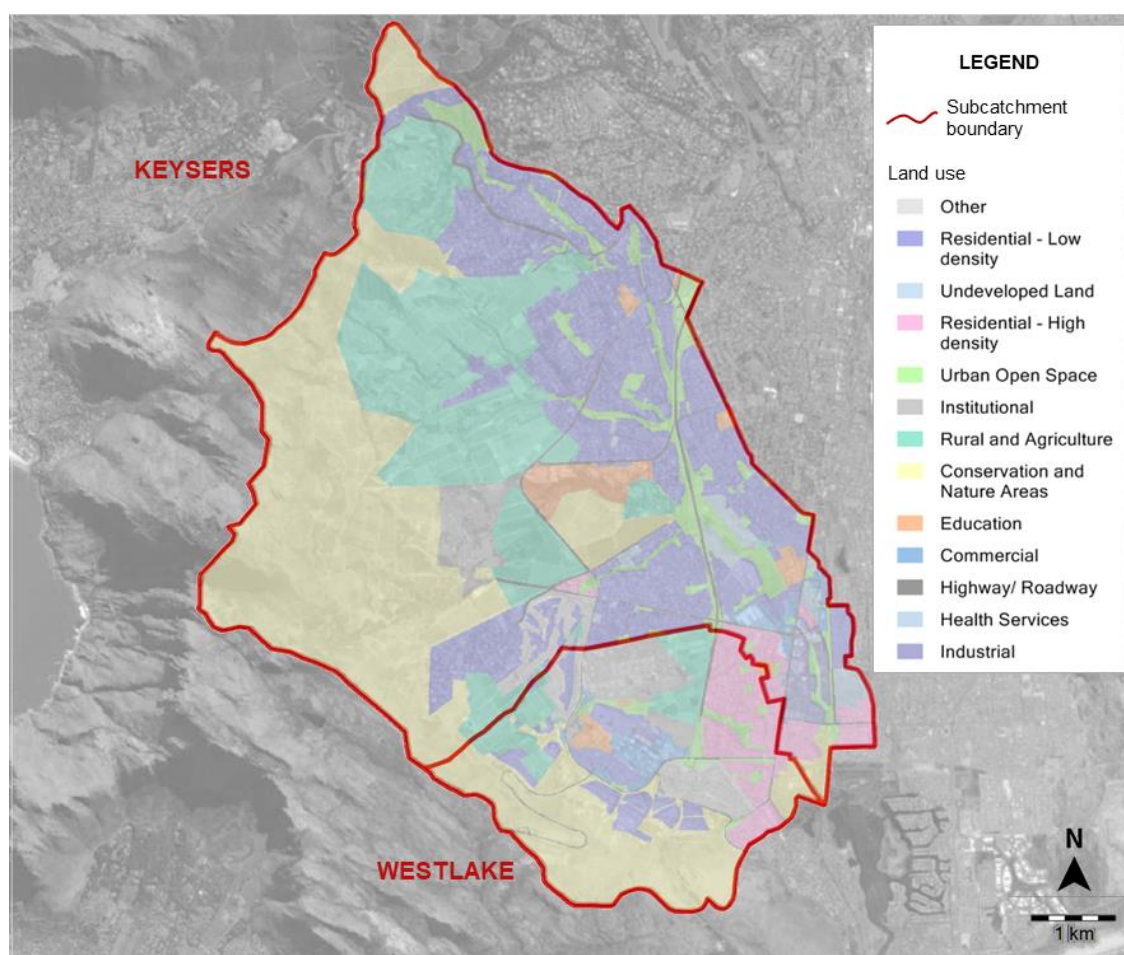


Figure 3-3: Land use within the study area

3.2 River systems feeding into the Zandvlei estuary

3.2.1 The Keyzers River system

The Keyzers River has many tributaries rising on the east-facing slopes of the Constantiaberg and Table Mountain, the largest being the Spaanschemat, Prinskasteel and Prinseskasteel River systems, draining the areas of Constantia, Firgrove and Tokai. These rivers all converge just upstream of the M3 freeway. The merged river then crosses under the M3 and is known as the Keyzers River from this point. Downstream of the M3 the river flows through Dreyersdal Farm alongside a dam and wetland which are only connected to the river through overflow channels. The Keyzers River crosses Main Road (M4) in Tokai before merging with the Westlake River and entering the Zandvlei estuary at its north-western edge.

The Keyzers River sub-catchment is the largest within the Zandvlei catchment. The area covers roughly 37 km² (3700 ha) and contains many different types of land use. Agriculture dominates the high-lying areas; the middle reaches are occupied by middle- to high-income level households with some agriculture; while the low-lying region is characterised by commercial and light industrial areas, including Blue Route Mall.

3.2.2 The Westlake River system

The Westlake River rises on Steenberg Mountain and initially passes through various small vineyards, after which it flows through the sculpture garden of the Norval Foundation gallery, the Pollsmoor Prison correctional facility grounds, and Kirstenhof Greenbelt where it flows past a small wetland which it is currently connected to via an overflow channel. The Westlake River crosses the M4 roadway in Lakeside, after which it merges with the Keyzers River and enters the estuary.

Two small tributaries join the main river downstream of the M3 freeway – the first is a shallow stream that drains the Westlake Golf Club and residential areas around the Orchard Park. The second starts at Hutton Road Park and runs along Sea Breeze Road, joining the main river at Raapkraal Road. This channel drains a large part of the suburb of Kirstenhof.

The Westlake River sub-catchment is much smaller than the other sub-catchments at approximately 10 km² (1000 ha). Land use in the catchment ranges from agricultural and education in the upper reaches to commercial, low-income residential and institutional in the middle reaches. Downstream of the M3 freeway, the lower reaches consist mainly of middle-income residential areas and a golf course.

3.2.3 The Diep/Sand River system

The area drained by the Diep River stretches from the southern end of Table Mountain, through Constantia, parts of Wynberg and down to Retreat and Steenberg. In the upper reaches, the river

flows through broad areas of open space, but downstream of the M3 freeway it flows through a concrete-lined canal until it enters the Zandvlei estuary along its northern boundary. Downstream of a small lake called Little Princessvlei, the river is known as the Sand River.

The Diep River sub-catchment spreads across 32 km² (3200 ha). It comprises high-income level households and some agriculture in the upper reaches, middle-income level households, some commercial and institutional areas in the middle reaches, and mainly low-income level households in the lower reaches.

3.3 The Zandvlei Estuary

3.3.1 Overview

Estuaries are highly significant areas in terms of biodiversity; as the interface between land and sea, estuaries are characterised by a combination of saline water from the ocean and freshwater flowing from upstream rivers, creating a unique habitat ideal for many fledgling fish species. The Zandvlei estuary is no exception; as the only functioning estuary on the False Bay coastline it is an important nursery for migrant juvenile fish in the area (DEA&DP, 2011).

In 2015 an area of 220 ha, including the estuary, was proclaimed as the Zandvlei Estuary Nature Reserve (ZENR) in terms of the Protected Areas Act. The reserve is home to a variety of endangered flora (e.g., the Cape Flats Dune Strandveld and Cape Peninsula Granite Fynbos) and fauna (e.g., the Western Leopard Toad, *Amietophrynus pantherinus* and the unique Ranger Butterfly, *Kedestes lenis*). Having previously been a bird sanctuary, the area is also a popular bird-watching site boasting 168 recorded species including the threatened African Marsh Harrier (*Circus ranivorus*) (DEA&DP, 2011). Aware of the importance of awareness regarding the area's biodiversity value, the ZENR regularly conducts environmental education programmes that create opportunities for collaboration between the reserve, local students and environmental clubs and the local active citizenry.

Besides the conservation value of the estuary, it also has significant recreational and socio-economic value. Activities such as sailing and fishing regularly occur in the main body of the estuary which contribute to tourism in the area, and the estuary's function as a fish nursery is essential in the replenishing of fish stocks in the False Bay area.

The estuary itself consists of the shallow main body and the constructed Marina da Gama housing development in the north-east corner of the estuary. As previously described, three river sub-catchments drain into the estuary: the Diep/Sand River joins the Langevlei Canal and enters the estuary through a concrete channel along its north-east boundary near Marina da Gama, whilst the Keyzers and Westlake Rivers join downstream of the Westlake wetland before entering the estuary on the north-east.

The estuary discharges into the ocean at Muizenberg where a sandbar and rubble weir are used to control discharge and thus the water level in the estuary. The manual control of the water

level in the estuary is just one in a series of human interventions that have affected the functioning and health of the estuary, as described in the following sections.

3.3.2 Brief history

The well-documented history of the estuary reveals numerous cases of human manipulation in its functioning. Morant and Grindley (1982) have traced the appearance of Zandvlei on maps as far back as the 1700s and it was supposedly named by the Jan van Riebeeck in the 1650s, before the region was established as a cattle post. About 200 years later, after the water level in the estuary drastically declined due to a severe drought in 1886, a plan to drain the estuary and reclaim the land was initiated. This ultimately failed as the rivers that flow into the estuary were not diverted – causing the estuary to flood with the rains of the following winter (DEA&DP, 2018). However, part of the region continued to be used for agriculture thereafter.

The landscape surrounding the estuary was permanently transformed with the arrival of the railway at Muizenberg in 1882 which ushered in an era of increased recreational use of the estuary. This development severed the area currently referred to as the Westlake Wetland and Westlake Pond from the main estuary body (Hutchings *et al.*, 2016). While tourism in the area continued to rise in the decades that followed, it was still an important agricultural site. One of the first additions for agricultural use was the construction of weirs where the rivers enter the estuary in the 1930s – this was done to control flow between the rivers and estuary and preserve the freshwater quality (CCT, 2010).

Soon after, further interventions were required – this time to maintain the recreational role of the estuary. The seasonal lowering of the water level – so drastic that the estuary occasionally dried up completely by the end of summer – impeded boating activities to such an extent that the estuary was dredged in 1947 to allow for more water in the estuary. As a result, 32 ha of wetland was eradicated and the gently sloped shoreline transformed into steep banks requiring artificial stabilisation (Morant & Grindley, 1982).

In the 1950s the outlet channel was canalized; this accompanied the construction of the Royal Road Bridge (over the mouth of the estuary) and a concrete weir just below it. Natural scouring caused the concrete weir to collapse and expose an upstream sewer pipeline, after which it was replaced with a rubble weir (DEA&DP, 2018).

The construction of the Marina da Gama residential development and its canals on the eastern shore of the estuary was completed in the early 1970s, causing a surge in the number of inhabitants in the vicinity and adding to the gradual urbanisation of the catchment. With the increased use of the estuary, maintaining an adequate water level for the popular boating activities became ever more crucial – yet continued to prove a challenge. In the 1980s the rubble weir gained the additional function of controlling the water level in the estuary, with a sandbar over the mouth of the estuary aiding this function (CCT, 2010). As previously alluded to, these two control devices are still used to manage the water level in the estuary.

3.4 Some brief catchment challenges

The Zandvlei catchment and estuary experience many challenges; some of the main catchment management issues are described in the following sections.

3.4.1 Salinity

Maintaining salinity levels is important for the role the estuary serves as a fish nursery. The history of mouth manipulation has resulted in the estuary being unable to naturally achieve the salinity levels required for fledgling fish species to grow (Quick & Harding, 1994). Historically, salinity has continuously fluctuated from the 1980s, with changes often linked to adjustments of the weir level (Hutchings *et al.*, 2016).

3.4.2 Sedimentation

Sedimentation, largely from bank and bed erosion in response to urbanisation in the upstream catchments, is a long-established problem in the estuary; Decades ago, Morant & Grindley's (1982) study pointed to the loose material from non-stabilised areas, such as building sites, infiltrating rivers and streams as the cause of sedimentation in the estuary. They predicted that continuing urbanisation in the catchment would only intensify the silt levels in the rivers – a prophetic statement considering Ninham Shand's (2000) report (as cited in DEA&DP, 2018) that indicated that between 1988 and 2000, 44 000 m³ of sediment had been deposited in the estuary. For that 12-year period, this equated to 77 mm across the entire estuary area – higher than the 50 mm for the 12-year period between 1958-1970.

Besides upstream sediments, tidal inflows carrying sand from the sea increases the sediment load in the estuary's outlet channel, where it gets trapped by the weir. Even though various interventions exist to mitigate the sedimentation from both upstream and downstream of the estuary – such as the extensive reed bed at the confluence of the Keysers and Westlake Rivers and a light dredging programme at the outlet channel – reducing sedimentation continues to be a key priority for the catchment (CCT, 2010; DEA&DP, 2011; Hutchings *et al.*, 2016; DEA&DP, 2018).

3.4.3 Litter and debris

A problem that has continued to aesthetically plague the Zandvlei estuary is the stream of litter and debris that flows into the estuary, particularly in the rainy winter months. Litter enters the estuary from both the upstream rivers, particularly those draining low-income or informal residential areas, and along the banks of the estuary through recreational use (CCT, 2010). The prevalence of debris in the waterbody presents a danger to local flora and fauna in the form of entanglement and ingestion.

Various measures have been instituted to control litter flow into the estuary, such as the litter trap on the Sand River Canal (Figure 3-4). This litter trap frequently gets blocked during a storm event, causing the canal to flood and the litter to overflow and thus flow into the estuary. The ZPAAC recently commissioned the design of an improved litter trap, which has yet to be constructed (ZPAAC, 2018).



Figure 3-4: Blocked litter trap on the Sand River Canal during a storm

3.4.4 Nutrient content in rivers

With such a spectrum of land uses in the catchment, it is no wonder that elevated nutrient content is one of the more serious issues afflicting the rivers and the estuary.

Concerns arose as early as the 1970s when a monitoring of nitrogen and phosphorus in the estuary indicated that it was in a eutrophic state. More recent monitoring between 2000-2009 found a similar, though slightly improved, situation (CCT, 2010). Eutrophication in any waterbody is undesirable, but in one with the biodiversity and recreational value of the Zandvlei estuary, it could be catastrophic. The excessive plant growth caused by eutrophication blocks sunlight from penetrating the water surface, depletes oxygen levels in the water – affecting both aquatic plant and animal and life – and thus allows the plants to rapidly grow. In extreme cases

toxins may be released, exacerbating this vicious cycle. Such was the case in 2012 when a ‘golden’ algal bloom (specifically *Prymnesium parvum*) left hundreds of dead fish washed up on the estuary shore, birds floating lifelessly on the water surface and the estuary itself in an inaccessible state (Isaacs, 2012). A similar bloom, resulting in yellow surface waters and gasping fish, occurred in 2017 (Lwandle, 2017; ZPAAC, 2017).

Increased nutrient levels produce other blooms in the catchment that, while less destructive than golden algae blooms, are still undesirable. The Westlake River experiences frequent hyacinth blooms (Figure 3-5), while the estuary and Marina da Gama canals often suffer pondweed overgrowths (*Stuckenia pectinata*, previously known as *Potamogeton pectinatus*) (DEA&DP, 2018). However, pondweed occurs naturally in the estuary, so its existence itself is not due to nutrient-enriched waters – rather, it is a vital component of the estuary’s ecosystem (Quick & Harding, 1994). This is explored further in the following section.

The possible sources of raised nutrients in the catchment are discussed in Chapter 5.



Figure 3-5: Water hyacinth bloom in the Westlake River at Kerner Close, facing Main Road (downstream)

3.4.5 Pondweed management

Although managing the pondweed population is a continuous challenge in the Zandvlei estuary, it is an essential biotic component of its ecosystem responsible for maintaining good water quality. Besides providing feeding areas for juvenile fish and a variety of aquatic fauna, it oxygenates the water while reducing nutrient loads. Furthermore, it increases water clarity by reducing sediment resuspension caused by wind or recreational activities (Quick & Harding, 1994). However, these benefits only occur when the pondweed population is at an optimum level. When there is an overgrowth of pondweed, dense plant mats form that have typical eutrophication effects: limited sunlight penetration, unpleasant odours and raised nutrient levels. In the Zandvlei estuary, excessive pondweed growth also hinders boating activities – one of the more popular recreational uses of the estuary – as well as restricting the flow of current, which adds to the existing problem of circulation in the Marina canals.

Currently the pondweed is managed by mechanical harvesting; but occasionally too much pondweed is removed causing an imbalance in the estuary's ecosystem (CCT, 2010).

3.4.6 Sewer spills

The estuary repeatedly undergoes closure on account of sewer spills, largely caused by faulty or vandalised pump stations and/or power failures. There are 14 pump stations in the areas around the estuary which regularly overflow. The pollution from overflowing pump stations is exacerbated by overflowing manholes caused by sewer blockages as a result of incorrect disposal of solid objects (DEA&DP, 2018). These spills flood the estuary *E. coli* and other pathogens that are harmful to human health – at times posing a risk considering the frequent recreational use of the estuary. One site that frequently experiences sewage overflows is Pollsmoor Prison, which drains into the Westlake River (Source-to-Sea, 2016).

Currently these spills are managed by estuary closure and regular monitoring, as well as pump station maintenance.

3.4.7 Urban runoff

Stormwater drains carry urban runoff directly into the Zandvlei estuary, carrying pollutants collected all over the catchment into the estuary. Runoff from the various land uses in the catchment are highlighted by many studies as sources of raised nutrients in the estuary. Particular regions of concern are the agricultural areas on the western slopes of the catchment, the industrial area in the lower reaches of the Keyzers sub-catchment, the two golf courses in the study area and the low-cost housing area in Westlake, which the Westlake River flows alongside (CCT, 2010; Source-to-Sea, 2016; DEA&DP, 2018).

3.5 Summary

The Zandvlei catchment derives its name from the estuary at its south, which is fed by three river systems. The catchment is diverse in its land uses, topography, rainfall and in the challenges it faces. Some of these challenges are the result of a history of human intervention in the functioning of the estuary, while others are caused by activities associated with the various land uses.



Figure 3-6: Litter in the Westlake River downstream of the low-cost housing area

4. Methods

This chapter describes the various methods employed in the study, commencing with a brief study overview. The process of water quality sampling and testing is explained, followed by a discussion on the data collection for construction of a hydrological, hydraulic and water quality model of the study area. Thereafter, the method of constructing the model is described, and finally the chapter closes with explaining factors for consideration in the design of SuDS alternatives.

All map backgrounds in this and the following sections were sourced from Bing Maps (2021).

4.1 Study overview

The study method can broadly be divided into two components: data gathering and modelling (Figure 4-1).

The data gathering commenced with a desktop study into the catchment to assess the drainage problems it faces. Studies of the catchment, which have been summarised in Chapter 3, have identified elevated nutrient levels – and ensuing eutrophication – in the rivers and estuary as a threat to human and aquatic health and local biodiversity. Water quality sampling was undertaken to identify locations of elevated nutrient levels (Section 4.2). Historical water quality data were also collected to identify long-term and seasonal trends.

Hydrological and hydraulic modelling of the study area was undertaken to investigate solutions to the raised nutrient levels in the form of SuDS implementation. Various types of data and information were collected and processed to create a model representing the study area (Section 4.3). Construction of the model involved many steps (Section 4.4). Once an acceptable representation of the ‘current’ state of the study area was achieved, through a process of calibration, various SuDS alternatives were modelled and grouped as scenarios (Sections 4.5 and 4.6).

4.2 Water quality sampling and testing

Water quality sampling and testing was carried out in the period between June – December 2019. The samples were tested in the UCT Water Quality Laboratory. This section describes the methods employed in the sampling programme and laboratory testing.

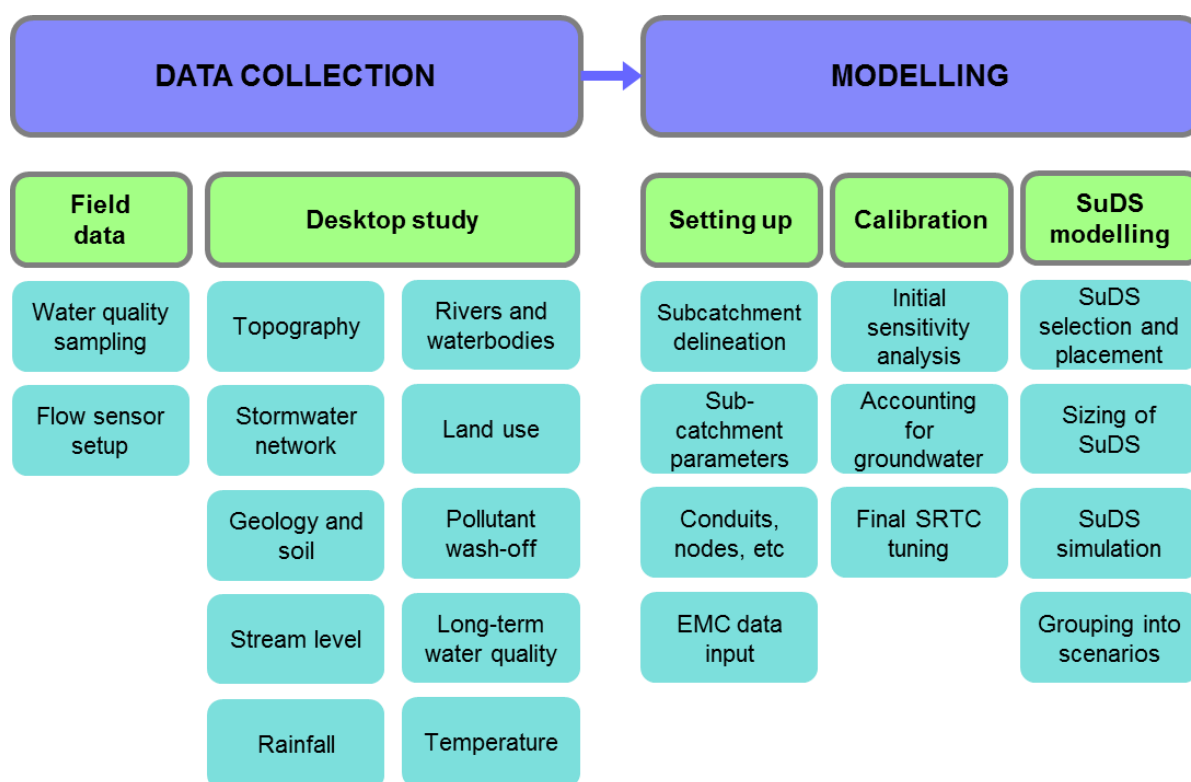


Figure 4-1: A graphic depicting various elements in the method

4.2.1 Limitations

Most of the water quality data presented in this study were limited to the results of grab sampling. Grab samples give an indication of water quality at a particular point in time and are not necessarily representative of the most significant or average values. The historical data supplied by the CCT were all based on grab samples. Continuous sampling was however undertaken by the researcher for two storms (Section 5.2.2) but limited financial resources did not allow for the analysis of additional continuous sampling.

Safety concerns were a key factor in selecting sample points. On multiple occasions during initial site visits, the researcher happened upon a group of men in a secluded area blocked from public view. Since sampling was often undertaken as a duo of two females – and sometimes only one as partners could not always be secured – locations that appeared potentially unsafe were avoided.

4.2.2 Sampling programme

The sampling programme had three main objectives: i) to investigate where nutrient concentrations were likely to be high based on EC readings so as to identify key sample locations,

ii) to compare the nutrient levels in winter and summer, and iii) to track how nutrient levels change during a storm. The various phases of the sampling programme are described below.

4.2.2.1 Preliminary river testing

Preliminary river testing consisted of grab sampling throughout the Zandvlei catchment during which three parameters were measured: electrical conductivity (EC), temperature and pH. These were chosen for the following reasons: i) they are easily measurable on site with portable pen meters, ii) electrical conductivity provides an indication of ions present in the water which, in turn, may give an indication of dissolved nutrients, and iii) high/low pH values could highlight areas of concern.

This phase of testing made use of OHAUS ‘starter pen meters’; the *ST20 pH* pen meter and the *ST20C-B Conductivity* pen meter were used to measure pH and EC respectively. Both meters have built-in thermometers for temperature measurement (OHAUS, 2019). Each pen meter was rinsed with distilled water and then calibrated before each use in the field, using standard procedures (OHAUS, 2014). The EC pen meter was calibrated using a pre-defined calibration standard of 1413 $\mu\text{S}/\text{cm}$. The pH pen meter was calibrated using standard buffer solutions of pH 4, 7 and 10 by being placed into each buffer solution consecutively. It was rinsed with distilled water between buffer solutions.

On site, pen meters were initially inserted directly into the river to obtain readings; however after a few attempts it was evident that readings would not stabilise – possibly due to the flowing water. Thus, on subsequent sampling trips, plastic bottles were used to collect a water sample. Bottles were rinsed with river water twice, after which a sample was collected. The pen meters were inserted into the bottles (Figure 4-2); after waiting a few seconds for readings to stabilise, the measurements were recorded, and the water was returned to the river. The pen meters were rinsed with distilled water before and after each use.

Measurements indicated that the highest concentration of ions was found in the Westlake River system, where EC values were usually within the range of 700-2000 $\mu\text{S}/\text{cm}$. The Diep River system displayed the lowest EC values, between 300-800 $\mu\text{S}/\text{cm}$, while the Keysers River system had measurements mostly between 400-1100 $\mu\text{S}/\text{cm}$, with greater EC values displayed in the lower reaches. A summary of the EC measurements is shown in Appendix E.

Sampling within and constructing a model of the entire Zandvlei catchment was considered too large a scope for this research project; since the EC readings of the Diep River system were so much lower than the other two sub-catchments, it was considered of lower priority and subsequently omitted from the rest of the study



Figure 4-2: A portable pH pen meter taking a reading in a sample bottle

The Keyzers River and its many tributaries have limited points of access; sampling in this system was thus limited to the lower reaches where EC was the highest and locations that were easily accessible and largely within public view. The Westlake system displayed the highest EC values and most of the river was easily accessible; sampling efforts were therefore concentrated on the Westlake River.

A map of the preliminary testing points, and the points which thereafter became sampling points, is shown in Figure 4-3. These were chosen before the CCT data became available. The EC readings from the preliminary testing phase are listed in Appendix B.

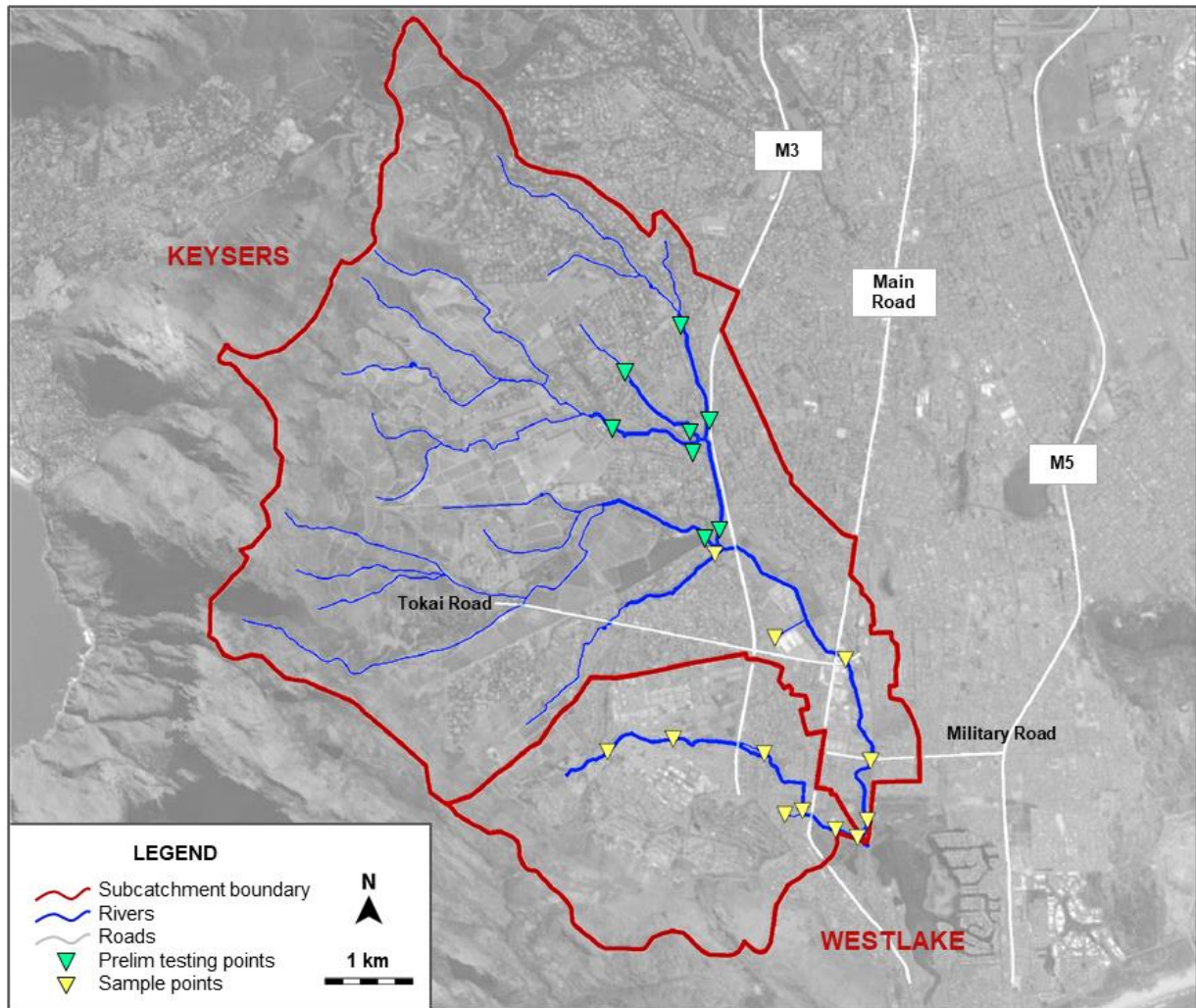


Figure 4-3: Map of the preliminary testing points and sampling points

4.2.2.2 Phase 1: Winter sampling

Phase 1 sampling consisted of two parts: i) weekly spot sampling on dry days (no rain for at least a week prior) for three weeks and ii) regular sampling during two storm events in the Westlake River.

For this phase, sampling locations were chosen based on EC readings captured in the preliminary testing phase. The South African Aquatic Guidelines (DWA, 1996) do not specify a limit for EC levels in freshwater streams; however the special limit for EC stipulated by the National Water Act for treated effluent is 1000 $\mu\text{S}/\text{cm}$ (DWA, 2013). This was used as a guideline, but since few of the measurements exceeded this threshold, 800 $\mu\text{S}/\text{cm}$ was instead chosen as the limit above which locations would be sampled.

As with the preliminary testing phase, river water was rinsed in sample bottles twice after which EC, pH and temperature readings were recorded. Instead of returning the river water, the

bottles were sealed, kept in a cooler box and transported to UCT's Water Quality Laboratory for testing.

4.2.2.3 Phase 2: Summer sampling

Phase 2 consisted of weekly sampling over two weeks, with revised sampling locations. When comparing the change in EC and the change in nutrient concentrations during storms, it appeared that they did not increase and decrease in tandem, suggesting that other ions were contributing to a greater degree to ionic activity in the water. For Phase 2, EC was no longer used as an indication of where to sample and more sampling points were added to the regimen.

In the upper reaches of the Keyzers river system there was seldom enough water in the tributary streams to allow for a sample to be taken. In the middle reaches, much of the Keyzers River is inaccessible – either behind an access-controlled complex (Tokai Estate) or alongside a residential area but behind a locked gate. Therefore the revised sampling points were largely in the lower reaches.

In the Westlake system, as much of the river that could be easily accessed was sampled. In the upper reaches, the river flows through the Norval Foundation art gallery and the US consulate however requests to sample in both these institutions were rejected. Pollsmoor Prison grounds was omitted as they would not allow regular sampling without being accompanied by a senior official, whose availability they could not guarantee. The areas near the confluence of the Keyzers and Westlake Rivers are surrounded in dense reeds (as shown in Figure 4-4). This region could only be accessed with a 4x4 vehicle, which fortunately was obtained in the second phase of sampling.

4.2.3 Testing procedures

All collected samples were tested by the researcher under standard operating procedures using a Thermo Fisher Scientific GalleryTM Discrete Analyser. Samples were tested for the following:

- Soluble reactive phosphate (SRP) – which are orthophosphates (PO₄³⁻)
- Total oxidised nitrogen (TOxN) – including nitrites (NO₂) and nitrates (NO₃⁻)
- Total ammonia nitrogen (TAN) – encompassing both ammonia (NH₃) and ammonium (NH₄⁺)

All samples were first vacuum filtered through a 0.45 µm membrane filter paper to remove any turbidity which can interfere with colorimetric analysis (Hach, 2013). Samples were then placed into sample cups, loaded into the designated racks and inserted into the analyser. The required



Figure 4-4: Dense reed cover surrounding the Keyzers and Westlake River confluence

reagents were similarly placed into cups and loaded into racks. After being inserted into the machine, each sample and reagent cup required labelling on the analyser's computer interface.

Each test requires a calibration; where it had expired, calibration was undertaken using the required solutions and method outlined in the laboratory's standard operating procedure manual. The analyser was able to perform automatic dilutions where the concentrations could not be measured. If results fell outside of the calibration curve despite the maximum number of dilutions being made, a new calibration was required.

SRP concentrations, measured using a colorimetric method, were calculated by measuring the absorbance of the compound formed when orthophosphates in the sample react with ammonium molybdate, which in turn react with ascorbic acid (Thermo Fisher Scientific Inc., 2015c).

TOxN concentrations were measured via the colorimetric hydrazine method; the nitrate in the sample is reduced to nitrite by hydrazine, and then undergoes a series of other reactions until it forms a pink compound. Similar to the TAN test method, the absorbance of this compound is measured and TOxN concentrations are calculated based on a calibration curve that relates absorbance to concentration (Thermo Fisher Scientific Inc., 2015b).

TAN was also determined using a colorimetric method; the ammonia in the sample reacts with hypochlorite ions, and then with salicylate ions in the presence of sodium nitroprusside to form a compound whose absorbance is then measured spectrophotometrically. The analyser calculates the ammonia concentration from the measured absorbance relative to a calibration curve (Thermo Fisher Scientific Inc., 2015a).

The results of the water quality testing are discussed in Chapter 5.

4.3 Data collection and processing

This section describes the various datasets required for this study, lists where they were obtained from and explains how they were used.

4.3.1 Topography

Topography data for the study area were obtained in the form of a digital elevation model (DEM) of the Cape Town metropolitan area, obtained from the CCT. The DEM has a 5 m horizontal and 1 m vertical resolution, and was used to create a TIN (triangular irregular network) surface using ArcGIS by interpolating between data points. While not more accurate than the original 5 m DEM, the TIN surface allowed for greater detail in the hydrological model. According to CHI (2020), a DEM of 5 m horizontal resolution is suitable for large catchment models such as the one created for this study.

Topography data were used primarily to delineate flow paths and slopes in the model (Section 4.4.1).

4.3.2 Rivers and waterbodies

Geographical information system (GIS) shapefiles containing data on the location and geometry of rivers, streams and waterbodies in the study area were obtained from CCT. These datasets did not require any processing.

River, stream and waterbody datasets were used with topography data to identify flow paths and areas of ponding/storage.

4.3.3 Stormwater network data

The existing stormwater network layout was obtained from CCT in the form of a GIS shapefile, which contained information on pipe diameters, start and end invert elevations and locations of culverts. The network records were largely intact, with occasional smaller pipes (< 600 mm

diameter) disconnected from the rest of the network. Where these pipes were near a watercourse, they were modelled to discharge into the watercourse; otherwise they were connected to the nearest pipe that was of equal size or larger in the downstream direction.

Stormwater network information (along with other data) was used to simulate flow in larger pipes in the model.

4.3.4 Land use data

Land use data for the catchment were obtained from CCT as a GIS shapefile. The shapefile was verified through site visits to the catchment and satellite imagery (from Google Earth). Where areas appeared to be misclassified, they were adjusted to the observed land use. Roads and golf courses were identified as significant areas with unique runoff characteristics; additional land use categories were thus created to account for these areas.

For this study, land use was required for two purposes: i) to estimate surface properties affecting runoff and ii) to determine the amount of pollutants washed off during rainfall events (Section 4.4.2 and Section 4.4.5 respectively).

4.3.5 Geological and soil data

GIS shapefiles containing the data on upper soil types and underlying geology in the study area were obtained from CCT. The soil shapefile was modified to include associated infiltration parameters.

The model requires data on soil characteristics to account for infiltration during and after storm events (Section 4.4.2).

4.3.6 Pollutant wash-off data

The amount of pollutants and nutrients washed off in storm events are usually represented by event mean concentration (EMC) values. According to DEFRA (2013), the EMC value for a specific pollutant is the flow-weighted mean concentration which, for a given event, is the total mass discharged divided by the runoff volume. Published EMC values representing wash-off from different land uses were obtained from various studies, as listed in Appendix C.

EMC values were required to estimate pollutant wash-off from the land uses present in the study area. Published EMC values were used in conjunction with field data to determine EMC values for model input (Section 5.5).

4.3.7 Stream level data

Stream flow data were initially only required for model calibration and dry weather baseflow estimation; long-term stream level data were obtained from CCT to fulfil this requirement. However, since CCT only operates one stream gauge in the study area, water level sensors were installed as part of this study to obtain additional data. Later in the study, data from the level sensors were also used in the EMC estimation process (explained in Section 5.5.1).

Development of the water level sensors and data processing of all water level data are explained in the following sections.

4.3.7.1 Long term data from CCT

The only long-term stream data available in the study area is from a CCT stream level gauge, KEYS05DR, located in the central region of the study area. This station measures water depth in the channel using an ultrasonic sensor, installed under the culvert where the river crosses the M3 freeway (Figure 4-5). According to CCT, a rating curve was not available for this site.

Manning's equation (Equation 4.1) was used to approximate volumetric flow from the water depth. The use of Manning's equation requires a prismatic channel of relatively constant roughness. This channel has a natural lining and is not prismatic throughout its length. Although the conditions were not ideal for the use of Manning's equation, it was used as there was no other long-term data to use for model flow calibration, and more detailed assessments that required additional site measurements were difficult to execute as these calculations were done during the initial Covid lockdown.

$$Q = \frac{A^{5/3} P^{3/2} S^{1/2}}{n} \quad (4.1)$$

where Q = flow (m^3/s); A = cross-sectional flow area; P = wetted perimeter; S = slope and n = Manning's roughness coefficient.



Figure 4-5: The Keysers River flowing underneath the M3 freeway through Dreyersdal
(Google Maps, 2022)



Figure 4-6: The Keysers River just upstream of the M3 culvert. The vegetation growth obscures the view of the culvert (Source: Geordie Thewlis)

4.3.7.2 Development of level sensors

Since stream level data was only available at one location throughout the study area, a part of this study was dedicated to the development and installation of ultrasonic flow level sensors to generate additional stream data. A programmer with experience in constructing sensors for river monitoring applications was consulted. The design and set-up of the required sensor was subject to the following considerations:

- Safety – the set-up had to be small or inconspicuous enough to be easily hidden or disguised to reduce the risk of it being stolen.
- Cost – the limited project budget necessitated a low-cost solution, thus precluding the selection of expensive components.
- Travel limits – a set-up that limited the amount of times required to travel to the monitoring sites was preferred; this would lower cost for the current and future projects that would utilise these sensors.

After a series of meetings with the programmer, the following set-up was selected: an ultrasonic sensor programmed with a low-cost MCU (microcontroller unit) using the GSM (global system for mobile communications) network to transfer data. The device was battery powered. This was chosen for the following reasons:

- Ultrasonic sensors are cost-effective, easy to install and can be set up to provide contactless readings (Abdelal & Al-Hmoud, 2021).
- Transferring data to a server using the GSM network, instead of storing it on the device itself, saves time and money by limiting trips to the monitoring sites and also ensures the data cannot be lost in the event that the device is stolen or tampered with.
- Batteries were the most convenient and cost-effective choice of power supply; although they needed to be recharged and replaced, they were a cheaper and safer option than solar panels, which would attract attention and be vulnerable to theft.

After consulting with various outlets, components were selected based on cost, availability and salesmen recommendations. Once all the components were gathered, the programmer assembled and coded the device.

The ultrasonic sensor was made up of a transmitter and receiver; the transmitter emitted ultrasonic sound waves at a known speed towards the water surface, which reflected them back to the receiver. The receiver used the speed and return time to calculate the distance to the water surface. Using a SIM card and the GSM module, this measurement was sent to a web server via a communications provider, using the GSM network (colloquially known as the cellular network). These processes were dictated by the microcontroller based on the original input

coding. The depth of the water in the channel was calculated by subtracting the reading given by the receiver from the distance between the sensor and the bottom of the channel.

For this study, sensor units were powered by a set of eight rechargeable lithium-ion batteries. The device transmitted readings every 30 minutes, entering sleep mode in between transmissions to preserve battery life. During initial tests, the batteries lasted 7-8 days before needing to be recharged. The components were arranged in a plastic container, with a hole drilled through the centre to allow the protrusion of the ultrasonic sensor. The arrangement of the components in the container is displayed in Figure 4-7. The containers were closed with plastic bolts and attached to concrete culvert soffits with a two-part epoxy adhesive.

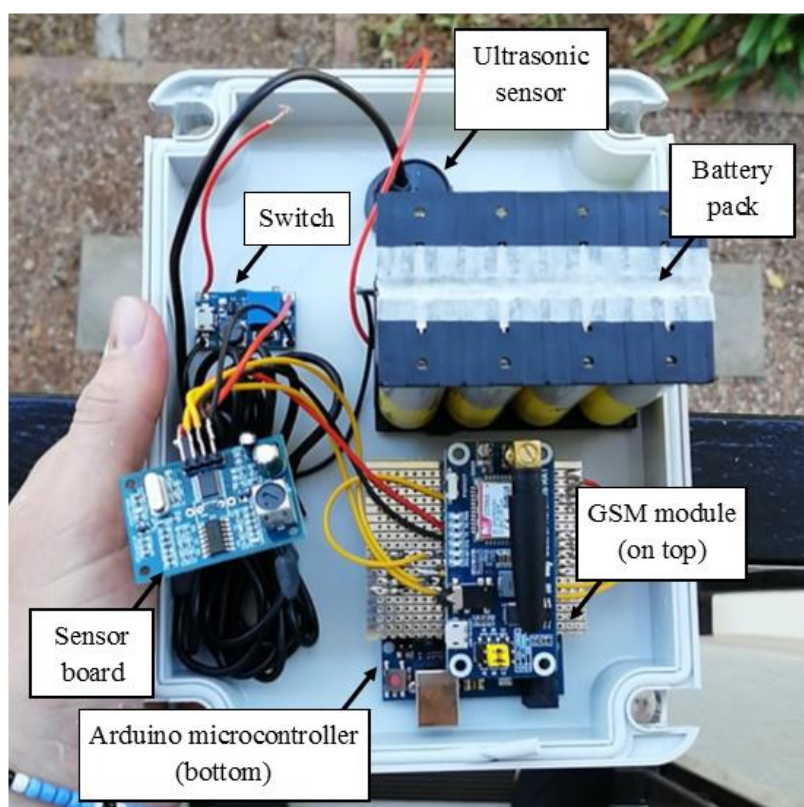


Figure 4-7: Flow sensor setup

Locations to place the sensors were chosen based on strength of network signal, safety and ease of access (to replace batteries) and availability of a mountable surface. Additional considerations included safety of the unit against theft and being washed away by flooding.

Water level sensors were placed at one location in the lower reaches of each river system. The Keyzers River sensor was placed underneath the Tokai Road bridge, just downstream of the river's intersection with Main Road (Figure 4-8), while the Westlake sensor was placed underneath the bridge that enters the Camargue housing complex off Kerner Close, just upstream

of Main Road. Both these locations were used as sample sites; their descriptors are Site K2 and Site W5 respectively. A full list and map of the sample locations are given in Section 5.2.1.



Figure 4-8: Replacing batteries in the Keysers River level sensor

Water levels were checked by measuring the water level with a custom-made metre stick (ruler) whenever the batteries were changed, and then comparing these values to the sensor readings recorded at the same time. Levels were within 20 mm for the Westlake sensor, but the Keysers sensor showed differences of up to 300 mm and sometimes the level did not change during a storm. The Keysers level data were deemed unreliable and therefore discarded. Unfortunately, the programmer was not available to help fix the sensor during the data collection phase. The level sensor locations are displayed in Figure 4-9.

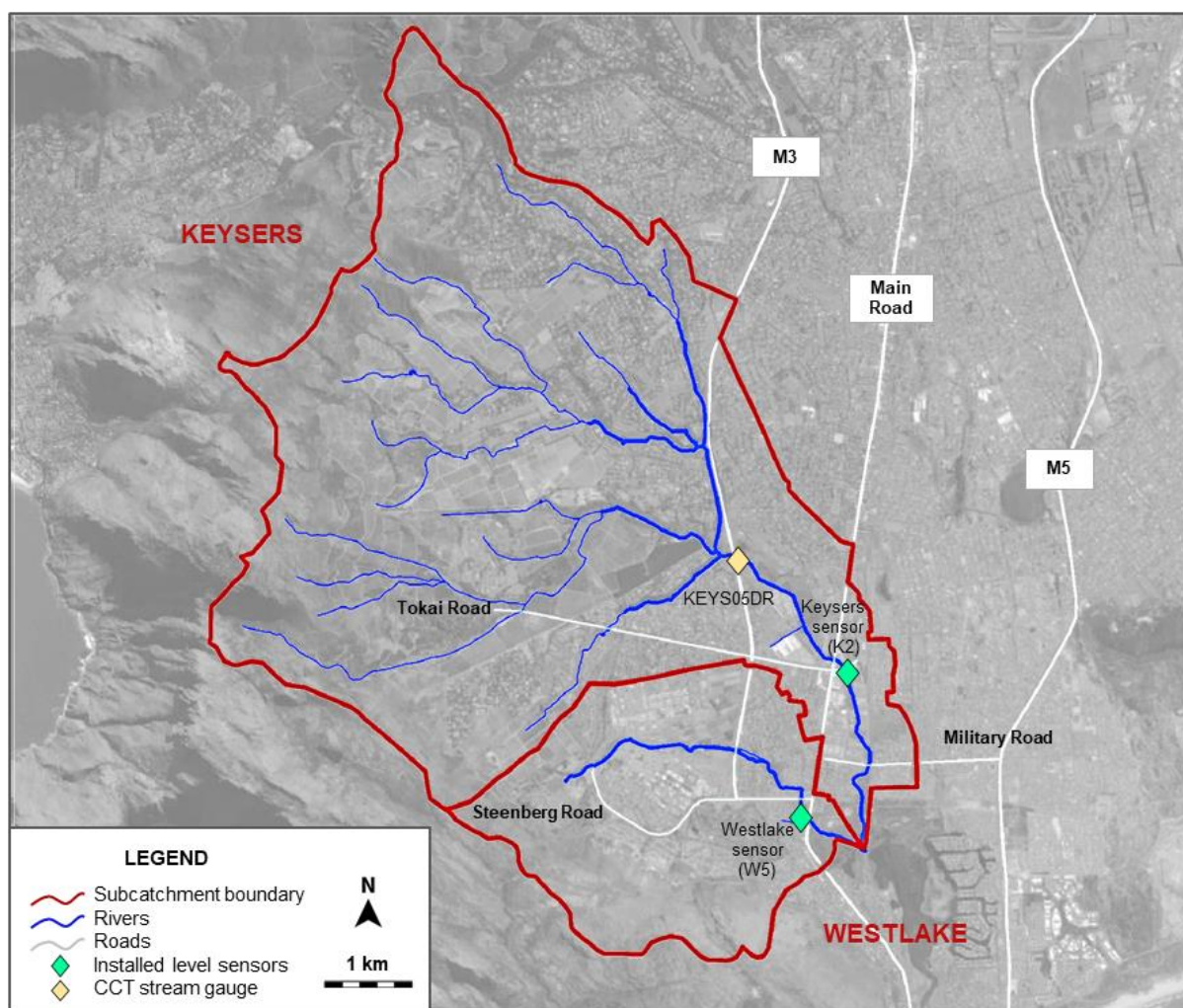


Figure 4-9: Location of level sensors and CCT stream gauge

4.3.8 Long term water quality data

Long-term water quality data were obtained from CCT in MS Excel format, comprising the same nutrients that were tested in the sampling programme, namely SRP, TAN and TO_xN . Spanning 2012-2019, the dataset gives concentrations of monthly grab samples taken at various monitoring sites across the study area. The data were used to identify long-term and seasonal water quality trends in the study area, to identify areas of elevated nutrient levels and thus guide the placement of SuDS in the model. Observed trends are discussed in Chapter 5.

4.3.9 Rainfall data

Rainfall data were obtained from three main sources: i) CCT records, ii) the Agricultural Research Council (ARC) and iii) private citizen records. Mean annual precipitation (MAP) data were also obtained from CCT GIS shapefiles. Continuous rainfall data were also acquired; CCT

monitors rainfall via several automatic gauges around Cape Town which record rainfall at 5-minute intervals. For this study area, rainfall records were available for a period of 8 years, from 2012-2019. However, only one gauge was situated inside the study area (KEYS05FR), while another was situated just outside the study area boundary (DIEP05AR). Since the catchment is known to have highly variable rainfall (CCT, 2010), more data was required.

Additional rainfall data were gathered from the ARC, which supplied daily rainfall data in Constantia (Groot Constantia) for the same time period (2012-2019), as well as local residents. Advertisements were placed in a local community newspaper in the study area (the *Constantia Bulletin*) requesting rainfall records. In response, 14 individuals graciously made their private rainfall data available. Of the 14, one record (Drostdy Road, Bergvliet) provided rainfall depths at daily intervals, while the rest were recorded at monthly intervals. Unfortunately, only the daily record and two of the monthly records (Vineyard Road and Yorkshire Close, in Bergvliet and Kirstenhof respectively) matched the CCT data in terms of record length (8 years). Details of the rain gauge data collected are shown in Appendix A.

Since the CCT data were of a finer resolution than the rest of the data, its length was used as the length of the modelling period. While longer records are preferred for continuous simulations records of these length are almost never available at the fine resolution (sub-hourly) that continuous modelling requires (James, 2005). Given what was available, the length of the records was deemed sufficient.

Rainfall data was required at a resolution of at least 15-minute intervals for analysis in PCSWMM. The CCT data satisfied this requirement, but the daily and monthly data required disaggregation before it could be used. Monthly data can only be disaggregated if daily/hourly data are available at the same location – unfortunately, this was unavailable. Instead, a rudimentary approximation of the triangular distribution disaggregation method, which is used to disaggregate daily data into hourly (Gao *et al.*, 2012), was used. This method distributes the monthly rainfall totals into days proportionally based on the daily record. The daily data were then disaggregated to hourly data using NetSTORM, a freely available stochastic disaggregation package. To disaggregate daily data, the software requires hourly/sub-hourly data of a station nearby in the same climatic region. Data from KEYS05FR was used to disaggregate the daily data at Groot Constantia and Yorkshire Close, while DIEP05AR records were used to disaggregate Drostdy and Vineyard Road. The hourly data were then disaggregated further into 15-minute intervals using PCSWMM's rainfall disaggregation tool. The rain gauges used in the simulation are displayed in Figure 4-10.

4.3.10 Temperature data

Temperature data was obtained from the ARC for the period 2012-2019. Long-term data consisting of the daily maximum and minimum temperatures was acquired for the Groot Constantia gauging station.

Temperature data was required to estimate evapotranspiration in the study area as evaporation data for the study area for the entire time period could not be found. Daily temperature data was used by the model to estimate annual evapotranspiration with the Hargreaves Formula (Hargreaves & Samani, 1982) using a built-in function, but without using the pan coefficient due to the aforementioned lack of data. While the acquired temperature data was unlikely to closely apply to entire study area, it was deemed a sufficient approximation.

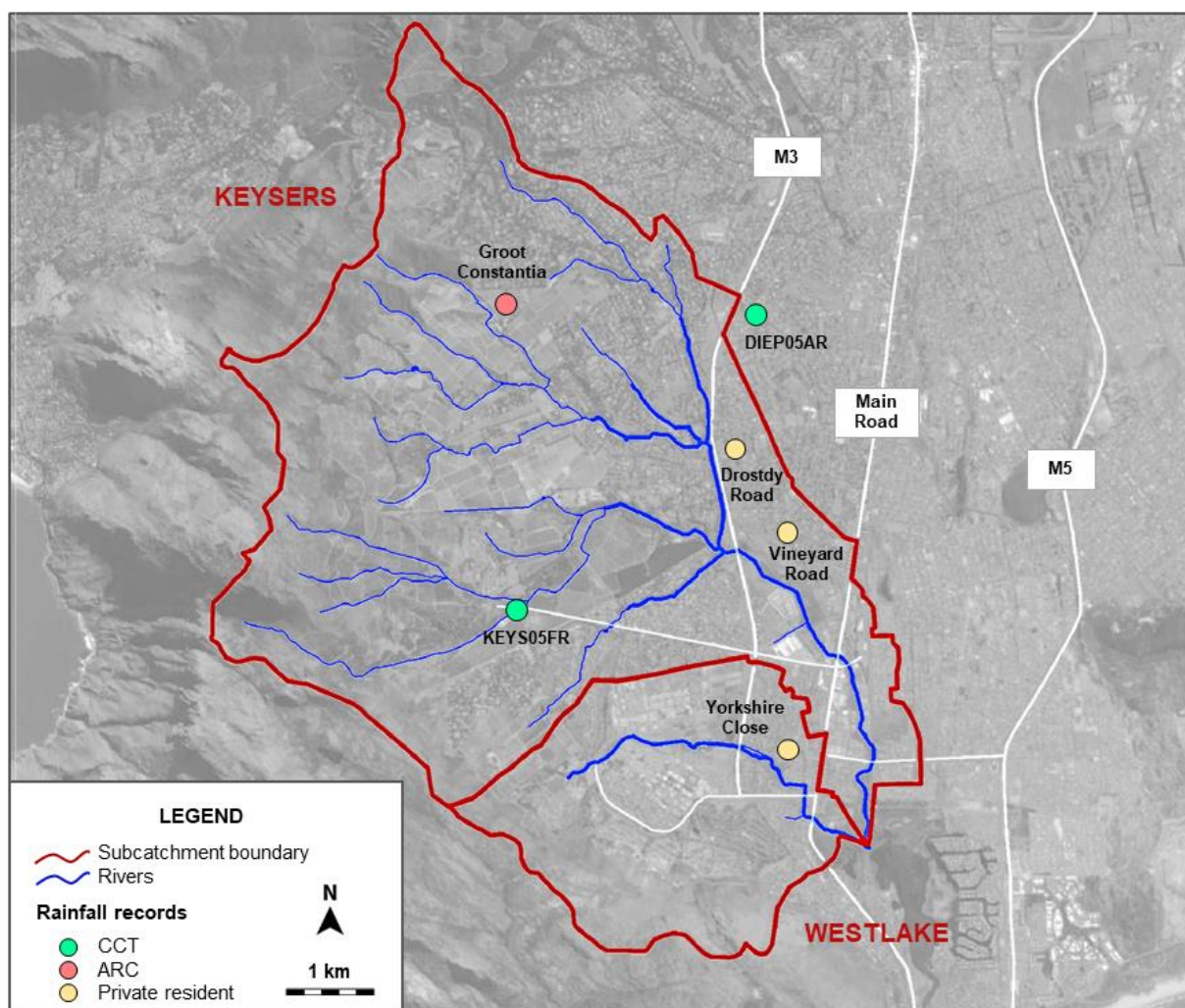


Figure 4-10: Locations of rain gauges used in the model

4.4 Hydrological model construction

Hydrological models represent complex real systems that can be used to understand current and historical conditions, as well as possible future scenarios (James, 2005). This study required the construction of a hydrological and hydraulic model to identify existing water quantity and quality conditions in the study area, and then to examine how implementing various SuDS interventions

can improve on current conditions. For ease of reading, the coupled hydrological, hydraulic and water quality models are referred to simply as ‘the model’.

The software package developed by Computational Hydraulics International (CHI), PCSWMM, was selected for the construction of the model. This was selected because it was familiar to the department in which this research was conducted and because student licences were easily and freely available. Additionally, the software has a user-friendly, integrated GIS interface that allows for easy importing of GIS data, which streamlines model development (James *et al.*, 2010).

PCSWMM uses the SWMM (stormwater management model) engine created by the USEPA, which can compute dynamic rainfall-runoff for both single event and continuous runoff. SWMM can simulate developed and undeveloped urban areas, as well as rural areas using various input data – typically information on land use, land cover, pervious areas, etc (Section 4.4.2).

The unit of analysis for a study area in PCSWMM is referred to as a sub-catchment; these are ‘hydrologic units of land’ that direct surface runoff to a single discharge point – either an element of the drainage system or another sub-catchment (James *et al.*, 2010). The two main components run by the engine are the runoff and routing components – the former operates on sub-catchments that each receive rainfall and produce (in the model) hydrographs and pollutographs, based on input rainfall data and pollutant concentrations. The routing component transports this runoff through various mechanisms – overland or underground through conduits, storage units, pumps, regulators or treatment devices (James *et al.*, 2010). SWMM accounts for infiltration based on input soil data and can also account for evapotranspiration if the required input data is available (evaporation, temperature and wind speed). Each simulation produces rainfall, runoff, generated pollutant loads, infiltration and evapotranspiration results per sub-catchment. Water quantity, flow rate and pollutant concentrations are produced for each conduit across the simulated time period (Rossman, 2015). SWMM can also model the effect of SuDS on runoff quantity and quality in a catchment using low-impact development (LID) controls and/or user-defined treatment equations (Rossman, 2015) – these are discussed further in Section 4.6. Some of the processes listed above are shown in Figure 4-11.

4.4.1 Sub-catchment delineation

Elevation data from the DEM, PCSWMM’s ‘watershed delineation tool’ was used to create sub-catchments, each with a defined flow path. The sub-catchments were then manually resized and refined based on existing catchment boundaries on maps (Jeffares & Green, 2003), supplemented by site observations of the existing drainage network. Flow paths delineated by the model were compared to known river lines (from the river shapefiles) to assess the accuracy of the DEM; the two largely coincided, affirming confidence in the DEM.

The final model contained 104 sub-catchments of varying size depending on the development density, topography and the existing stormwater network – as shown in Figure 4-12.

In the developed areas, sub-catchments ranged between 2-80 ha, while on the largely undeveloped mountain slopes in the eastern region they ranged between 30-300 ha.

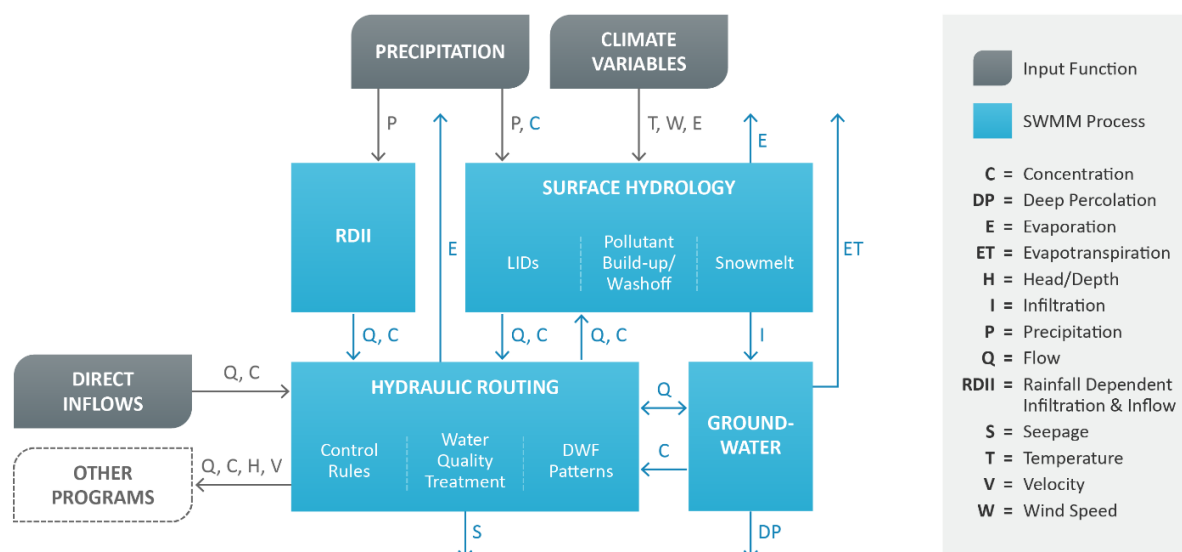


Figure 4-11: Schematic of processes in SWMM (CHI, 2018)

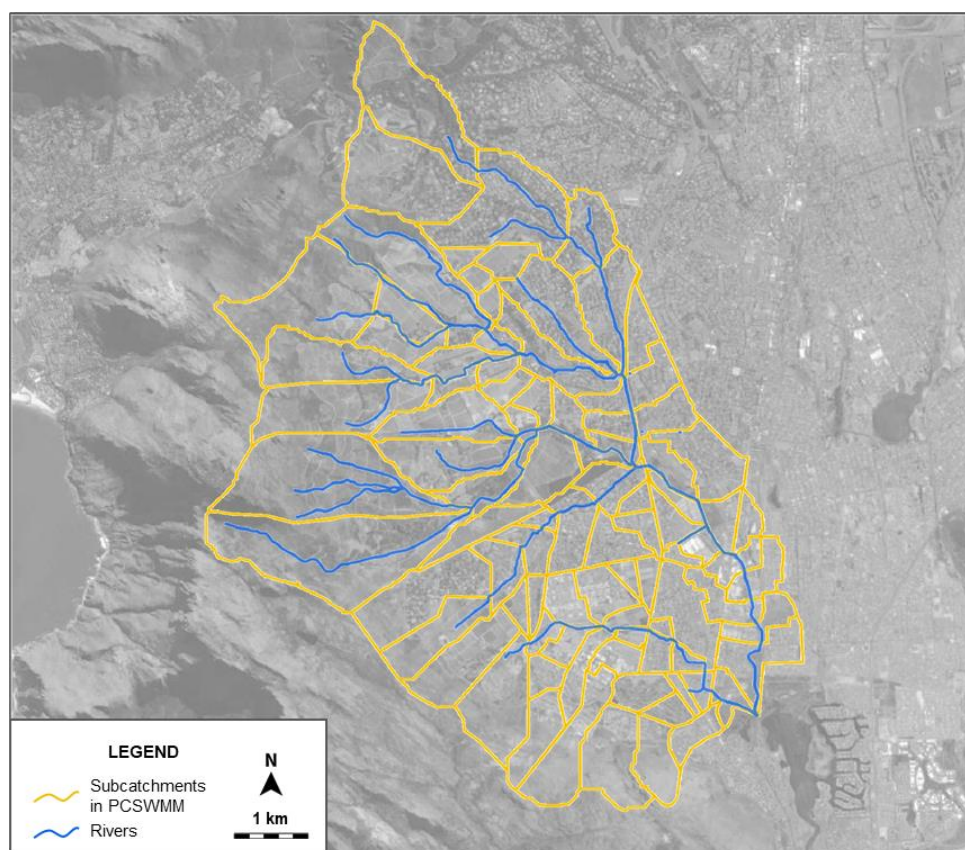


Figure 4-12: Sub-catchment delineation in the model

4.4.2 Sub-catchment parameters

Sub-catchments have various user-defined parameters that guide how the model simulates runoff and infiltration, such as the proportion of the sub-catchment that is impervious, surface roughness (Manning's n value), infiltration properties, etc. Some of these parameters vary with factors such as land use and soil type; the software allows the user to define these parameters based on land use and soil type, and then conduct an area weighting exercise on the sub-catchments to attain the relevant combination of the various parameters. These input parameters, and how they were determined, are listed in Table 4-1.

Table 4-1: Sub-catchment input parameters

Parameter	Determined by
Slope	Elevation data from DEM.
Area	Calculated using GIS functionality on PCSWMM.
Width	Calculated by PCSWMM by dividing the area by the effective flow path length of the sub-catchment.
Land use	The proportion of different land uses in each sub-catchment was area weighted from the modified CCT land use shapefile (Section 4.3.4). Used in conjunction with EMC values to calculate pollutant wash-off loads per sub-catchment.
Rainfall	Rainfall records were obtained and processed as described in Section 4.3.9. Each sub-catchment was assigned the nearest record.
Impervious area	Based on the literature/satellite imagery/site visits, these parameters were defined per land use category, and then area weighted to calculate composite sub-catchment values. Typical literature values are shown in Appendix D.
Depression storage	
Manning's n for overland flow	Defined separately for pervious and impervious surfaces for each land use category. These were based on values in the literature and area weighted to calculate composite sub-catchment values. Typical values are shown in Appendix D.
Infiltration parameters	Data on soil types were obtained from CCT (Section 4.3.5). Parameters were defined per soil type assuming the Green-Ampt method and area weighted to indicate composite sub-catchment values. Typical values are shown in Appendix D.
Routing of runoff within sub-catchment	Initially set to direct runoff to each sub-catchment outlet; during the calibration exercise, routing to pervious areas was found to match observed flow data better. The calibration method is discussed further in Section 4.4.4.

4.4.3 Conduits, nodes and storages

Conduits are elements that the model uses to represent the flow paths of water, either in a river, channel or pipe. Conduits are connected by nodes which function as connection points at which

flow can enter or leave a conduit, or where water can be stored. When sub-catchments are created in PCSWMM, flow paths are automatically delineated and represented as conduits.

The conduits created during the sub-catchment delineation process mostly aligned with existing rivers and streams; where they did not, they were edited to follow known stream paths. Additional conduits were then manually added to take existing channels and the stormwater drainage network into account. Including the entire stormwater network would have unnecessarily added to model complexity as increasing detail over-and-above a critical minimum increases the risk of increasing uncertainty and thus compromising model reliability (James, 2005). Considering the number of sub-catchments and the scale of this model, pipes with a diameter less than 600 mm were mostly omitted from the analysis; the exceptions were pipes in areas where SuDS were modelled. The input parameters required for conduits, and how they were estimated, are summarised in Table 4-2.

Table 4-2: Conduit input parameters

Parameter	Determined by
Length	Calculated using GIS functionality on PCSWMM.
Slope	Elevation data from DEM.
Manning's n for channels/ pipes	Pipe material data obtained from shapefiles provided by CCT (Section 4.3.3). Channel cover based on site visits and satellite imagery (Google Earth). n -value estimated using reference tables on the CHI support site (CHI, 2021).
Cross-section (for channels)	Regular cross-sections (rectangular or trapezoidal) were based on on-site measurement and the measuring tool on Google Earth. Irregular cross-sections were determined using PCSWMM's 'transect creator' tool which uses the DEM to determine channel sections.
Diameter (for pipes)	Based on GIS data obtained from the CCT.

Various ponds and dams in the study area were represented in the model as storage nodes. Storage nodes are points in the model with an allocated defined volume, based on a function of surface area versus height (James *et al.*, 2010).

4.4.4 Calibration and verification

Calibration of modelled output against observed data are an essential and important step in model construction (James, 2005). The model was calibrated against observed flow data from KEYS05DR, a CCT-operated stream gauge on the Keyzers River (Section 4.3.7.1). Data from this gauge and from the Westlake flow sensor were used in the verification process.

Prior to attempting calibration using PCSWMM's 'sensitivity radio tuning calibration' (SRTC) tool – which allows the user to assess the sensitivity of the model to changes in various

input parameters – a visual comparison was made between the modelled and observed hydrograph at KEYS05DR. There was a stark difference in the hydrograph shape – the modelled hydrograph exhibited steep recession curves while the observed graph displayed shallower recession curves and lower peaks. Shallow recession curves are typically caused by subsurface flow, indicating that the stream flow is fed from deep subsurface storage and/or delayed shallow subsurface storage between rainfall events (Price, 2011; Jakada *et al.*, 2019). Until this point, modelling parameters were limited to overland and pipe flow; however, articles on CHI's open-source online knowledge-sharing portal, OpenSWMM, indicated that surface flow parameters are not well equipped to produce slower falling limbs on stream flow hydrographs (CHI, 2020b). This assertion was tested by undertaking an exhaustive sensitivity analysis testing all the sub-catchment and conduit input parameters. No change in these parameters could produce the flatter recession curves of the observed data, suggesting that the model would need to consider subsurface flow as well.

4.4.4.1 Accounting for subsurface flow

Experienced PCSWMM users on OpenSWMM suggested using PCSWMM's groundwater module to model subsurface flow to attain flatter recession curves (CHI, 2014a; CHI, 2020b). The groundwater module uses an aquifer function and the following groundwater flow equation (Equation 4.2) to account for delayed flow:

$$Q_{GW} = A1(H_{GW} - H^*)^{B1} - A2(H_{SW} - H^*)^{B2} + A3(H_{GW}H_{SW}) \quad (4.2)$$

where Q_{GW} = groundwater flow (m^3/s per ha); H_{GW} = height of saturated zone above bottom of aquifer (m); H_{SW} = height of surface water at receiving node above aquifer bottom (m); and H^* = threshold groundwater height (m).

In the groundwater equation, variables are either user-defined or automatically assigned by the model based on sub-catchment parameters; however, information on groundwater characteristics and water table levels for the study area was not available. Aquifer characteristics were assumed based on the recommendation that modelling shallow aquifers accounts for subsurface flow (CHI, 2010). Since the lower reaches of the study area are near the Cape Flats aquifer and underlain by the same geology (CCT, 2010), they were assumed to have 1-metre deep aquifers, extending toward the middle reaches. Further assumptions regarding the groundwater and aquifer input parameters are shown in Table 4-3.

In addition to utilising the groundwater module, subsurface flow was also accounted for by changing the routing option for sub-catchments from 'route flow from impervious areas to sub-catchment outlet' to 'route flow to pervious areas' – this change allows for more infiltration and thus greater subsurface flow.

4.4.4.2 Final calibration

The model was calibrated against flow data from KEYS05DR. From the 15 storm events identified, 10 were used for calibration and five for verification. Splitting the storms thus follows the ratio of number of storms used for calibration to verification of 2:1 in line with other studies (Mancipe-Munoz, *et al.*, 2014).

Table 4-3: Groundwater and aquifer input parameters

Parameter	Determined by
Aquifer input parameters	
Porosity	Based on soil type. Soil type data from CCT shapefile (Section 4.3.5); soil characteristics from Rawls <i>et al.</i> (1983). Typical values are presented in Appendix D.
Wilting point	
Field capacity	
Hydraulic conductivity	
Initial moisture deficit	
Bottom elevation	Varied with topography; based on recommendations by CHI (2010) to specifically model shallow subsurface flow.
Water table elevation	
Conductivity slope	Based on suggestions from CHI (2008). Was subjected to sensitivity analysis, but model was found to be largely insensitive to changes in these parameters.
Tension slope	
Lower groundwater loss rate	
Groundwater equation inputs	
Coefficients A1 – A3	Based on recommendations by CHI (2014b). Was subjected to sensitivity analysis; these parameters were found to greatly influence the slope of recession curves. Calibrated values are listed in Appendix D.
Exponents B1 – B2	
H _{SW} , H _{GW} , H*	Calculated by the model based on sub-catchment and aquifer input parameters.

While calibration is a vital part of any modelling process, there is no standard for the adequacy of calibration (James, 2005). In this investigation, the amount of data available considering the scale of the model was the main factor that affected the standard of calibration – particularly the following aspects:

- Sub-hourly rainfall data were only available for one rain gauge, KEYS05FR, in the study area. While daily data were available at other locations, the disaggregation of this data

(Section 4.3.9) is not representative of actual rainfall events; rather, it represents rainfall that could statistically have occurred.

- Long term stream level data were limited to one location in the study area.
- Stream flow data was estimated using the Manning equation; however these values are a rough approximate as the site was not ideal for the application of the Manning equation (Section 4.3.7.1)
- There was limited groundwater and aquifer data available for the study area; using the groundwater module in the model demanded many assumptions (Section 4.4.4.1).

The aspects listed above all contribute to model uncertainty. It is difficult to quantify the error that each aspect introduces to the model as it requires enough data to characterise the uncertainties in the input data (James, 2005) which was beyond the scope of this research.

Sensitivity analysis was once again undertaken, based on recommendations by James (2005). During a sensitivity analysis, each parameter is assigned an uncertainty percentage. The model then completes a specified number of runs (the user can choose between two, four or eight), changing the parameter in question by a factor of the assigned uncertainty percentage. For example, if total flow volume was assigned an uncertainty of 10% and two model runs were chosen, the model would run with area 10% greater and 10% less than its current value (for each sub-catchment). If four runs are chosen, the model changes the area by +10 %, -10% (as before) as well as +5% and -5%, with the model interpolating results between these points. Uncertainty percentages were assigned based on recommendations by James (2005).

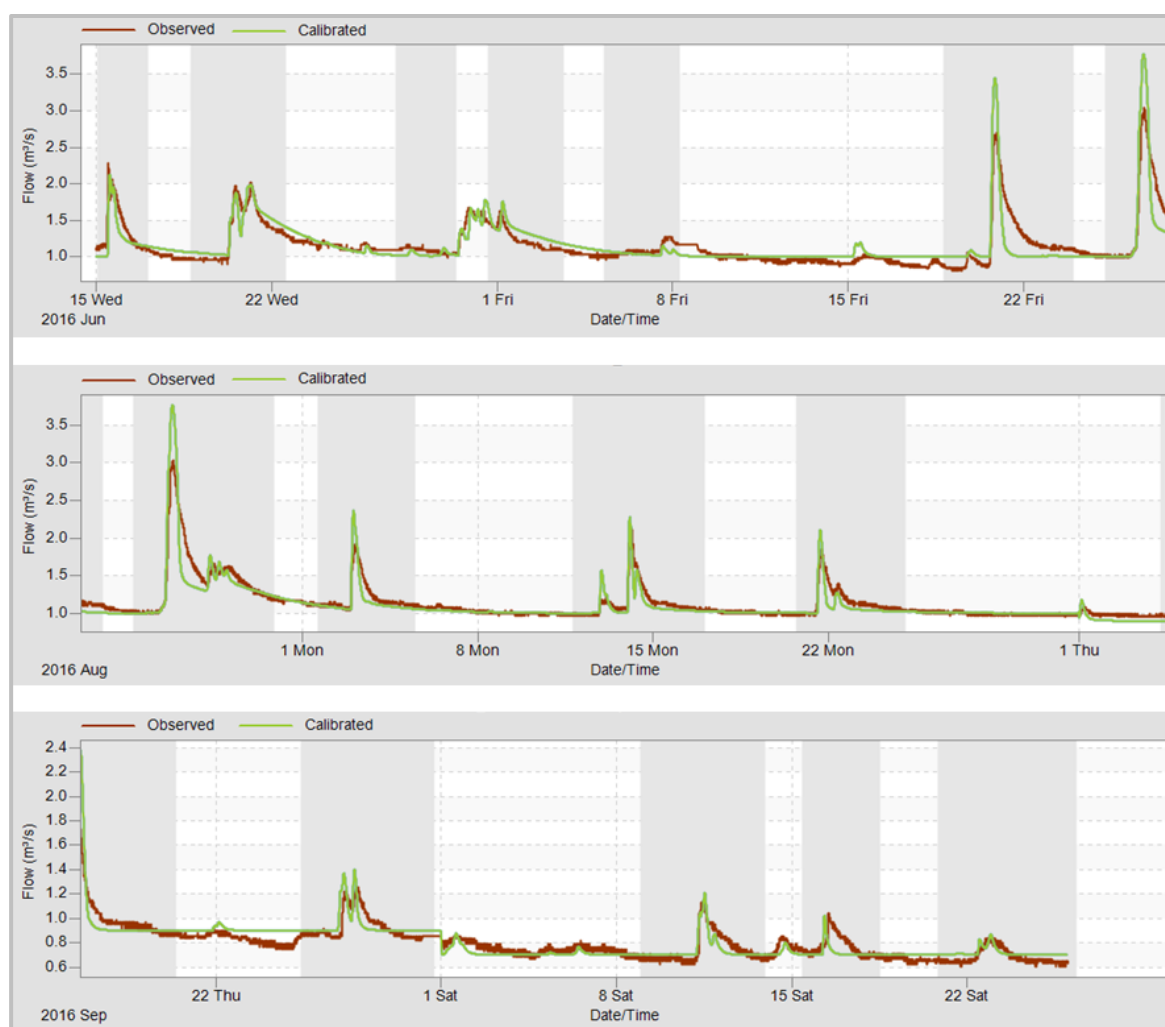
The model was found to be sensitive to the following parameters: sub-catchment width, impervious proportion of the sub-catchment, percentage of the flow routed to pervious areas, depression storage in pervious areas and the groundwater equation coefficients A1, A2, B1 and B2. Calibrated values were checked to ensure their changes fell within acceptable ranges as defined by James (2005), with the exception of the groundwater equation coefficients, for which no data were available for comparison.

The calibration exercise considered the following parameters: total flow volumes, peak flow volumes and storm hydrograph shape. The calibration was evaluated based on the Nash-Sutcliffe efficiency (NSE) and coefficient of determination (R^2) error functions. The NSE ranged between $-\infty$ and 1, while the R^2 values ranged between 0 and 1 – with 1 being the optimal value for both. According to Moriasi *et al.* (2007) values above 0.5 for both statistics are satisfactory for watershed models. The calibrated and verified results largely displayed error functions larger than 0.7 and were thus deemed acceptable.

A summary of the calibration and verification error functions is displayed in Table 4-4, followed by graphical output from PCSWMM showing the observed and calibrated hydrographs for the calibration storms (Figure 4-13).

Table 4-4: Calibration of total flow, peak flow and storm hydrographs

Parameter	Error function	Calibrated	Verified
Total flow	NSE	0.95	0.91
	R ²	0.96	0.93
Peak flow	NSE	0.84	0.86
	R ²	0.87	0.88
Hydrograph	NSE	0.77	0.75
	R ²	0.81	0.79

**Figure 4-13: Hydrograph of the observed and calibrated storm data**

4.4.5 Input of EMC data

The model uses EMC data to estimate pollutant loads washed off during storm events. EMC data are typically expressed as the concentrations of various pollutants washed off a particular land use type. Using EMC values adds to the uncertainty in the model since a single value represents wash off from all storms while runoff quality actually varies with storms and over seasons (Schiff & Tiefenthaler, 2011). Furthermore, EMC data for South Africa are limited; no data could be found for the study area or the surrounding region. Literature values from other parts of the world were thus used.

For this study, composite EMC values were created by adjusting values from the literature based on nutrient concentrations measured at Site W5 (this is explained further in Section 5.5). The outcome of this process was a set of EMC values for each land use category in the study area, as shown in Table 4-5.

Table 4-5: Final EMC values used in the model (in mg/l)

Land use	Final EMC	Land use	Final EMC
TIN		SRP	
Residential	1.43	Residential	0.31
Golf course	2.41	Golf course	0.24
Industrial	1.34	Industrial	0.26
Agricultural	2.14	Agricultural	0.65
Commercial	1.53	Commercial	0.26
Open space	1.98	Open space	0.10
Freeways	2.00	Freeways	0.23
TP		TSS	
Residential	0.50	Residential	125
Golf course	1.07	Golf course	202
Industrial	0.34	Industrial	226
Agricultural	2.14	Agricultural	151
Commercial	0.33	Commercial	128
Open space	0.41	Open space	486
Freeways	0.29	Freeways	132

4.5 Development of SuDS scenarios

One of the objectives of this study was to investigate the effect that SuDS interventions would have on water quality in the study area. This section explains the factors that influenced the choice of SuDS measures, their placement, and their simulation.

4.5.1 SuDS selection and placement

In this study, the greatest factors influencing SuDS selection and placement were the distribution of poor water quality, the availability of open space where SuDS might fit, and existing resources such as wetland areas.

Typically, SuDS are designed as treatment trains, usually consisting of a mixture of source, local and regional controls (see Figure 2-3). SuDS philosophy encourages the use of on-site controls as the best first line of defence (Armitage *et al.*, 2013). However, source controls implemented at private residence level, such as green roofs and rainwater harvesting, are difficult to model at the scale of this project; thus the modelling of SuDS measures was focused on local and regional controls.

The first step in selecting SuDS measures was locating areas with sufficient space, and comparing those locations to where SuDS measures were needed – ideally downstream of suspected sources of elevated nutrients.

After areas were selected for SuDS modelling, the choice of potential SuDS measures was determined by what would be suitable considering the space available. For example, long narrow vegetated areas are suitable for swales. These initial choices would then be refined based on other site characteristics – in the previous example, even if a swale fits in an area, the site would be unsuitable if the slope is too steep since swales require slower runoff velocities to encourage infiltration and prevent erosion.

Once a SuDS component was selected, it was sized (Section 4.5.2) and then modelled (Section 6.3). In the model, SuDS were checked to ensure they adhered to the basic design standards and produced ‘realistic’ pollutant reductions – such as expected SuDS removal percentage ranges from Armitage *et al.*, (2013) Table 2-3). Where modelled results appeared unrealistic, the model was checked for errors; the errors were typically found to be disconnected nodes in the model. After each SuDS component was modelled with the identified errors corrected, different combinations of SuDS were modelled as scenarios. Figure 4-14 summarises the steps leading to modelling the SuDS scenarios.

The following scenarios were created in the model:

- **Current Scenario:** the model was set up using data that represents the current catchment (Section 4.4). The Current Scenario was used to gain an understanding, however approximate, of the existing water quantity and quality in the study area. This scenario was also used as a baseline to compare the relative pollutant removal of the various SuDS scenarios. The Current Scenario is further discussed in Section 6.1.
- **Pre-development Scenario:** The Pre-development Scenario was used to give an indication of how current water quantity conditions differ from what they might possibly have been before the area was developed and urbanised. One of the goals of the SuDS philosophy is to control peak runoff rates so that the rate of runoff is limited to what it would have been

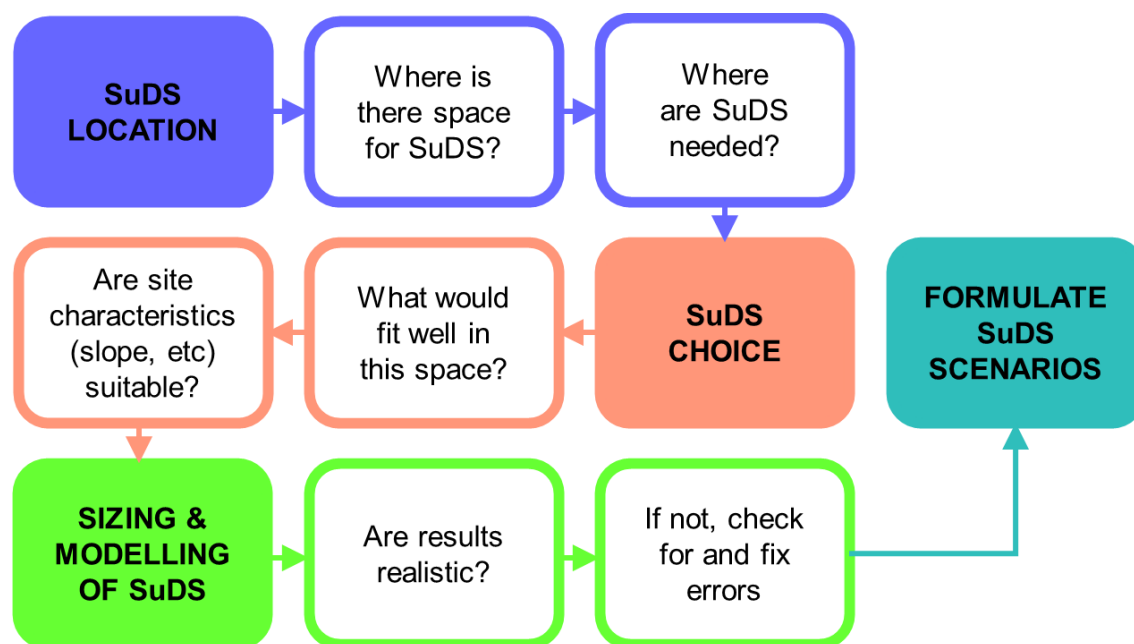


Figure 4-14: Brief process leading up to formulating SuDS scenarios

prior to development (Woods Ballard *et al.*, 2015). The Pre-development Scenario is further explained in Section 6.2.

- **SuDS Scenarios:** Individual SuDS components were chosen and modelled as described in the paragraphs above. In locations where different SuDS options were modelled, the one with the greater pollutant removal was selected to be grouped in a scenario. SuDS scenarios were then created from various groupings of these individual SuDS components. The different scenarios and how they were grouped are discussed further in Section 6.3.

4.5.2 Sizing of SuDS

For this study, SuDS were sized based on the water quality volume (WQV) required for treatment; the aim of modelling SuDS in the study area was to assess pollutant removal capability rather than reduce flow volumes, since the area is already equipped with a functional drainage system (this would not be the case for new developments). The WQV is defined as the volume of water from smaller storm events where the focus is on water quality treatment (Armitage *et al.*, 2013). WQV can be estimated in a variety of ways; for this study, the WQV was the runoff associated with the 1 in 6 month, 24-hour storm – based on recommendations by Armitage *et al.* (2013).

Rainfall frequency data for 24-hour storm durations were obtained from the CCT (Section 4.3.9). The dataset contains rainfall depths for 2- to 200-year recurrence interval (RI) storms for various locations throughout the study area. These data points were plotted as a

logarithmic function for each location, from which the 1 in 6 month, 24-hour storm depth was extrapolated for each location – giving rainfall depths that varied between 24-47 mm across the study area. SuDS components were sized to store the volume associated with this storm depth. Based on the eight years of data that was collected, this depth is greater than that associated with the first significant rainfalls of the wet season which typically flush pollutants down the catchment – i.e. the modelled SuDS measures will be theoretically able to accommodate the first flush rainfall.

The SuDS components were sized according to specifications outlined by Woods Ballard *et al.* (2015) and Armitage *et al.* (2013). The modelled SuDS measures are described in Section 6.3, and details on the relevant design standards and equations, as well as diagrams, are displayed in Appendix E.

4.5.3 SuDS treatment objectives

The study area falls within the jurisdiction of the CCT Municipality who have developed a policy framework regarding stormwater management objectives (CCT, 2009). This document outlines criteria that any implemented SuDS intervention must adhere to; the stipulated water quality objectives are a 45% reduction in TP and an 80% reduction in TSS.

South African aquatic guidelines classify the eutrophic status of rivers based on the average summer concentrations of inorganic phosphorus and inorganic nitrogen, over at least 4 weeks (DWAF, 1996). This data was not available, therefore the eutrophic status of the rivers in the study area could not be classified. In terms of SRP and TIN removal, the objective was for modelled SuDS to reduce SRP and TIN concentrations to below the eutrophic threshold.

4.6 SuDS simulation

4.6.1 Modelling methods in PCSWMM

Source and local controls, such as green roofs, swales, infiltration trenches and bioretention areas, can be modelled using the LID controls function in PCWMM. ‘LID controls’ is an umbrella term for a variety of blue-green technologies developed to reduce the impacts of stormwater runoff in cities (Khurelbataar *et al.*, 2021), and is the term used for SuDS in the USA. In the model, LID controls capture and treat runoff through simulating the processes of detention, infiltration and evapotranspiration (James *et al.*, 2010).

The available LID controls do not include all types of SuDS interventions, so SWMM is unable to simulate certain measures – such as wetlands and ponds – using LID controls. On the other hand, wetlands can be modelled as storage facilities with user-defined treatment equations to simulate the removal of various pollutants (Rossman & Huber, 2016). These treatment equations can take on various forms: for example, the first-order decay method was made popular for wetland treatment by Kadlec and Knight (1996). Known as the k-C* method, it employs the

use of empirically determined decay coefficients (k) and background pollutant concentrations (C^*) to estimate pollutant removal. Decay coefficients are highly specific to the wetland from which the coefficient was derived and are dependent on many other variables, such as hydraulic loading and inlet pollutant concentrations (Kadlec, 2000). According to Kadlec and Wallace (2008), these models “purport to describe general features of complicated processes” and they discourage the use of k and C^* values in conditions different to the operating values from which they were derived.

For this study, wetlands were modelled using decay functions derived from a variety of studies (Huang *et al.*, 2000; Akrotos & Tsihrintzis, 2007; Abbassi *et al.*, 2011; Alayu & Leta, 2020). These functions give the fraction of each pollutant remaining as a function of hydraulic residence time (HRT). Some studies have shown that increased HRT results in greater pollutant removal in constructed wetlands, including TN, TP and TSS (Alayu & Leta, 2020). Other studies found that while increased residence times provided greater nitrogen and suspended solids removal, phosphorus removal was unaffected (Akrotos & Tsihrintzis, 2007; Abbassi *et al.*, 2011; Toet *et al.*, 2005). More specifically, Huang *et al.* (2000) found that ammonia and Total Kjeldahl Nitrogen (TKN) decreased exponentially with increased retention time, also noting that vegetation type had little influence on nitrogen removal.

4.6.2 SuDS simulation in the model

The SuDS interventions were modelled in this project in one of two ways: i) local controls such as swales and bioretention areas were modelled using PCSWMM’s LID controls tool and ii) regional controls such as wetlands were modelled as storage units with pollutant removal defined by equations.

Creating LID controls requires the input of basic geometry of the control (such as berm heights, surface area, length) and the amount of vegetation. Controls like infiltration trenches and bioretention areas require additional inputs like soil and underdrain data. The model uses this information to calculate the effect each control has on water quantity and quality. More detail on the design of LID controls is given in Section 6.3, which describes each SuDS scenario and its components.

The treatment in wetlands was modelled as a function of HRT; pollutant removal in wetlands is strongly influenced by HRT (Section 4.6.1), with longer retention times generally producing greater pollutant removal. Wetland treatment is wetland-specific and affected by many other factors besides HRT – such as temperature, type and/or amount of vegetation and hydraulic loading rate (Lee *et al.*, 2009). The derived equations represent specific wetlands and do not account for other factors that influence pollutant removal – in a project of this scale, it would not be possible to account for all these factors. These equations thus offer only an approximation of what the treatment potential could be, in order to compare the relative scenarios. Each equation resulted in a different removal capacity per pollutant; the equations resulting in the lowest and

highest treatment for each pollutant are displayed in Table 4-6, described as lower and upper treatment limits.

Table 4-6: Treatment equations used to model wetland pollutant removal
(Huang *et al.*, 2000; Akrotos & Tsihrintzis, 2007; Abbassi *et al.*, 2011; Alayu & Leta, 2020)

Pollutant	Treatment equations	
	Lower limit	Upper limit
SRP	$R = 1 - e^{-0.0048 * HRT}$	$R = 1 - e^{-0.017 * HRT}$
TIN	$R = 1 - e^{-0.004 * HRT}$	$R = 1 - e^{-0.012 * HRT}$
TP	$R = 1 - e^{-0.0043 * HRT}$	$R = 1 - e^{-0.012 * HRT}$
TSS	$R = 1 - e^{-0.013 * HRT}$	$R = 1 - e^{-0.03 * HRT}$

The following pollutants were modelled:

- **SRP and TIN:** These pollutants are used as an indication of the eutrophic status of a waterbody (DWAF, 1996), and
- **TP and TSS:** The City of Cape Town Metropolitan Municipality (CCT) specifies target pollutant removals for SuDS development in terms of these pollutants (CCT, 2009).

4.7 Summary

The methods employed in this study comprised various components, ranging from field work on rainy days to running 24-hour computer simulations. The steps followed can be summarised as follows:

- Conducting a desktop study gathering information about the problems in the Zandvlei catchment. Elevated nutrient levels were identified as a major challenge as the eutrophication it causes has far-reaching detrimental effects on human and aquatic health and biodiversity.
- Undertaking water quality sampling to briefly identify potential sources of elevated nutrient concentrations;
- Gathering data on the catchment to assist in the construction of a hydrological model. Requirements included data on topography, river locations, stormwater drainage

infrastructure, land use, geology and soil, pollutant wash-off, water quality, rainfall and stream levels;

- Creating a model that represents the current state of the catchment, including calibrating it against observed stream flow data; and
- Simulating different individual SuDS components and formulating various SuDS scenarios to compare their respective pollutant removal capabilities.

5. Water quality in the study area

This chapter describes the water quality in the study area and compares it to South Africa's aquatic water standards (Section 2.2) with particular reference to eutrophication-inducing nutrients. The analysis is based on the results of the sampling programme undertaken as part of this study as well as long-term data obtained from CCT.

5.1 South African water quality standards

The South African Water Quality Guidelines for Aquatic Ecosystems (DWAF, 1996) describe eutrophic conditions in waterbodies in relation to inorganic phosphorus and inorganic nitrogen concentrations. As mentioned in Section 2.2, eutrophic conditions are expected when the mean summer concentration of inorganic phosphorus is between 0.025-0.25 mg/ℓ or the average summer concentration of inorganic nitrogen is between 2.5-10 mg/ℓ. Values exceeding the given ranges result in hypertrophic conditions (Table 2-2).

Insufficient data were available to compare average summer concentrations to these ranges; since only grab samples were measured, the study can only suggest where concentrations are more likely to induce eutrophication.

Data representing total inorganic phosphorus concentrations were not available; the analysis instead considers only dissolved inorganic phosphorus, represented by orthophosphates, also called SRP (Watson *et al.*, 2018). Total inorganic nitrogen (TIN) comprises ammonia (NH₃-N), nitrites and nitrates (NO_x-N). Sufficient nitrogen data were available to examine the TIN concentrations of the grab samples.

5.2 Water quality sampling and testing

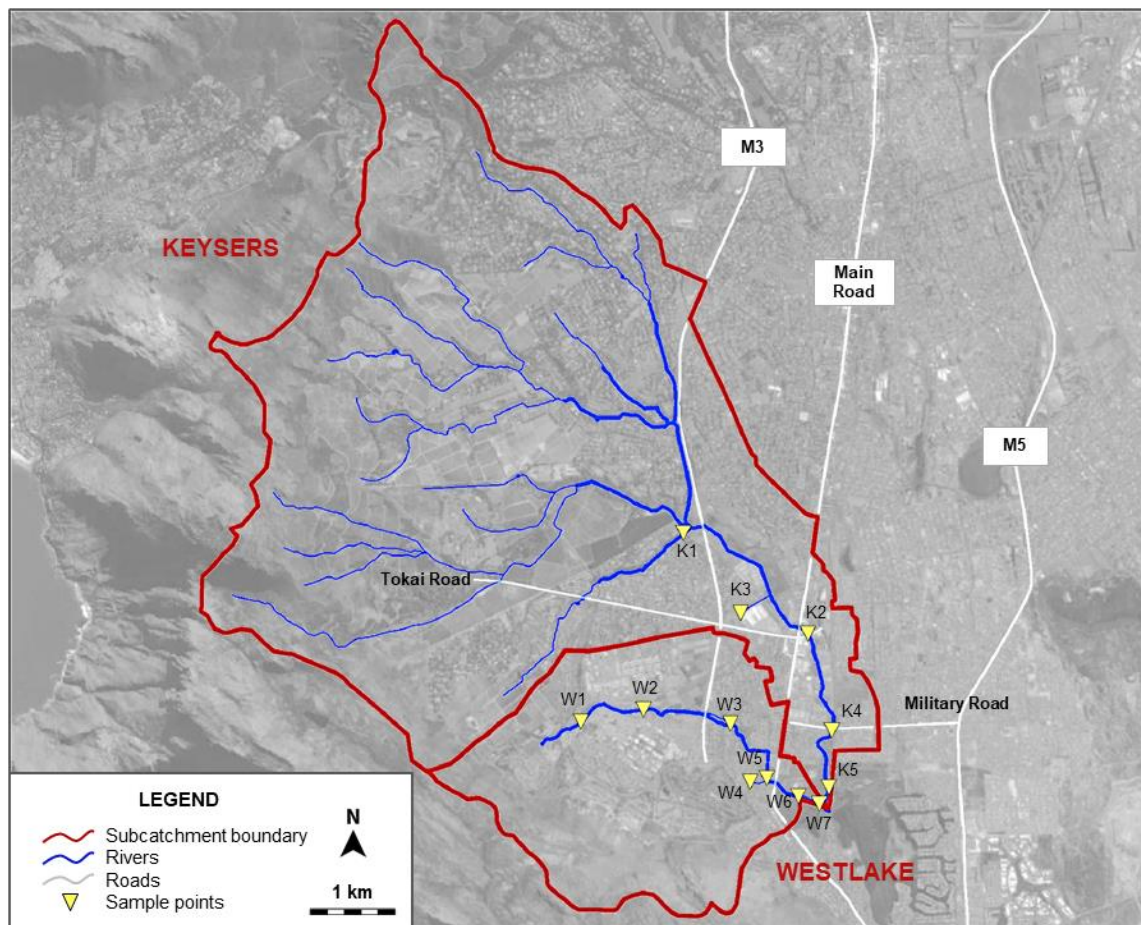
Water quality sampling was undertaken between June and December 2019, as detailed in Section 4.2. Preliminary testing was followed by two phases of sampling; the results of these stages is described in the following sections.

5.2.1 Sampling locations

Sampling locations were chosen as described in Section 4.2.2. The sampling locations for each phase are described in Table 5-1, and shown in Figure 5-1. As discussed in Section 4.2.2, Phase 1 had fewer sampling locations than Phase 2 due to low EC readings.

Table 5-1: Sampling locations for Phase 1 and 2

Sampling point		Location	Phase 1	Phase 2
KEYSERS	K1	Prinseskasteel River off Beatrice Close		x
	K2	Tokai Road (before bridge); flow level sensor site	x	x
	K3	Stream behind Blue Route Mall, adjacent to Access Road		x
	K4	Military Road (after bridge)	x	x
	K5	Just upstream of confluence		x
WESTLAKE	W1	Just downstream of US Consulate, at Reddam Avenue		x
	W2	In the field downstream of Westlake Primary School		x
	W3	Kirstenhof Greenbelt off Oranje Rd	x	x
	W4	Stream on Orchard Park draining the Westlake Golf Club	x	x
	W5	Kerner Close; flow level sensor site	x	x
	W6	Downstream of Main Road		x
	W7	Just upstream of confluence		x

**Figure 5-1: All sampling locations (for both Phase 1 and 2)**

5.2.2 Sampling results

The results of both phases of the brief sampling programme are shown and compared below. Findings are described in terms of the nutrients of interest – phosphates and nitrogen – and insights gathered from sampling through storms. Photographs of some sample points are shown in this section; sites that are not shown here are displayed in Appendix H.

5.2.2.1 SRP

Almost all samples taken displayed eutrophic or hypertrophic SRP levels. Most of the summer SRP measurements fell within the range of the winter levels, suggesting there is not much seasonal difference in concentration. Table 5-2 compares the minimum, maximum, median and mean SRP concentrations.

Table 5-2: SRP concentrations measured in Phase 1 and 2

Sampling point	Winter (wet season)					Summer (dry season)				
	Samples	Min	Max	Median	Mean	Samples	Min	Max	Median	Mean
K1	-					1 ³	1.75	1.75	1.75	1.75 ²
K2	4	0.08	0.24	0.13	0.15 ¹	1	0.02	0.02	0.02	0.02
K3	-					1	0.04	0.04	0.04	0.04
K4	2	0.09	0.11	0.10	0.10	1	0.05	0.05	0.05	0.05
K5	-					1	0.03	0.03	0.03	0.03
W1	-					1	0.05	0.05	0.05	0.05
W2	-					1	0.04	0.04	0.04	0.04
W3	3	0.03	0.2	0.07	0.10	1	0.07	0.07	0.07	0.07
W4	3	0.02	0.21	0.19	0.14	1	0.06	0.06	0.06	0.06
W5	4	0.06	0.42	0.14	0.19	1	0.10	0.10	0.10	0.10
W6	-					1	0.28	0.28	0.28	0.28
W7	-					1	1.52	1.52	1.52	1.52

¹ Figures shaded in orange indicate mean levels in the eutrophic range

² Figures shaded in red indicate mean levels in the hypertrophic range

³ SRP could not be tested in the second week of summer sampling due to a shortage of the required reagents

With eutrophic readings at almost every site, the entire study area is a concern. The sites with the highest measured concentrations were K1 (1.75 mg/ℓ) and K4 (0.05 mg/ℓ) in the Keysers system, and W7 (1.52 mg/ℓ), W6 (0.28 mg/ℓ) and W5 (0.1 mg/ℓ) in the Westlake River. All these fell within the hyper- or eutrophic band based on the SA aquatic guidelines.

Site K1 (Figure 5-2), in the Prinseskasteel River in the middle reaches of the Keyzers system, drains Tokai and upper Steenberg, which comprise residential, agricultural and commercial regions and the Steenberg Golf Club. Raised nutrient levels could be caused by the golf club, which makes use of treated effluent for irrigation (Smith, 2019), or the vineyards upstream of the golf course. K4 is located in the lower reaches, downstream of industrial and commercial areas.



Figure 5-2: Site K1 on the Prinseskasteel River, just upstream of its confluence with the Keyzers River (downstream toward the right)

W5, W6 and W7 are the three most downstream sites in the Westlake River; from these three sites, the SRP concentration increases at each successive site – suggesting that the greater sources of SRP are in the lower reaches. W5 drains residential parts of Westlake and Steenberg Road. W6 drains Main Road, the mixed area of Lakeside (mainly residential with some commercial) and the south-eastern section of the Westlake Golf Club, which is irrigated with treated effluent in the summer months. W7 is approximately 5 m upstream of the confluence of the Keyzers and Westlake Rivers, which itself is near the rivers' entrance into the Zandvlei estuary.



Figure 5-3: Site W5, looking downstream towards Main Road



**Figure 5-4: Site W6, just downstream of the Main Road bridge
(flow towards the right)**

5.2.2.2 Nitrogen

The mean TIN concentrations showed that levels were mesotrophic in winter, but were eutrophic and hypertrophic at various locations in summer. The minimum, maximum, median and mean TIN concentrations for both seasons are displayed in Table 5-3.

The locations with the highest mean TIN levels are K3 (15.22 mg/ℓ) and K2 (2.53 mg/ℓ) in the Keyzers system, and in the Westlake system they are W4 (15.05 mg/ℓ), W3 (12.60 mg/ℓ) and W2 (8.21 mg/ℓ) – all these levels fall within the hyper- or eutrophic range. K3 is a stream that flows behind Blue Route Mall which drains residential parts of Tokai, a section of the M3 freeway and Tokai Road, and a portion of the agricultural lands in the Pollsmoor Prison facility. K2, just off Tokai Road, is located between commercial and industrial areas in Tokai, and also drains the upstream residential areas.

The Pollsmoor agricultural area also drains to W3. It is possible that nutrients from fertilizers are leached into the soil and carried to the river downstream through interflow. W3, K3 and K2 are near the M3 freeway, which connects the False Bay area with Cape Town CBD,

and Tokai Road, a main transport link in the region. Vehicle emissions in this region could be a source of elevated TIN levels through the combustion of fossil fuels (Howarth *et al.*, 2000).



Figure 5-5: The downstream end of the stream behind Blue Route Mall, upstream of its confluence with the Keyzers River

Table 5-3: TIN concentrations measured in Phase 1 and 2

Sampling point	Winter (wet season)					Summer (dry season)				
	Samples	Min	Max	Median	Mean	Samples	Min	Max	Median	Mean
K1	-					2	1.19	1.60	1.40	1.40
K2	4	0.65	1.76	0.80	1.00	2	1.50	3.56	2.53	2.53 ¹
K3	-					2	12.94	17.50	15.22	15.22 ²
K4	2	0.96	1.24	1.10	1.10	2	1.59	1.70	1.65	1.65
K5	-					2	1.34	3.50	2.42	2.42
W1	-					2	1.91	3.97	2.94	2.94
W2	-					2	4.50	11.91	8.21	8.21
W3	3	1.40	2.30	1.67	1.79	2	12.50	12.69	12.60	12.60
W4	3	0.48	1.61	1.39	1.16	2	14.40	15.70	15.05	15.05
W5	4	0.67	3.51	1.89	1.99	2	1.59	1.70	1.65	1.65
W6	-					1 ³	1.71	1.71	1.71	1.71
W7	-					2	1.29	4.50	2.90	2.90

¹ Figures shaded in orange indicate mean levels in the eutrophic range

² Figures in shaded red indicate mean levels in the hypertrophic range

³ One sample was lost due to a leak

W4 drains the Westlake Golf Club and the residential area to its east. The raised TIN levels are possibly a result of the golf course's use of treated effluent for irrigation in the summer months. W2 is downstream of the Steenberg Village shopping centre and the Westlake Village low-cost housing area (which contains numerous backyard/informal dwellings). The section of the river that flows past it often contained debris during site visits.

The second set of samples taken in the dry summer showed much higher concentrations in general in both river systems, suggesting that raised nutrients are not necessarily linked to stormwater runoff. TIN concentrations remained largely constant going downstream. Furthermore, summer TIN concentrations predominantly constituted TO_xN, while in winter neither TO_xN nor TAN was consistently higher.

The mean concentrations measured in Phases 1 and 2 are displayed at their locations in Figure 5-6 and Figure 5-7 respectively.

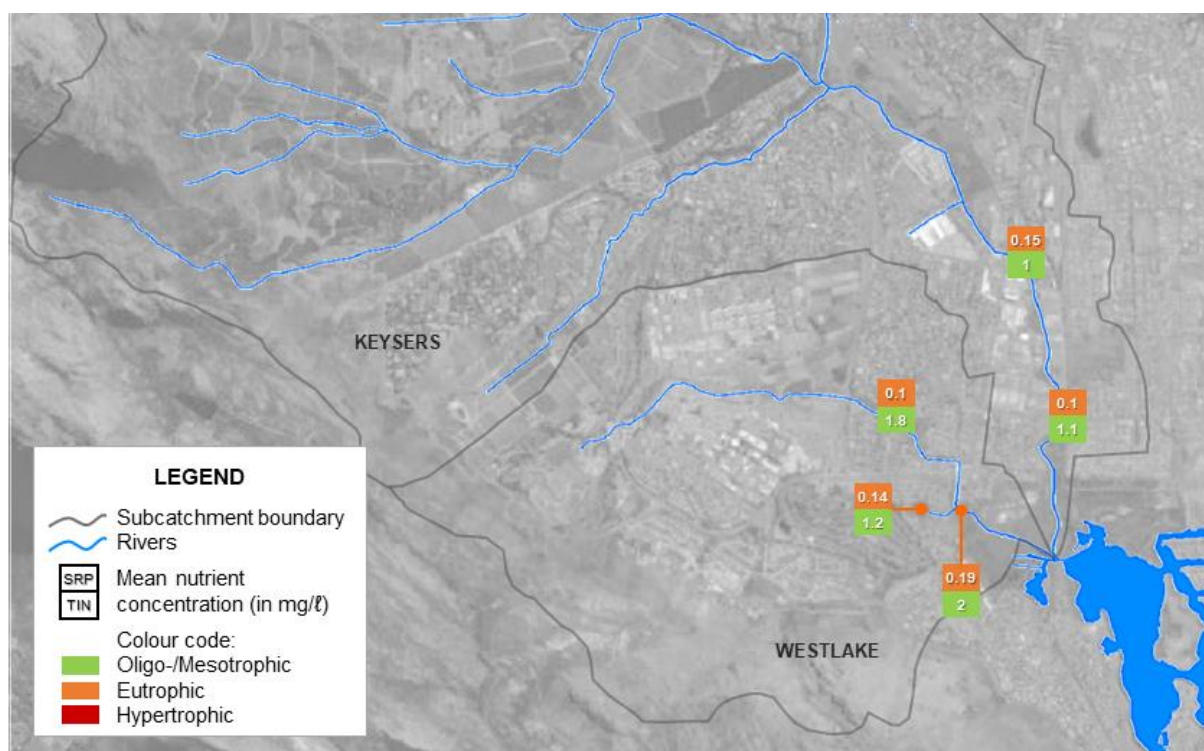


Figure 5-6: Map of mean SRP and TIN concentrations for Phase 1

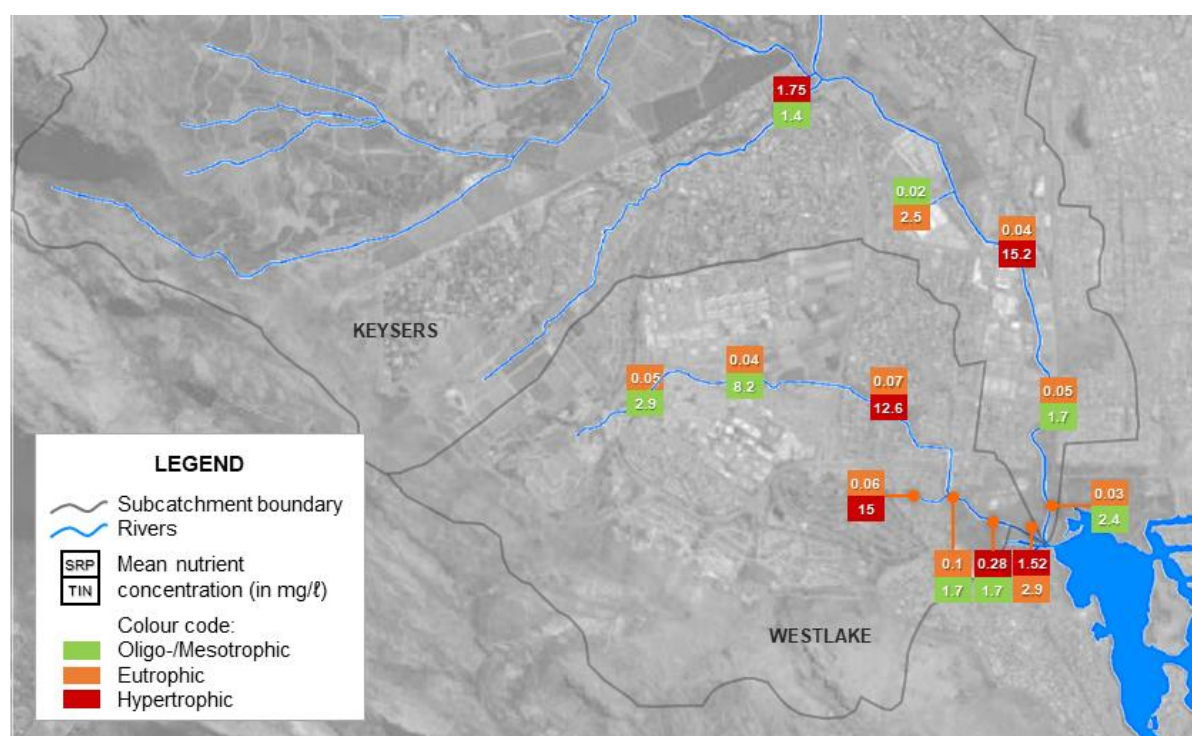


Figure 5-7: Map of mean SRP and TIN concentrations for Phase 2

5.2.2.3 Nutrient flow during storms

Phase 1 sampling in winter included monitoring storms at regular intervals during rainfall events to track how nutrient concentrations change in a storm. Storms on 2 July and 22-24 July 2019 were monitored at Site W5 – the same location as the Westlake flow level sensor. The Westlake River was chosen for continuous rainfall sampling because it displayed the highest EC readings during the preliminary sampling phase.

Sample-collection was timed to coincide with flow level readings. Continuous flow data could only be recovered for one of these storms however due to a battery malfunction during the first storm, after which new longer-lasting batteries were procured.

The following observations were made for both storms (as shown in Figures 5-8 to 5-10):

- The nutrient concentrations appeared to peak during the storms
- The EC profile did not match that of any of the measured nutrients
- The nutrient peak concentrations aligned with stream flow peaks

A comparison of EC levels and nutrient concentrations showed that EC readings did not rise and fall in tandem with all the nutrient concentrations, as shown in Figure 5-8 and Figure 5-9. Figure 5-9 shows that TO_xN and TAN concentrations peak when EC dips. This suggests that the nutrients being measured were not necessarily the most significant contributors to EC. In other words, the evidence did not confirm the initial assumption that higher EC levels implied higher nutrient concentrations.

For the second storm, stream flow data spanning the entire storm were available for comparison with nutrient levels and rainfall data from KEYS05FR, approximately 4 km away from W5. The comparison shows that stream flow peaks lag behind rainfall peaks by approximately 5 hours, as displayed in Figure 5-10. TO_xN and TAN both peak with stream flow; this may be because pollutants take a while to mobilise and wash further down the catchment. SRP appears to fluctuate throughout the storm, but in general dips when stream flow peaks.

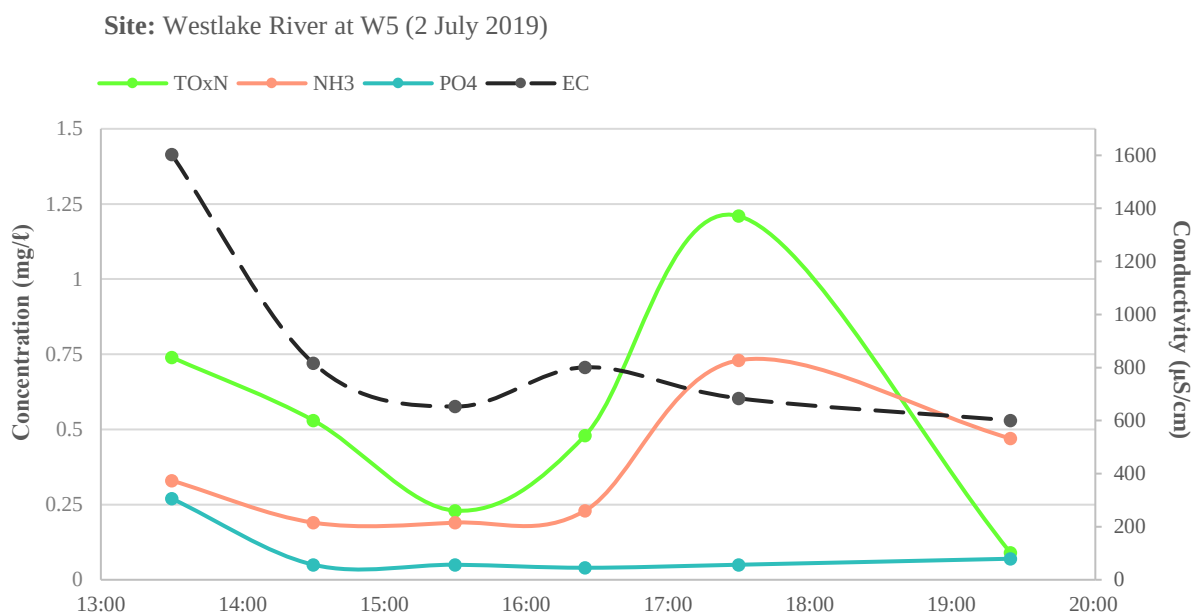


Figure 5-8: Nutrient and EC measurements at W5 for the 2 July storm

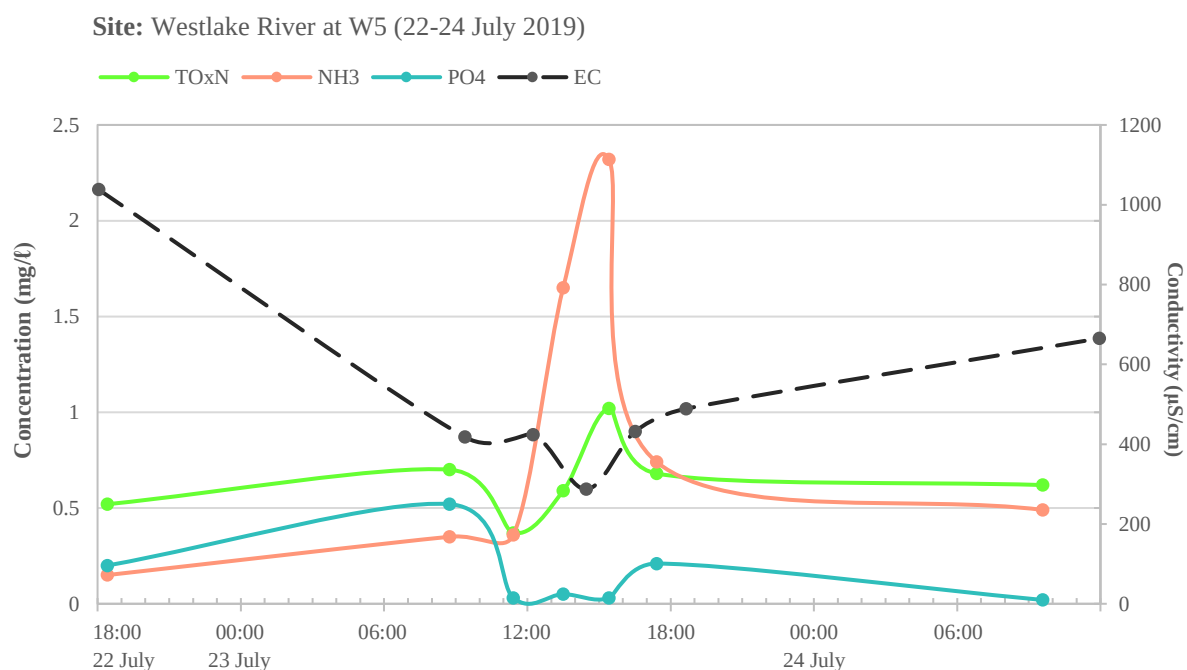


Figure 5-9: Nutrient and EC measurements before, during and after the 22-24 July storm

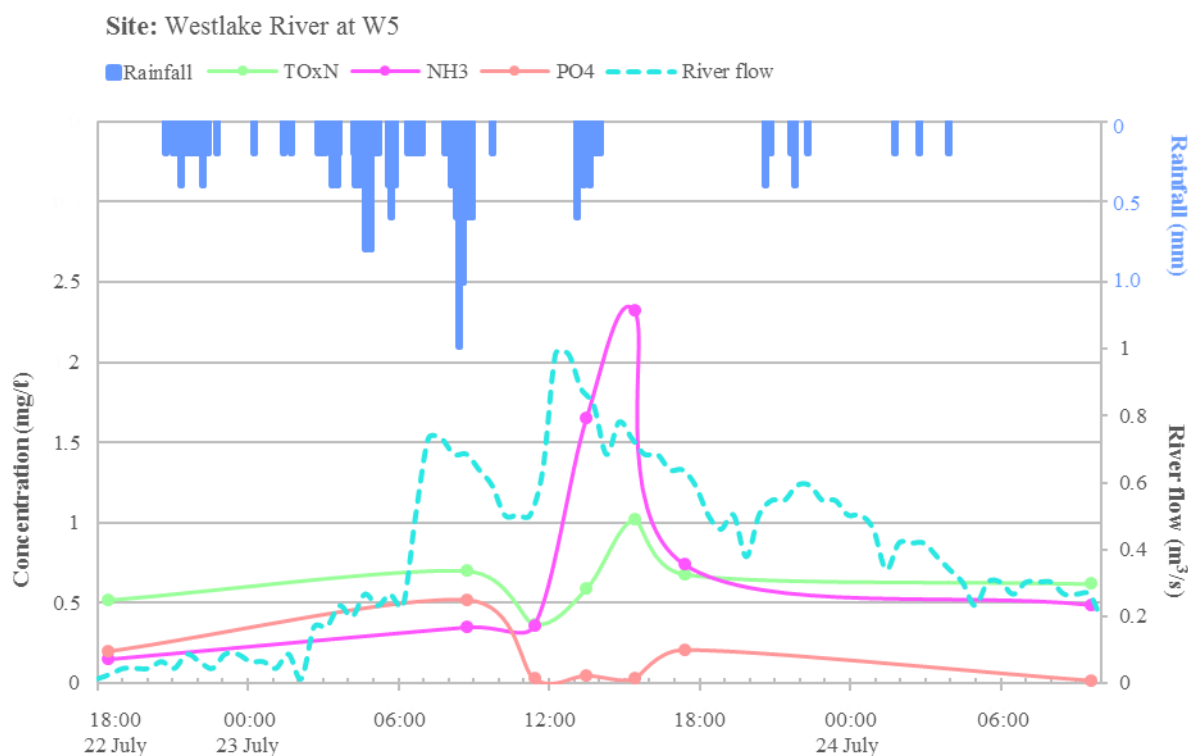


Figure 5-10: Rainfall, stream flow and water quality levels spanning the 22-24 July storm

5.3 CCT historical data

Historical water quality data were obtained from the CCT for the period 2012 to 2019, at monthly intervals. Data were limited to functional monitoring sites; CCT records list nine monitoring sites in the Keyzers system, but a complete dataset for the analysis period could only be found for two sites. The other sites appear to have no data beyond March 2009. The Westlake River system shows five monitoring points; data were available for all five locations but one site, CR05, had no data beyond 2014.

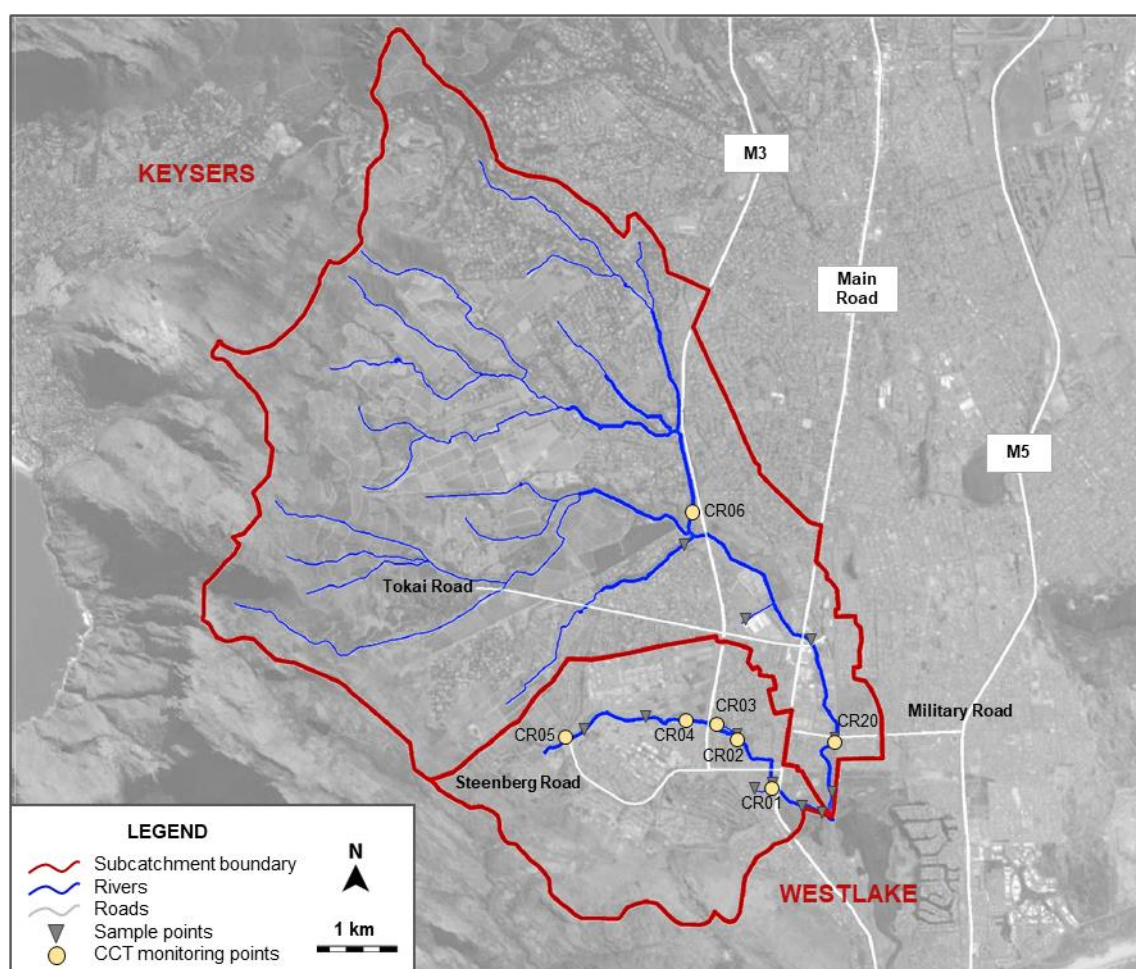
5.3.1 Monitoring locations

Some of the CCT monitoring locations coincided with sampling sites during the water quality testing phases. The CCT monitoring locations are listed in Table 5-4, with the overlapping sampling points shown in brackets. These locations are displayed in Figure 5-11.

Table 5-4: CCT water quality monitoring points for which data were available

Monitoring point	Location
Keyzers	
CR06	Spaanschemat River in greenbelt off Lismore Road
CR20	Keyzers River at Military Road (K4) ¹
Westlake	
CR05	Steenberg Road (upper)
CR04	In Pollsmoor Prison grounds, upstream of the irrigation pond
CR03	Altenburg Road, just downstream of the M3 freeway
CR02	Kirstenhof Greenbelt off Oranje Rd (W3)
CR01	Kerner Close (W5)

¹ Items in brackets denote sampling sites that coincide with CCT monitoring points

**Figure 5-11: CCT monitoring locations for which data were available from 2012-2019**

5.3.2 Historical nutrient trends

5.3.2.1 SRP

Historical recorded data show that SRP concentrations have frequently been in or above the eutrophic band, with the Keyzers system exhibiting generally higher levels than the Westlake River.

Figure 5-12 shows a broad indication of the water quality in the rivers based on monthly CCT data; it shows the percentage of samples within the eutrophic and hypertrophic range for both SRP and TIN at the various monitoring sites on a map. As shown, the SRP levels are much higher than the TIN levels.

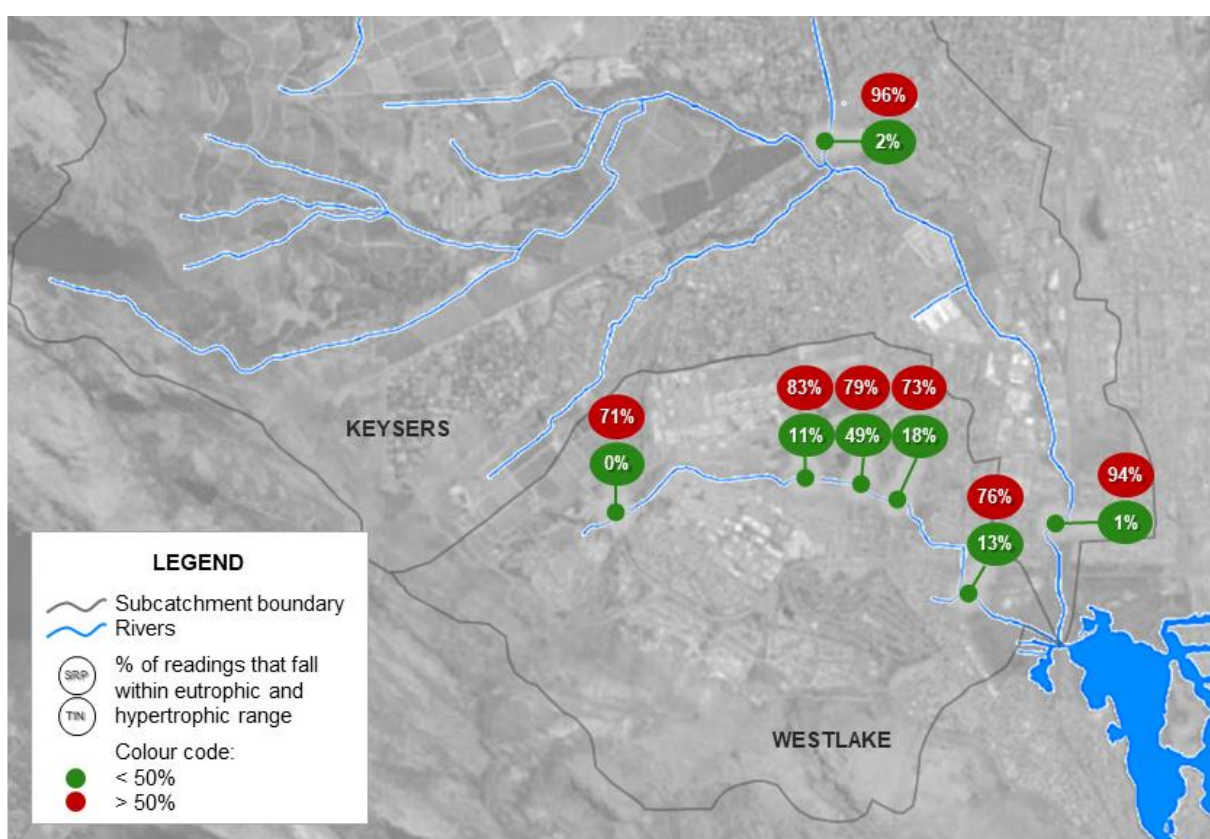


Figure 5-12: Map showing percentage of CCT monthly samples at/above eutrophic levels

The monthly readings for all monitoring sites over the 9-year period are presented in Figure 5-13 which shows that the mean and median values for all sites fall above the eutrophic threshold. The range of SRP measurements are similar across all the sites except CR06, which has a much larger range. Figure 5-14 displays the full extent of the CR06 data over the entire 8-year analysis period, showing seasonal fluctuations.

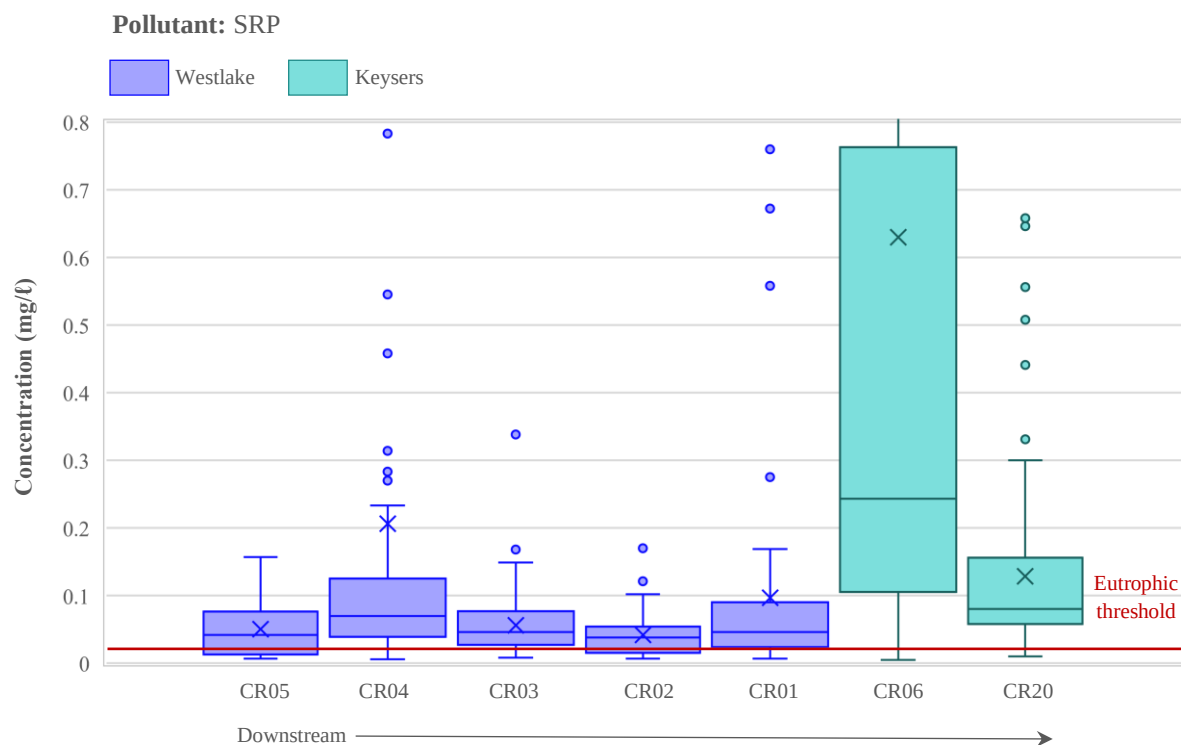


Figure 5-13: Partial range of SRP concentrations from 2012-2019
(chosen scale omits some outlier data; CR05 data until 2014)

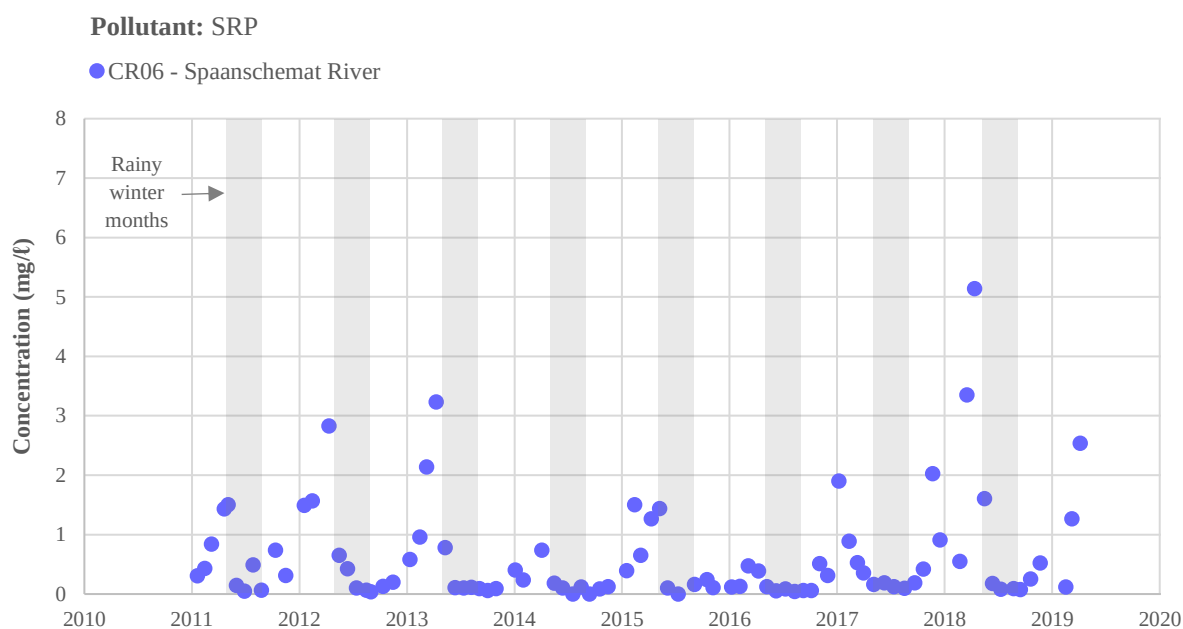


Figure 5-14: Seasonal fluctuations of SRP concentrations at CR06

While most of the sites do not display any seasonal trends, SRP concentrations at CR06 appear to increase in late summer, peak in autumn and dip during the rainy winter season – as shown in Figure 5-14. The peaks coincide with the period in which the first heavy rains of the year usually occur; this could imply that phosphates build up during the dry months and then get flushed downstream with the first annual storms.

CR06 is located in the middle reaches of the Keyzers system on the Spaanschemat River that drains the northern section of the Keyzers River sub-catchment. This includes the verdant low-density residential parts of Constantia and the vineyards above them. Fertilizers from both are the most likely sources of raised SRP levels in the river.

In the Westlake system site, CR04 recorded the highest SRP levels, with many outlier data points. This site is located in the Pollsmoor Prison grounds and drains part of the grounds, the Westlake Village low-cost housing area and residential and commercial areas in Westlake. High SRP levels could be the result of any combination of these land uses.

5.3.2.2 Nitrogen

Nitrogen concentrations in the study area only occasionally reached eutrophic levels, and generally appeared higher in the Westlake River than the Keyzers River system.

Figure 5-15 displays the TIN concentrations across the monitoring points – showing that CR02, CR03 and CR04 have the most recorded concentrations within the eutrophic range with approximately half the measurements at CR03 being greater than the eutrophic threshold. CR03 is approximately 150 m downstream of CR04; it drains the same areas as well as the M3 freeway, which likely carries pollutants from further afield. Figure 5-16 shows an increasing trend at these three sites over time.

In both river systems, TIN levels display a pattern of peaking in the latter part of the rainy winter season and dipping in summer – almost the opposite of observed SRP trends. Winter peaks suggest that increased nutrient levels are linked to stormwater runoff from rainfall events, rather than a nutrient build-up. The summer levels measured in the sampling programme were greater than the winter levels.

TO_xN and TAN concentrations display no clear seasonal trends over the study period. TO_xN is the dominant constituent of TIN at all sites except CR04, where TAN concentrations sporadically peaked two-three times a year (with peaks ranging between 4-16 mg/ℓ). CR04 is located in the Pollsmoor Prison grounds, which has historically experienced several sewage spills (DEA&DP, 2018). Since the TAN peaks are not accompanied by TO_xN or SRP peaks, sewage is more likely the cause of these peaks than fertilizers.

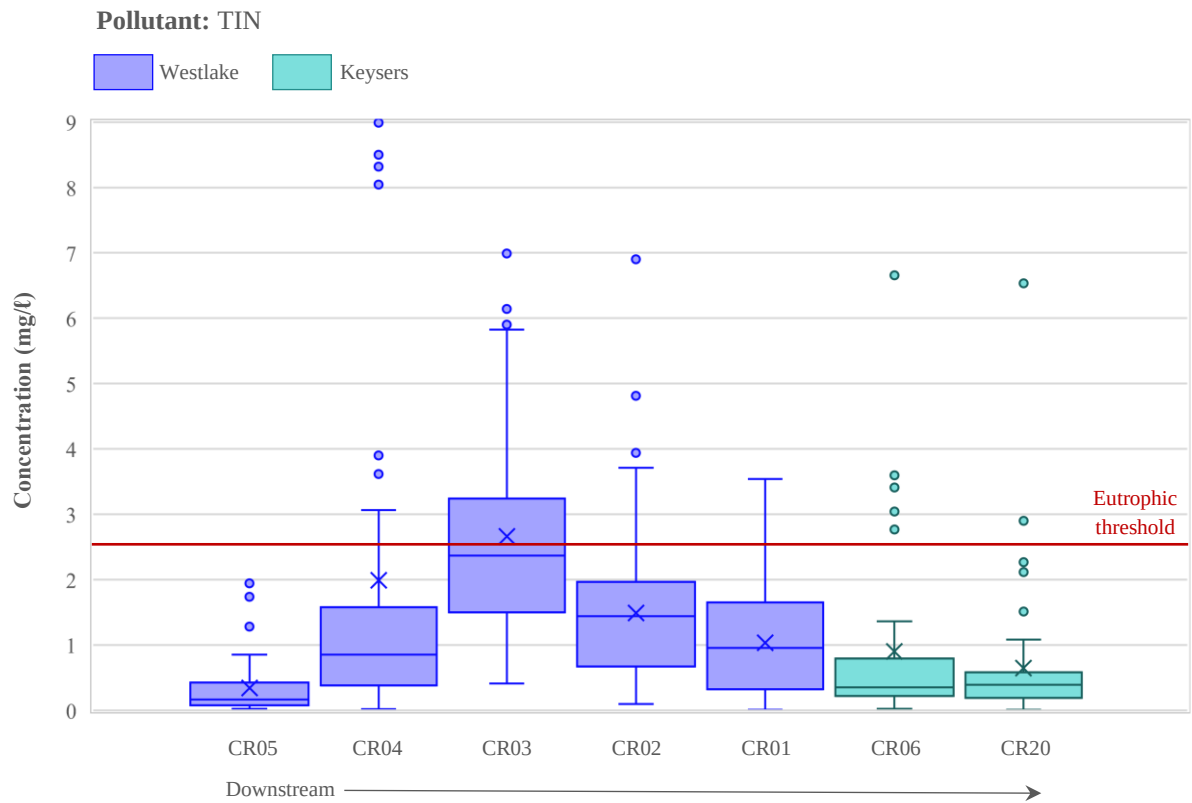


Figure 5-15: Partial range of TIN concentrations from 2012-2019
(chosen scale omits some outlier data; CR05 data until 2014)

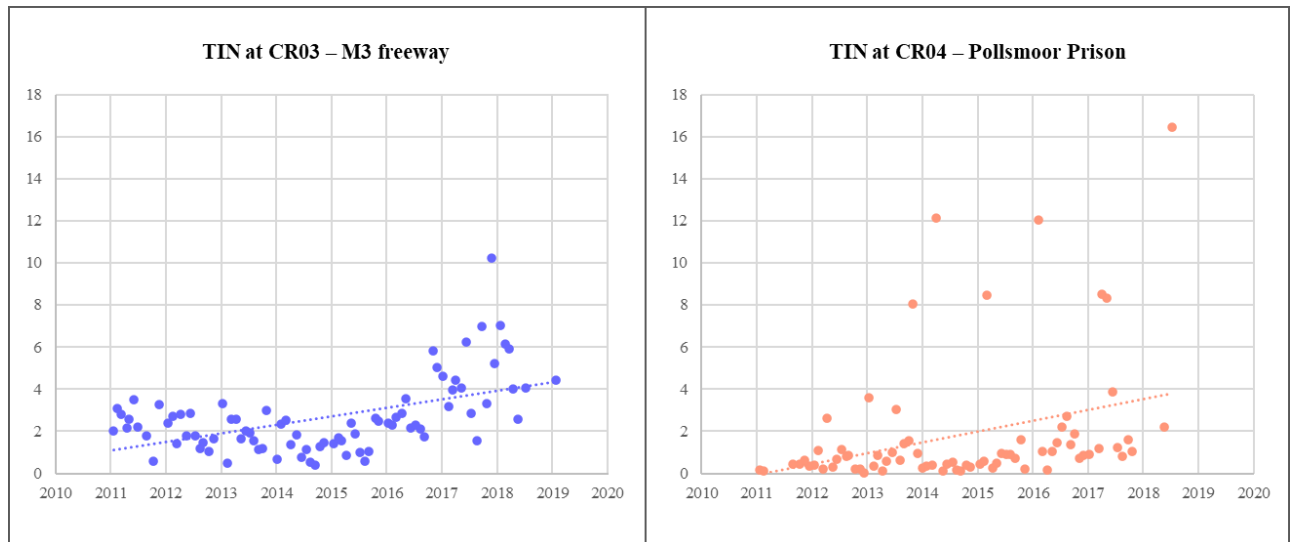


Figure 5-16: Increasing trends in TIN concentrations (in mg/l) at CR03-CR04 over time

5.4 Areas of concern

The sampling programme and the review of historical water quality data have shown various locations with elevated nutrient levels. The individual locations displaying raised nutrient levels are listed in Table 5-5, which also describes the areas drained by these locations. Studies on the Zandvlei catchment have commented on suspected sources of excessive nutrients in the rivers; these potential sources are summarised in Table 5-5.

Table 5-5: Summary of locations with elevated nutrient levels and possible sources

Site	Nutrient	Areas drained	Suspected sources (from studies)
Keysers (going downstream)			
CR06	SRP (H) ¹	Constantia residential and agricultural areas	Stabling and agricultural activity (Source-to-Sea, 2016), residential runoff (CCT, 2010)
K1	SRP (S)	Steenberg Golf Club, vineyards, Tokai residential and commercial areas	Golf course and vineyard fertilizers (Source-to-Sea, 2016)
K2	TIN (S)	Main Road, Tokai residential, commercial and industrial areas	Urban runoff and industrial waste (DEAD&DP, 2018)
K3	TIN (S)	M3, Tokai Road, Pollsmoor agricultural, Tokai residential and commercial areas	Agricultural activity (CCT, 2010), urban runoff (DEAD&DP, 2018; Source-to-Sea, 2016)
K4	SRP (S)	Military Road, industrial and commercial areas	Urban runoff and industrial waste (DEAD&DP, 2018)
Westlake (going downstream)			
W2	TIN (S)	Steenberg commercial area, Westlake low-cost/informal housing area	Runoff from low-cost housing area (Source-to-Sea, 2016)
CR04	SRP (H) TIN (H)	Pollsmoor grounds, Westlake low-cost/informal housing area	Pollsmoor sewage spills, urban runoff (DEAD&DP, 2018), runoff from low-cost housing area (Source-to-Sea, 2016)
CR03	TIN (H)	Pollsmoor grounds, M3	
W3 (CR02)	TIN (S) TIN (H)	M3, Pollsmoor agricultural, Westlake residential and commercial areas	Agricultural activity (CCT, 2010), urban runoff (DEAD&DP, 2018)
W4	TIN (S)	Westlake Golf Club, Westlake residential area	Residential runoff (CCT, 2010)
W5	SRP (S)	Steenberg Road, Westlake residential area	Residential runoff (CCT, 2010), urban runoff (DEAD&DP, 2018)
W6/W7	SRP (S)	Main Road, Lakeside residential area, Westlake Golf Club	

¹ Letters in brackets denote source of data: S – sampling and H – historical data

These locations listed in Table 5-5 were grouped into areas of concern, categorised as follows and displayed in Figure 5-17.

- Constantia agricultural area
- Steenberg golf course
- Pollsmoor Prison/M3 freeway
- Westlake golf course
- Tokai commercial and industrial area

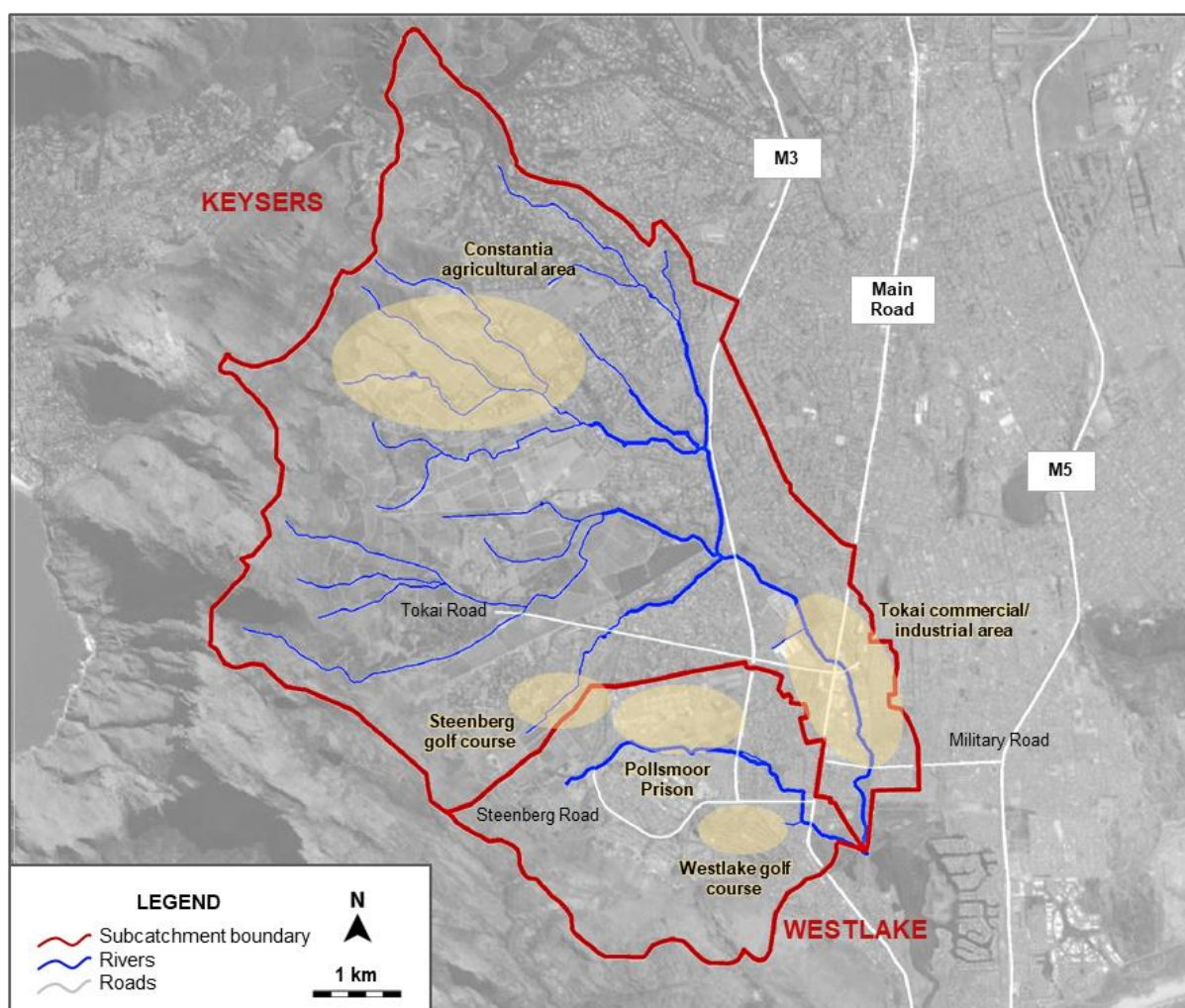


Figure 5-17: Areas suspected of causing raised nutrient levels in downstream rivers

5.5 Calculating EMC values from site measurements

EMC values are required as model input to estimate pollutant wash-off during storms. Since no published EMC data for the study area could be found, data from the sampling programme and historical monthly grab samples from CCT were used to estimate site-specific EMC values in conjunction with published EMC values from elsewhere (Appendix C).

EMC values were estimated for the following nutrients:

- SRP and TIN – to compare modelled pollutant concentrations to the eutrophic concentration ranges listed in the SA Aquatic Guidelines (DWAF, 1996) (Section 2.2). These were derived by adjusting published values based on measurements from continuous sampling during the 22-24 July storm.
- TP and TSS – to compare modelled pollutant removal with CCT guidelines (CT, 2009), mentioned in Section 4.5.3. These were derived by adjusting published values based on CCT monthly grab sample data.

5.5.1 Deriving SRP and TIN EMC values from continuous sampling

Composite EMC values for SRP and TIN were calculated for Site W5 on PCSWMM; this process used published EMC values which were adjusted based on concentrations measured at W5. This method is outlined by CHI (2020a) and is based on Equation 4.3:

$$EMC_{W5} = \frac{\text{Total pollutant load (kg)}}{\text{Total flow volume (M}\ell\text{)}} \quad (4.3)$$

Using SRP as an example, the steps of this process are described below:

- The first step was to insert published EMC values for each land use into the model – published values were obtained from a variety of literature sources shown in Appendix C.
- The model was then run over the same period that the storm was monitored, with rainfall data for the storm in question. The model output delivered the SRP loads and runoff volumes at W5. Using these figures in Equation 4.3 gave a composite (i.e. representing all the land uses draining to W5) SRP EMC value at W5 of 0.124 mg/ℓ.
- Next, the site measurements were used to calculate the same thing (the composite SRP EMC value at W5). The total SRP load over the monitoring period was determined by multiplying the SRP concentration time series by the stream flow (calculated from data

from the level sensors) time series, and adjusting for unit conversions – this was done in PCSWMM. Using Equation 4.3 gave a composite SRP EMC value at W5 of 0.201 mg/ℓ.

- Comparing these two composite EMC estimations showed that the estimated site-based EMC was greater than the literature-based value by a factor of 1.62.
- The initial input published EMC values (for each land use) were then scaled up by a factor of 1.62. When the model was run again, the scaled literature values resulted in a site-specific SRP EMC of 0.201, matching the estimation based on site measurements.

Table 5-6: Calculated SRP and TIN EMC values at W5

EMC calculation based on	SRP	TIN
Site measurements	0.201 mg/ℓ	1.638 mg/ℓ
Literature sources/model	0.124 mg/ℓ	1.233 mg/ℓ
Adjustment factor	1.62	1.33

5.5.2 Comparing TP and TSS EMC values to monthly grab samples

TP and TSS data were available as monthly concentrations at Site W5. These measurements are an indication of the nutrient levels at a specific point in time and do not reflect how the concentration may have fluctuated throughout the day. A simulation was run using published EMC values; the resultant modelled TP and TSS concentrations at W5 were compared to the range of concentrations measured at W5 during 2018-2019. The range of TP and TSS concentrations for this period are shown in Table 5-7.

The modelled concentrations were all within the ranges listed in Table 5-7; the published EMC values were therefore left unchanged for TP and TSS. The EMC values used in the model are listed in Section 4.4.5.

Table 5-7: Range of measured TP and TSS concentrations during winter months

Pollutant	Range of concentrations
TP	0.031 – 1.73 mg/ℓ
TSS	6 – 378 mg/ℓ

5.6 Summary

This chapter described the water quality in the study in terms of SRP and TIN, based on observed data gathered from sampling and historical data over an 8-year period (2012-2019).

SRP appears to be a concern in both river systems, with concentrations very frequently being above the eutrophic threshold. TIN concentrations are more of a concern in the Westlake system; at CR03 approximately half the historical samples over the 8-year period display were above-eutrophic levels. The locations displaying elevated nutrient concentrations were grouped into areas of concern, displayed in Figure 5-17. Nutrient concentration data were used to modify EMC values from the literature for model input.

6. Model scenarios

This chapter describes the various scenarios modelled commencing with the current scenario which describes the catchment as it currently stands. Thereafter, a scenario representing the pre-development scenario is described; the pre-development scenario typically offers an approximation of peak flow rates and volumes prior to development. Lastly, various potential SuDS interventions were modelled. From the list of interventions, three scenarios were developed using different combinations of SuDS measures. The first SuDS scenario is based on placing SuDS measures near suspected sources of nutrient enrichment (based on the evidence presented in Chapter 5). The second SuDS option opts for a different approach, using large-scale options (regional controls) to treat larger volumes of water lower in the study area. The third scenario offers a combination of both targeted and large-scale interventions.

6.1 Current Scenario

The current scenario represents the topography, rivers, streams, ponds, dams and stormwater network as is (in the year 2020). The following paragraphs describe how various components were modelled.

6.1.1 Rivers, streams and ponds

A river, stream, channel or pipe is represented in the model by a ‘conduit’ – the model differentiates between them based on their cross-section and surface roughness (Manning’s n) values.

Rivers, streams and channels were modelled to have either vegetative, earth or concrete linings, based on site inspections. Where channels had concrete linings punctuated with vegetative growth (as shown in Figure 6-2), they were modelled with composite roughness values.

The rivers have irregular cross-sections; these were modelled using the ‘transect creator’ tool, which uses the DEM to identify cross-sections along a conduit. Canalised sections are concrete-lined with mostly trapezoidal and some rectangular sections; geometries were assigned according to site inspections and satellite imagery. The study area contained various small ponds, which were modelled as storages. The dams at Dreyersdal Farm and in the Pollsmoor Prison grounds were also modelled as storages.

The complete system has one discharge point, termed an outfall. The outfall was placed after the confluence of the Keyzers and Westlake Rivers where the rivers enter the Zandvlei estuary. The various modelled components are shown in Figure 6-1.

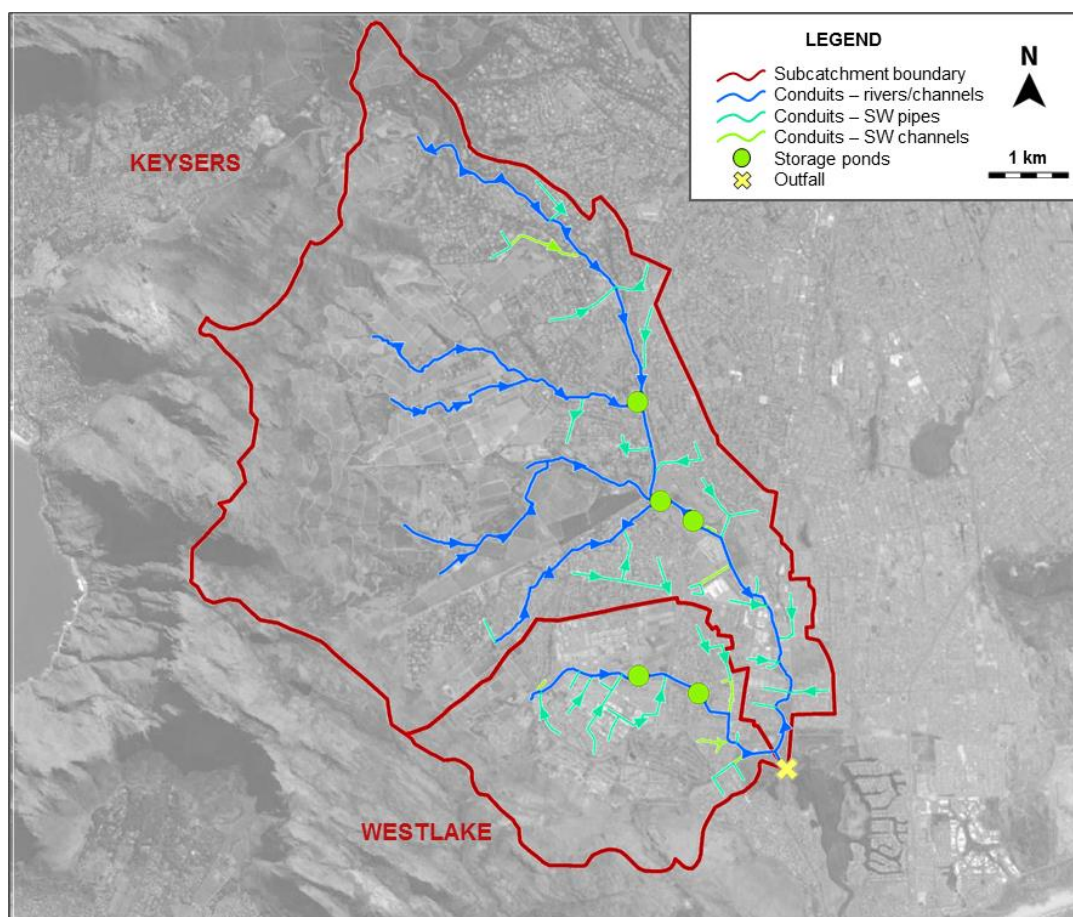


Figure 6-1: Map showing various components modelled in the current scenario



Figure 6-2: Concrete-lined channel with vegetative outgrowth in Westlake, downstream of Klein Wassenaar Road (Source: Geordie Thewlis)

6.1.2 Existing SuDS practices

There are various components in the study that can be considered as existing SuDS practices; these include shallow swales along roadways and various ponds and wetlands throughout the study area. The pollutant removal capacity of these components was not modelled; since EMC values were adjusted according to field water quality data, the treatment contribution of existing SuDS measures is already accounted for.

Three wetland areas exist in the study area currently: the wetlands at Dreyersdal (including a farm dam upstream of the wetlands); the Kirstenhof Greenbelt; and the Westlake Wetlands. The Dreyersdal and Kirstenhof wetlands are only fed during periods of high flow through a diversion channel. The more polluted low flows bypass these wetlands, resulting in their treatment potential remaining largely untapped (Source-to-Sea, 2016). In the Current Scenario model the Dreyersdal and Kirstenhof wetlands were simulated as storages without pollutant removal, fed by diversion channels.

6.2 Pre-development Scenario

A pre-development scenario was developed to compare how current water quantity conditions differ from what they would possibly have been before it was developed and urbanised. The typical goal of any SuDS scheme is to achieve pre-development flow and quality conditions.

To create the pre-development model, the following changes were made to the current scenario:

- All land uses were changed to nature and conservation areas, i.e., all impervious land cover was replaced with pervious land cover.
- The dams at Dreyersdal and Pollsmoor were removed.
- The stormwater network pipes and man-made channels were converted into shallow open channels.
- All conduits were given natural (earth) Manning roughness values.

Many courses of many rivers in the study area have been altered due to development. These were not adjusted in the pre-development model as information on the original river lines was unavailable. The changes listed above were considered an adequate approximation of the pre-development scenario.

6.3 Modelled SuDS interventions

One of the objectives of this study was to investigate the effect that SuDS interventions would have on water quality in the study area. This section explains the various SuDS options that were modelled, and thereafter groups them into different scenarios to compare their relative flow reduction and pollutant removal.

Since the area has already been largely developed, local controls with retrofit potential were mainly considered; for regional controls, the existing wetlands were considered for reintegration into the system. The suitability of the following SuDS components to the site will ultimately be subject to detailed site investigations including groundwater assessment, soil suitability, existing infrastructure and design requirements for larger storms. SuDS components were sized according to guidelines set out by Woods Ballard *et al.* (2015) and Armitage *et al.* (2013). All design assumptions, equations and standards used in this investigation are listed in Appendix E.

SuDS components typically require various attendant structures such as inlets, energy-dissipation measures and outlet/overflow structures. As these structures are not pertinent to water quality, they do not form part of the analysis here. Some examples of these structures are shown in Appendix F.

6.3.1 SuDS in the Keyzers system

There are many areas of open space in the Keyzers River system suitable for the implementation of SuDS measures. The open spaces in the upper reaches are ideal SuDS locations, especially since the farms there are considered potential sources of pollutants (Section 5.4). The M3 freeway, which spans both the Keyzers and Westlake River drainage basins, has wide (varies between 1 and 4 m) strips of open land flanking each carriageway (Figure 6-3). There are no kerbs, so runoff from the freeway flows directly onto these strips of land. The vineyards in the upper reaches and Dreyersdal Farm in the middle reaches have large open spaces compared to the rest of the sub-catchment, with the latter hosting a dam and wetland areas – although these are currently only accessible to the river via diversion channels. Downstream of Tokai Road, the Keyzers River is flanked by open grass fields (Figure 6-4) – ideal locations for SuDS interventions as this region is also adjacent to an industrial area. SuDS interventions modelled in these areas are described below.



Figure 6-3: Open grass along the M3 freeway with no kerbs along the road



Figure 6-4: Open field along the Keyzers River just downstream of Military Road

6.3.1.1 Constantia agricultural area

The agricultural fields in Constantia are at the highest point in the study area. This vicinity is a suspected pollution hot spot area, but since it is so far up in the catchment placing costly SuDS measures here was considered a potential waste of resources as the river is likely to be further polluted downstream. Swales were considered the most suitable option for their relatively low cost and ability to fit into the available space. Additionally, they can serve a conveyance function which can be used to convey runoff to existing dams.

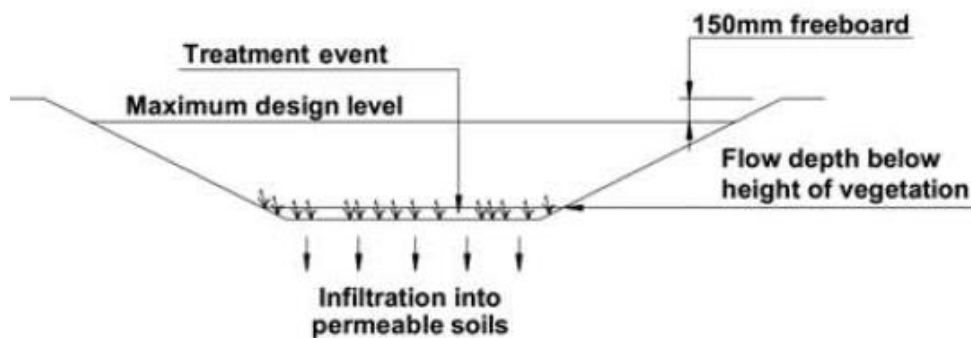


Figure 6-5: Cross-section of a typical conveyance swale
(Woods-Ballard *et al.*, 2007)

Five swales were placed in this area (aerial view shown in Figure 6-6). They were placed where longitudinal slopes could be maintained below the recommended maximum of 6% (Woods Ballard *et al.*, 2015), usually along property boundaries. This is a theoretical maximum; in practice, the maximum safe slope will be determined by soil erodibility, vegetation condition and bed shear stress. Where possible, the swales convey flow to existing ponds/dams where retention can facilitate the settling of sediments and removal of nutrients. PCSWMM allows the user to specify the amount of vegetation on a swale which, according to CHI (2019), increases infiltration. Swales that are located near to streams were modelled with more vegetation than the other swales to increase infiltration and act as a buffer before runoff reaches the river. These swales were all modelled with a berm height of 0.4 m and side slopes of 33%. Other modelled properties as well as the WQV for each swale are listed in Table 6-1.

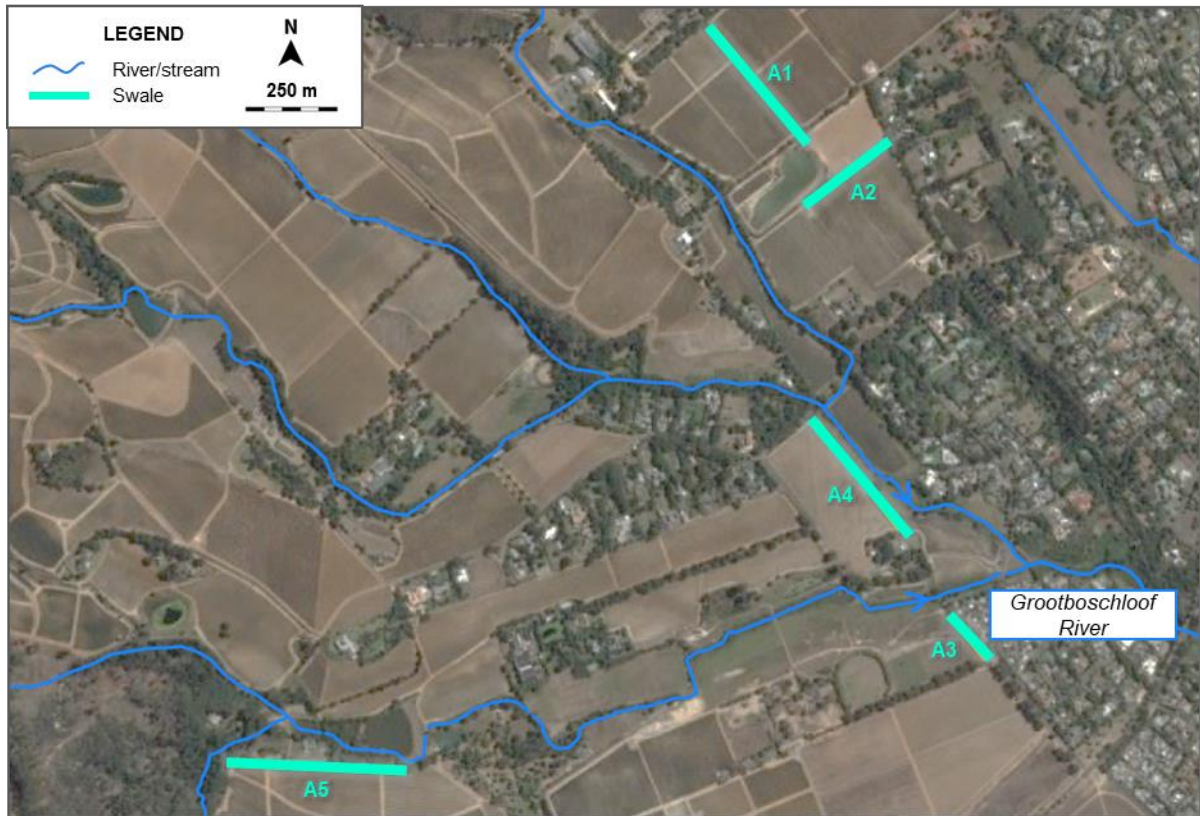


Figure 6-6: Plan view of the modelled Constantia agricultural swales
(Bing Maps, 2021)

Table 6-1: Modelled parameters of the Constantia agricultural swales

Swale no. ¹	WQV	Slope	Base width	Length	Vegetation height
A1	300 m ³	6.0%	0.8 m	300 m	0.10 m
A2	245 m ³	4.8%	0.8 m	200 m	
A3	210 m ³	5.0%	1.4 m	100 m	
A4	365 m ³	4.1%	1.6 m	275 m	0.15 m
A5	320 m ³	4.6%	0.8 m	350 m	

¹ Swales numbered according to Figure 6-6

6.3.1.2 Dreyersdal wetland upgrade

Downstream of the M3 freeway, the Keyzers River flows through the Dreyersdal Farm area, which includes a dam and wetland areas. Currently, the river flows alongside these elements separated by berms, and is only connected to the dam and wetlands via overflow channels. This assists in flood control but fails to utilise the treatment potential of the dam and wetlands for smaller, more frequent storms – which carry a greater pollutant load than larger, less frequent storms (Armitage *et al.*, 2013). Source-to-Sea (2016) notes that the separation of the wetlands from the polluted low flows probably contributes to the maintenance of the wetland habitat, as this wetland species (*Juncus kraussii*) has a high sensitivity to an increase in nutrients. If the wetland was connected to the high flows, the exposure to elevated nutrient concentrations could result in the proliferation of invasive wetland species.

In the model, the wetland was reconnected to the river allowing the treatment capacity of the wetland to permanently benefit the river and not just during high flows. In view of the wetland's sensitivity to increased nutrients, only a section was modelled as connected to the river – to protect the rest of the wetland habitat. Practically speaking, the berms currently separating the river from the wetland would need to be removed. A sediment bay would be required as a pre-treatment mechanism, and additional berms built to segregate the wetland into the protected area and the area used for treatment. These elements are shown in Figure 6-7. The modelled parameters are shown in Table 6-2. Design parameters adhered to are described in Appendix E.



Figure 6-7: Configuration of the modelled Dreyersdal wetland upgrade
(Plan view: Bing Maps, 2021; View 2: Google Maps, 2021)

Table 6-2: Modelled Dreyersdal wetland system parameters

Wetland	Slope	Surface area	WQV depth	Permanent depth	Maximum depth
Dreyersdal wetland	0.1 %	68 800 m ²	0.51 m	1.2 m	1.7 m

6.3.1.3 Blue Route Mall

The water quality study identified elevated nutrient concentrations in the stream flowing directly behind Blue Route Mall. The water sometimes appeared murky (Figure 6-8) and occasionally emitted an odour. According to CCT data, the residential and commercial areas to the west, north and south of the mall drain into this stream, which then drains into the Keyzers River. The stream flows through an open public grassed area often used as a recreational space, particularly during lunch time by Blue Route employees. A plan view of the stream with some pictures of the surrounding area are shown in Figure 6-9.



Figure 6-8: Murky water (potentially sewage) flowing in the stream behind Blue Route Mall



Figure 6-9: Plan view of the Blue Route stream with surrounding views
(Plan view: Bing Maps, 2021)

Three sections of the grassed area alongside the stream were modelled as bioretention areas, as shown in Figure 6-10. The bioretention systems were modelled as 0.4 m deep with side slopes of 25% to account for safety considerations (Woods Ballard *et al.*, 2015); they would need suitable overflow structures allowing water to flow into the stream when the bioretention area is full. Modelled parameters and the WQV for each bioretention system are shown in Table 6-3. All bioretention systems were given conservatively higher vegetation fractions (volume of vegetation in the surface storage area) to account for overgrowth, litter and debris, as these were frequently seen in the vegetated parts of the rivers on site visits.

Table 6-3: Modelled Blue Route Mall bioretention system parameters

Bioretention system no. ¹	WQV	Surface area	Berm height	Vegetative fraction
BRM-1	32 m ³	55 m ²	0.4 m	70%
BRM-2	18 m ³	35 m ²	0.4 m	60%
BRM-3	15 m ³	35 m ²	0.4 m	50%

¹ Bioretention areas numbered according to Figure 6-10



Figure 6-10: Modelled bioretention areas behind Blue Route Mall
(Plan view: Bing Maps, 2021)

6.3.1.4 Tokai industrial area

South of Tokai Road, the Keysers River passes through a light industrial area flanked by open grass fields on both sides. Numerous stormwater pipes draining the surrounding industrial region discharge into the river in this vicinity.

Four bioretention areas were modelled in these open fields in order to protect the river from directly receiving stormwater flows. The modelled bioretention systems vary in size and were placed at stormwater discharge points where, in practice, they would provide treatment before the stormwater enters the river. The stormwater pipes were then modelled to discharge into the bioretention areas, as shown in Figure 6-11. The bioretention system parameters are listed in Table 6-4.



Figure 6-11: Plan view of the modelled Tokai bioretention areas and photographs
(Plan view: Bing Maps, 2021)

Table 6-4: Modelled parameters for Tokai bioretention areas

Bioretention system no. ¹	WQV	Surface area	Berm height	Vegetative fraction
TOK-1	12 m ³	50 m ²	0.6 m	50%
TOK-2	36 m ³	50 m ²	0.6 m	50%
TOK-3	17 m ³	35 m ²	0.6 m	50%
TOK-4	30 m ³	60 m ²	0.5 m	40%

¹ Bioretention areas numbered according to Figure 6-11

6.3.2 SuDS in the Westlake system

In the area draining the Westlake River, the areas identified as suitable for SuDS implementation included the Steenberg and Westlake Golf Clubs, the Pollsmoor Prison grounds, and the Kirstenhof Greenbelt. The Kirstenhof Greenbelt houses a wetland area which is separated from

the river trunk by a berm. As with the Dreyersdal wetlands, this wetland only receives flow during large storms through a diversion channel. In the lowest area in the Westlake and Keyzers sub-catchments, the rivers flow between reed-marsh areas upstream of their convergence (Morant & Grindley, 1982). However, since the rivers are both separated from the reeds by berms so any treatment potential is unused. SuDS measures modelled in these areas are described below.

6.3.2.1 Steenberg Golf Course swales

A section of the Steenberg Golf Course is located in the upper reaches of the Westlake drainage area; Steenberg Estate, a high-income housing estate to which the golf course belongs, straddles the boundary between the Keyzers and Westlake drainage basins. The Prinseskasteel River, one of the major tributaries of the Keyzers River, flows through this estate. Observed elevated nutrient concentrations in the Prinseskasteel River are thought to originate in the Steenberg Golf Course (Section 5.2.2) which uses treated sewage effluent for irrigation.

Two swales were modelled in the golf course, along the eastern and south-western rims of the golf course respectively (Figure 6-12). The swales will capture runoff from the golf course as well as the adjacent agricultural area, promoting filtration and encouraging infiltration in the model. Both swales were modelled with a berm height of 0.4 m and side slopes of 33%. Swales in steeper areas typically require measures to reduce the inflow velocities (Woods Ballard *et al.*, 2015). The south-western swale has a slope greater than 3% and thus would require appropriate velocity-reducing measures, such as check dams. The modelled parameters are presented in Table 6-5.

Table 6-5: Details for modelled Steenberg Golf course swales

Swale no. ¹	WQV	Slope	Base width	Length	Vegetation height
S1	210 m ³	0.8%	1.2 m	800 m	0.10 m
S2	105 m ³	3.2%	1.2 m	420 m	

¹ Swales numbered according to Figure 6-12

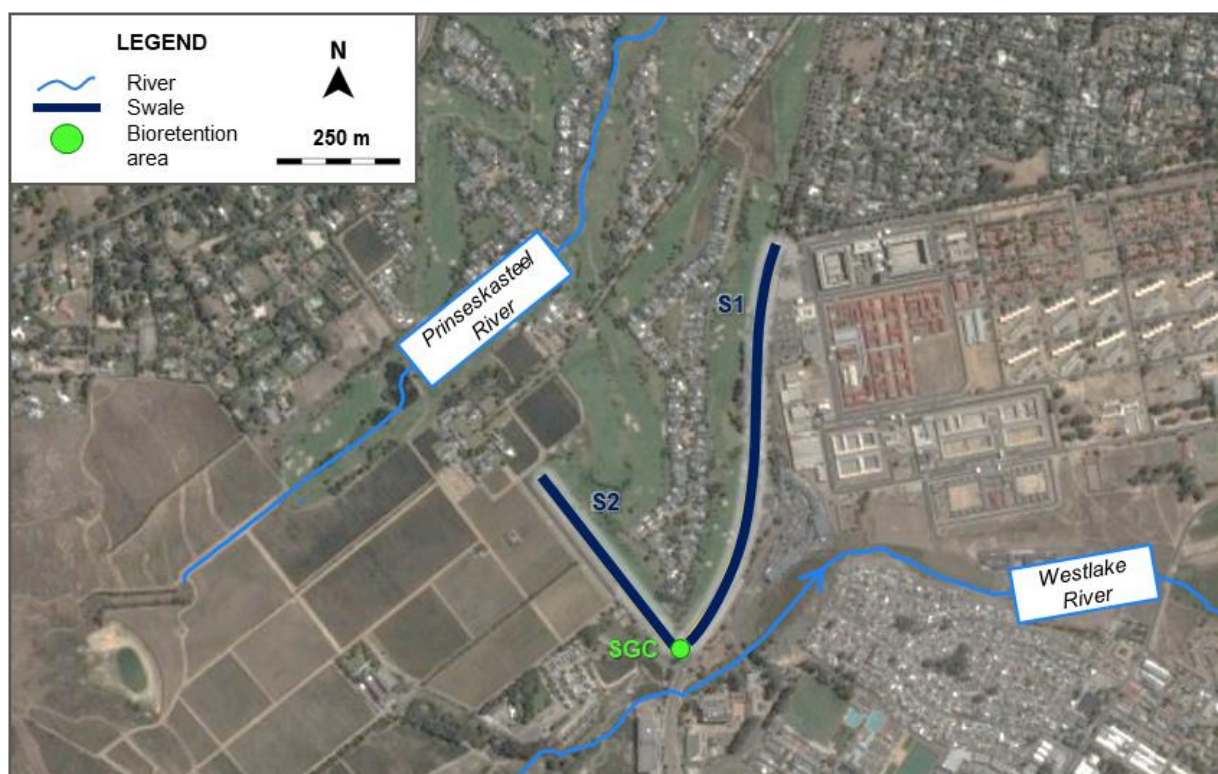


Figure 6-12: Plan view of Steenberg Golf Club swales and bioretention area
(Plan view: Bing Maps, 2021)

6.3.2.2 Steenberg Golf Course bioretention area

The lowest area of the Steenberg golf course is an ideal SuDS location; the whole course drains to this area which is currently a grass lawn. The open space could be converted into a bioretention area that could even be incorporated as an obstacle in the golf course. This section of the golf course currently drains into the channel that passes under Catharina Avenue.

A 35 m² bioretention system was modelled in this area (shown in Figure 6-12, not to scale). If used as part of a treatment train with the swales described in the previous section, the swales would then convey stormwater into the bioretention area. The system was modelled to discharge into the channel under Catharina Avenue. Modelled parameters are displayed in Table 6-6.

Table 6-6: Modelled parameters for Steenberg Golf Club bioretention area

Bioretention system no. ¹	WQV	Surface area	Berm height	Vegetative fraction
SGC	8 m ³	35 m ²	0.4 m	50%

¹ Bioretention areas labelled according to Figure 6-12

6.3.2.3 Westlake Golf Course swales

The Westlake Golf Club, like its Steenberg counterpart, uses treated effluent for irrigation during dry months. A stream draining the golf course displayed hypertrophic nutrient levels during the water quality testing phase. Modelled interventions in this area are similar to the measures modelled at the Steenberg Golf Course.

Two swales were modelled along the eastern boundary of the golf course (Figure 6-13), which both slope downward toward the lowest area on the golf course that drains into a channel in Twente Road. The swales were modelled with a berm height of 0.4 m and side slopes of 33%. One of the required inputs of swales in PCSWMM is the amount of vegetation. The Westlake swales were modelled with more vegetation than the Steenberg swales since the streams draining the Westlake golf course displayed higher nutrient concentrations. Other swale details are listed in Table 6-7.

Table 6-7: Details for modelled Westlake golf course swales

Swale no. ¹	WQV	Slope	Base width	Length	Vegetation height
W1	300 m ³	0.8%	1.0 m	700 m	0.15 m
W2	145 m ³	3.2%	1.0 m	250 m	

¹ Swales numbered according to Figure 6-13



Figure 6-13: Plan view of Westlake Golf Club swales and bioretention area
(Plan view: Bing Maps, 2021)

6.3.2.4 Westlake Golf Course bioretention area

The lowest area of the Westlake Golf Course is along its eastern rim at the point it drains into Twente Road and into a channel that has been termed the ‘Golf Course Stream’, shown in Figure 6-13.

A bioretention system was modelled at this low point, with the parameters displayed in Table 6-8. This bioretention system was modelled with more vegetation than its Steenberg counterpart because of the greater pollutant concentrations found in the Golf Course Stream. The bioretention system in each golf course was modelled with a berm height of 0.4 m – they were chosen to be shallow in order to blend in with other typical golf course features, such as shallow bunkers. Since they are draining permeable areas, controlling runoff is not a priority in these two locations.

Table 6-8: Modelled parameters for Westlake Golf Club bioretention area

Bioretention system no. ¹	WQV	Surface area	Berm height	Vegetative fraction
WGC	11 m ³	35 m ²	0.4 m	65%

¹ Bioretention areas labelled according to Figure 6-13

6.3.2.5 Pollsmoor Prison bioretention area

As discussed in Section 3.4.6, Pollsmoor Prison is often blamed for poor water quality in the Westlake River. Sewer spills have been cited as the cause of the poor water quality. To address these water quality concerns using SuDS, a bioretention system was modelled in the Pollsmoor grounds just upstream of the irrigation pond (Figure 6-14), discharging into the river just upstream of the pond. The bioretention area could provide a pre-treatment function for runoff flowing into the irrigation pond. Modelled parameters for this bioretention system are listed in Table 6-9.

Table 6-9: Modelled parameters for Pollsmoor Prison bioretention area

Bioretention system no. ¹	WQV	Surface area	Berm height	Vegetative fraction
POL	9 m ³	25 m ²	0.5 m	60%

¹ Bioretention areas labelled according to Figure 6-14



Figure 6-14: Plan view of Pollsmoor bioretention and M3 swales, with pictures
(Plan view: Bing Maps, 2021; Views 1 and 2: Google Maps, 2021)

6.3.2.6 M3 freeway swales

The M3 freeway, which connects the False Bay region to Cape Town CBD, has broad (2-4 m) strips of land alongside each carriageway. Pollutants from the heavily trafficked road and the Pollsmoor agricultural fields drain into the Keyzers and Westlake Rivers, which both cross the M3 through culverts.

Three swales were modelled in this area; two were modelled on the eastern side of the M3 and another one on the western side of the freeway, as shown in Figure 6-14. Each swale conveys runoff into existing discharge points into the Westlake River, where it crosses the M3 freeway. Swale M1 and a portion of swale M3 run along the eastern boundary of Pollsmoor Prison, draining the agricultural fields in the prison grounds. These swales were modelled with berm heights of 0.3 m and side slopes of 33%. Other modelled parameters and WQV values for each swale are presented in Table 6-10.

Table 6-10: Details for the modelled swales along the M3 freeway

Swale no. ¹	WQV	Slope	Base width	Length	Vegetation height
M1	445 m ³	1.1%	1.6 m	650 m	0.10 m
M2	410 m ³	1.1%	1.2 m	650 m	
M3	380 m ³	1.0%	1.0 m	400 m	

¹ Swales numbered according to Figure 6-14

6.3.2.7 Kirstenhof Greenbelt Wetland

The Kirstenhof Greenbelt is just downstream of the M3 freeway and pollutants from the freeway and the Pollsmoor Prison grounds pass through it. The greenbelt currently contains a wetland that is separated from the Westlake River by a berm. This wetland receives overflow from the river during high flows as well as surface runoff from the surrounding suburb of Kirstenhof.

Modelled parameters are shown in Table 6-11. The river was modelled as flowing into the wetland, which covers the area shown in Figure 6-15. The schematic also shows a sediment forebay as a proposed pre-treatment mechanism for the wetland. As this is a popular dog-walking area, some areas around the wetland were left as open grass in the model.

Table 6-11: Modelled Kirstenhof Greenbelt wetland system parameters

Wetland	Slope	Surface area	WQV depth	Permanent depth	Maximum depth
Kirstenhof Greenbelt	0.1 %	30 000 m ²	0.64 m	0.9 m	1.5 m



Figure 6-15: Configuration of the modelled Kirstenhof Greenbelt wetland upgrade
(Plan view: Bing Maps, 2021; View 2: Sean Paterson, 2019)

6.3.2.8 Keyzers/Westlake confluence area

The confluence of the Keyzers and Westlake Rivers is surrounded by a large reed-marsh area of approximately 0.2 km^2 (20 ha) – a surviving part of the now diminished floodplain wetlands of this region. This area is separated from the rivers by berms and a vehicle track that provides access to mechanical vegetation harvesters when pondweed and water hyacinth overgrowths need to be removed from the rivers. A plan view of this vehicle track, as well as access points, are shown in Figure 6-16.

This reed-marsh area can be upgraded to a functional wetland by allowing the rivers to flow into the wetland by removing the berms that currently separate them. Allowing the rivers access to the wetland area will provide additional storage during high flows and will provide treatment to water flowing through it.

This area was modelled as a wetland, with both the Keyzers and Westlake Rivers flowing into it, since they have both displayed eutrophic (and sometimes hypertrophic) nutrient concentrations. This setup would require sediment forebays prior to each river entering the



Figure 6-16: Plan view of Keyzers-Westlake confluence area, with views of reed beds
(Plan view: Bing Maps, 2021; Views 1 and 2: Google Maps, 2021)

wetland, as well as require an alternative means of access to pondweed harvesters as the existing vehicle track is inundated in the model. A new vehicle track could be constructed on the other side of each riverbank, which is currently open field. These elements are displayed in Figure 6-17.

This wetland area is just upstream of the Zandvlei estuary, the water level of which is manually controlled via a sandbar and rubble weir at its outlet. Modelled parameters are shown in Table 6-12.

Table 6-12: Modelled Keyzers/Westlake wetland system parameters

Wetland	Slope	Surface area	WQV depth	Permanent depth	Maximum depth
Keyzers/Westlake confluence	0.2 %	150 000 m ²	0.92 m	1.2 m	1.9 m

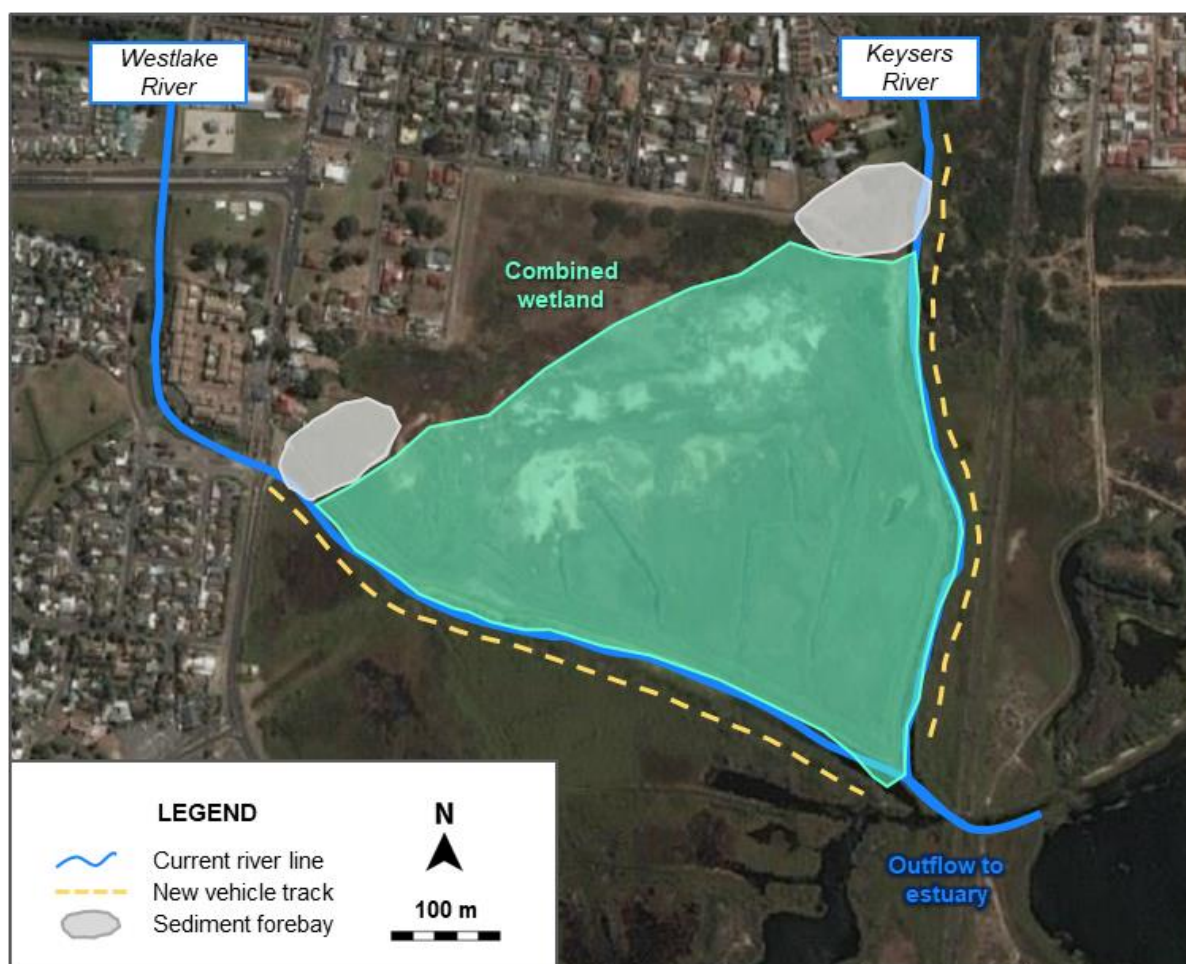


Figure 6-17: Plan view of the modelled Keyzers-Westlake confluence wetland upgrade
(Plan view: Bing Maps, 2021)

6.3.3 SuDS scenarios

The various SuDS components discussed in the previous sections were grouped and modelled as three scenarios, so that the effect of each could be compared. The first scenario was based on placing smaller-scale interventions near the suspected source of pollution, the second was based on the provision of fewer, but larger-scale measures with greater treatment ability to treat catchment-wide flows and the third was a combination of the first two scenarios.

6.3.3.1 Scenario 1: Targeted local interventions

As shown in Chapter 5, there are various locations around the study area that are suspected of contributing to elevated nutrient levels in the rivers. Scenario 1 entailed the modelling of a collection of smaller-scale SuDS interventions at these suspected locations. It is common SuDS practice to aim to reduce the pollutant load as close to the source as possible (Armitage *et al.*, 2013).

The first scenario modelled the following components (Figure 6-18):

- Constantia agricultural swales
- Steenberg golf course swales and bioretention
- Westlake golf course swales and bioretention
- Blue Route Mall stream bioretention
- Tokai industrial bioretention
- Pollsmoor Prison bioretention
- M3 swales

6.3.3.2 Scenario 2: Wetlands re-establishment

The second SuDS scenario was developed to assess the treatment capacity of regional controls in the study area. Instead of interventions placed in hotspot areas, this scenario employed large-scale SuDS measures like wetlands in the middle to lower reaches of the catchment. Since these types of SuDS interventions generally have higher space requirements, they were placed where space was available and not necessarily where the analysis identified raised nutrient levels. As stated in Section 6.1.2, the study area contains many wetland areas that are currently disconnected from the river system, thus not utilising their treatment potential. This scenario investigated re-incorporating existing wetlands into the river system by removing the berms/diversion channels that separate them.

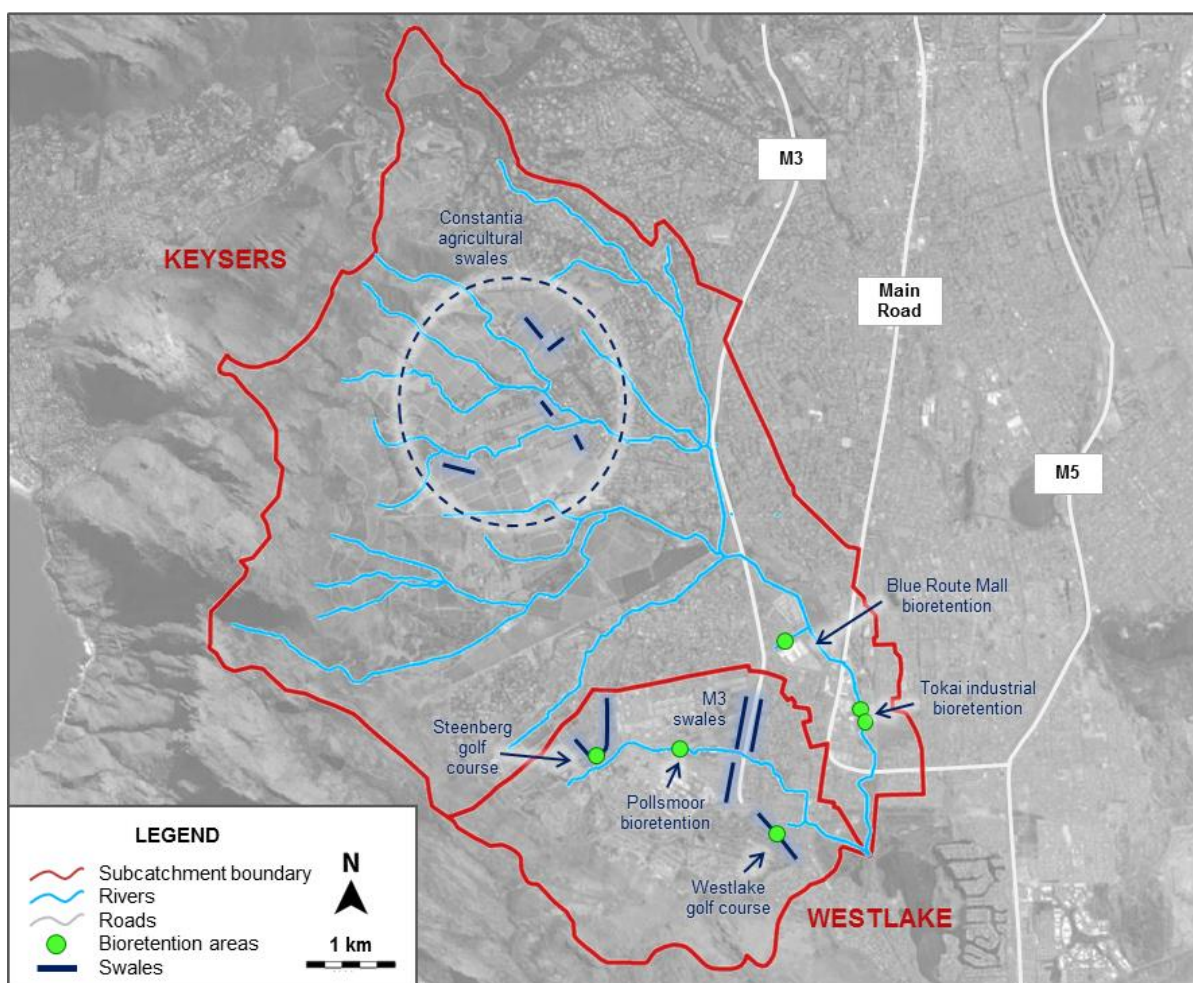


Figure 6-18: Modelled SuDS components in Scenario 1

Scenario 2 considered the use of wetlands instead of other regional controls, such as detention and retention ponds, for the following reasons:

- i) Wetlands are typically used for water quality treatment rather than controlling large volumes of stormwater (Wood-Ballard *et al.*, 2015), which makes them ideal for use in this study since the existing drainage system already provides flood control, but with minimal treatment.
- ii) Ponds are costly to construct; since wetland areas already exist in the study area, it would likely require less capital to upgrade the wetlands instead of constructing new ponds.
- iii) Many of the wetlands in this region are remnants of the original extensive floodplain wetlands that dominated this area, which have been largely eliminated due to widespread channelization, canalisation and infilling of floodplain areas (Day, 2013). Reintroducing these wetlands would assist in returning the region to its original natural state.

The second scenario modelled the following components (Figure 6-19):

- Dreyersdal wetland
- Kirstenhof Greenbelt wetland
- Keyzers-Westlake confluence wetland

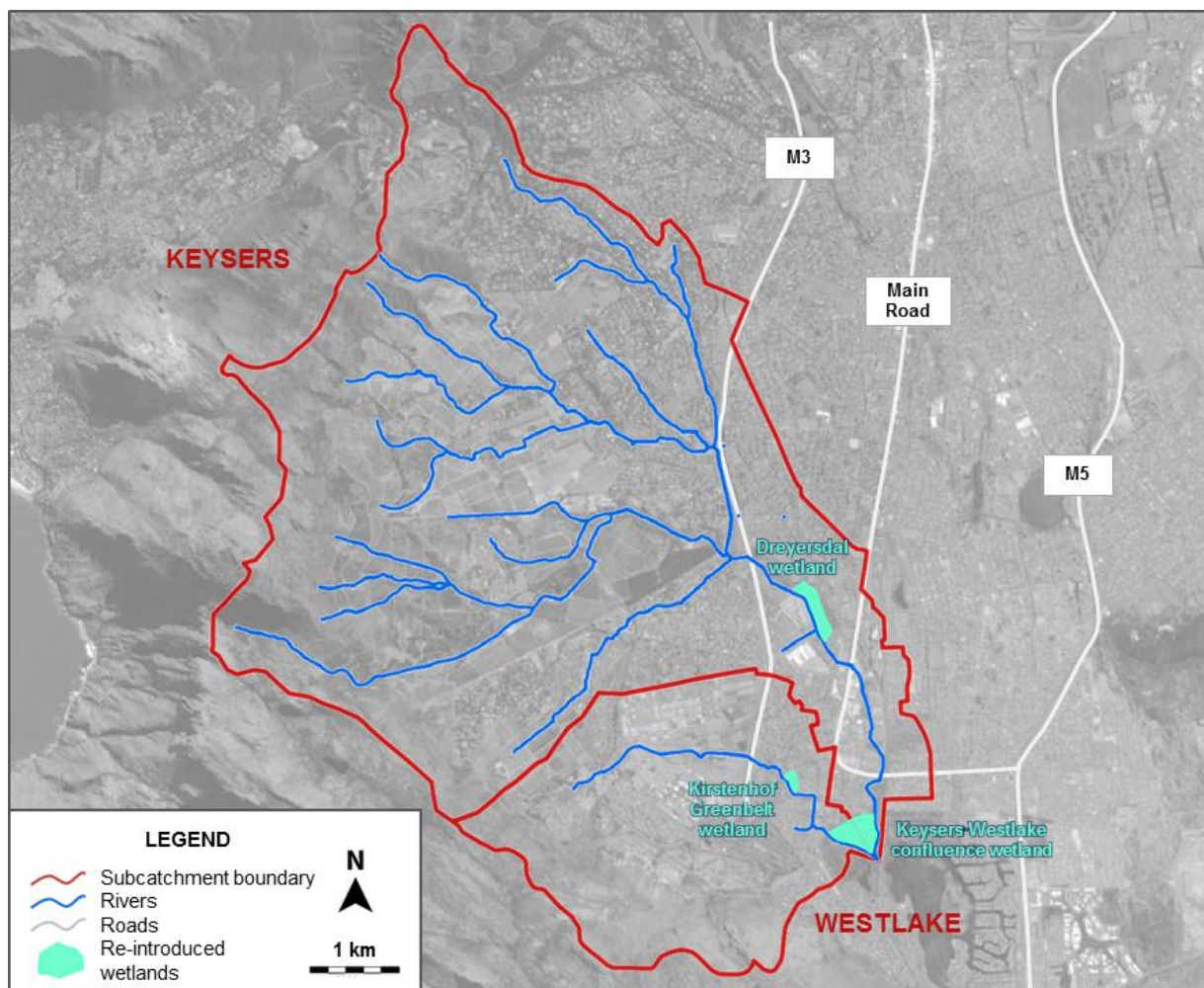


Figure 6-19: Modelled SuDS components in Scenario 2

6.3.3.3 Scenario 3: Combination scenario

The third SuDS scenario was a combination of interventions from Scenarios 1 and 2, to compare the overall effect of combining smaller and larger-scale measures (Figure 6-18). This also assisted in the comparison of the additional benefit of local controls when modelled with and without regional controls.

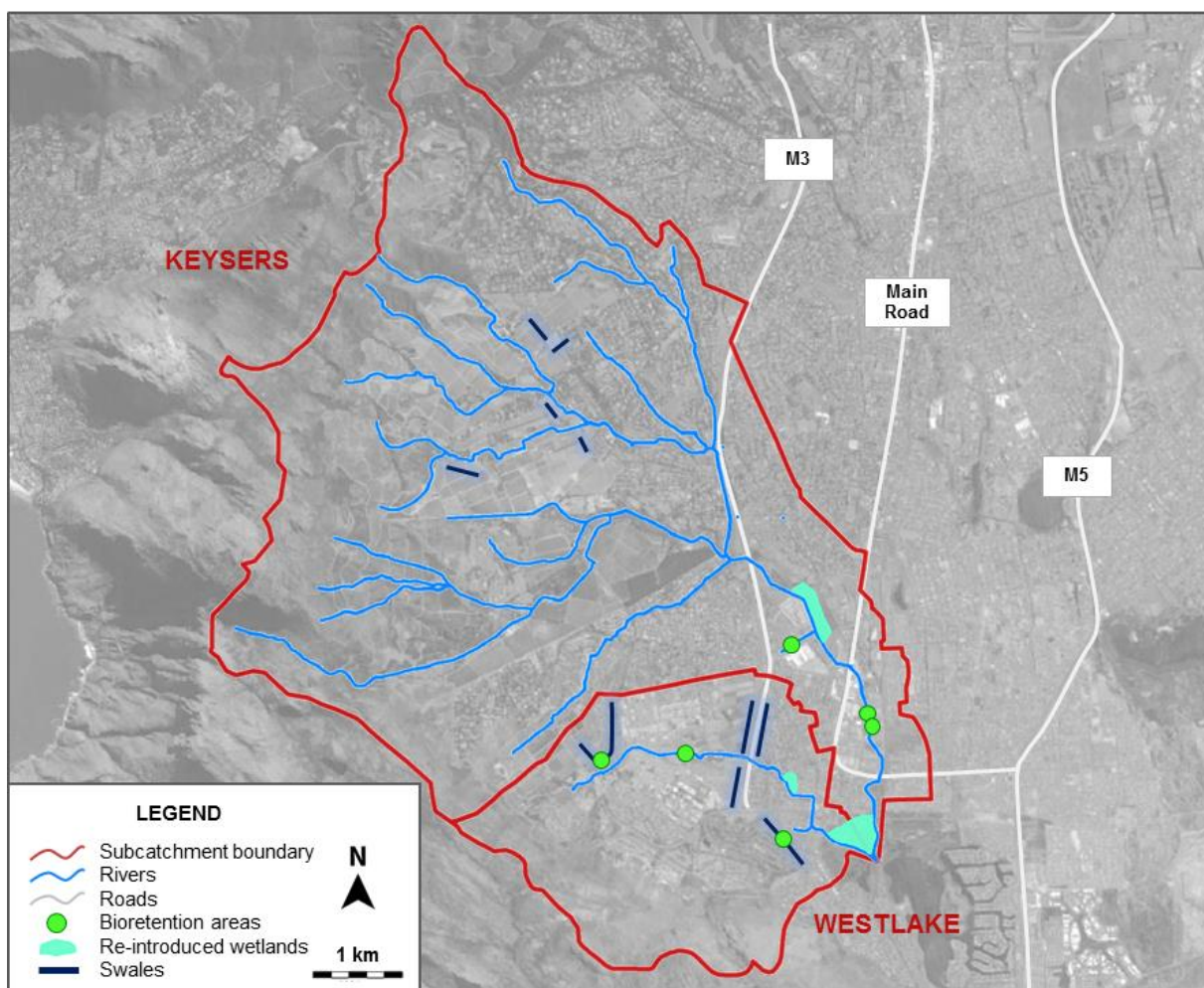


Figure 6-20: Modelled SuDS components in Scenario 3

6.4 Summary

This chapter described the various scenarios created in the model, including a scenario that represents existing catchment conditions and one that simulates pre-development conditions. Various SuDS components that were modelled were described and illustrated, after which they were grouped into three different combinations. These three SuDS scenarios were modelled to compare the relative pollutant removal and flow reduction, which will follow in Chapter 7.

7. Simulation results

This chapter discusses the results of the various scenarios modelled, with particular reference to pollutant removal and flow reduction. Model runs were completed for two different time periods:

- i) For 24 hours: The 24-hour simulation used statistical rainfall depth data for the study area (obtained from CCT – Section 4.5.2) corresponding to a 1 in 6-month, 24-hour storm, with a SCS rainfall distribution. This was chosen to estimate WQV requirements and to assess whether the modelled SuDS components met the CCT pollutant removal standards (Section 4.5.3).
- ii) For 8 years: After all the requisite data was collected, the longest continuous simulation that could be run was over an 8-year period. This simulation was used to assess the long-term impact of the modelled SuDS measures.

Wetland pollutant removal was modelled using both the upper and lower limit equations (Section 4.6.2); when results of wetland treatment or the scenarios which include wetlands are discussed, the pollutant removal will be referred to as a range. This range represents the difference between the upper and lower limit equations.

7.1 Current scenario model

The 8-year simulated time period gave the results listed in Table 7-1. The total volumetric flow and pollutant loads are modelled estimates at the outfall (where both the Keysers and Westlake Rivers enter the Zandvlei estuary), i.e. they represent what would flow into the estuary over the respective modelled time period.

Table 7-1: Modelled estimates at the outfall (for the period 2012-2019)

Parameter at outfall	8 years continuous rainfall
Total volumetric flow	28.7 × 10 ⁹ m ³
Total SRP load	9 tonnes
Total TIN load	50 tonnes
Total TP load	19 tonnes
Total TSS load	6 450 tonnes

Estimates of the flow and pollutant loads could not be compared to existing field data as no information on hypothetical 24-hour storms or annual flows or pollutant wash-off loads could be

found for the study area – nor for any other South African urban river. However, the pollutant wash-off loads were within ranges reported for urban rivers in Europe (Tockner *et al.*, 2009).

It is noteworthy that the TSS amount is less than expected. SA Aquatic guidelines suggest a TSS limit of 100 mg/ℓ; the model output gives an overall approximate concentration of 0.2 mg/ℓ – this is much lower than the suggested limit since the catchment is known to experience sedimentation problems. This is likely due to incorrect EMC values, which were estimated from international literature. The EMC values for TSS were not adjusted as there was insufficient site data to make adjustments. It is recommended that future studies focus on testing for TSS in the catchment so that more reliable EMC values can be calculated.

7.2 Pre-development and Current Scenario comparison

The Pre-development Scenario model attempted to predict the flow characteristics of the site if it were in its natural, undeveloped state. Since no data is available for this catchment prior to development the results of the simulation are a rough estimation.

A comparison of the runoff flows between the current and pre-development scenarios is presented in Figure 7-1; this figure shows the higher, steeper flow profile that is characteristic of

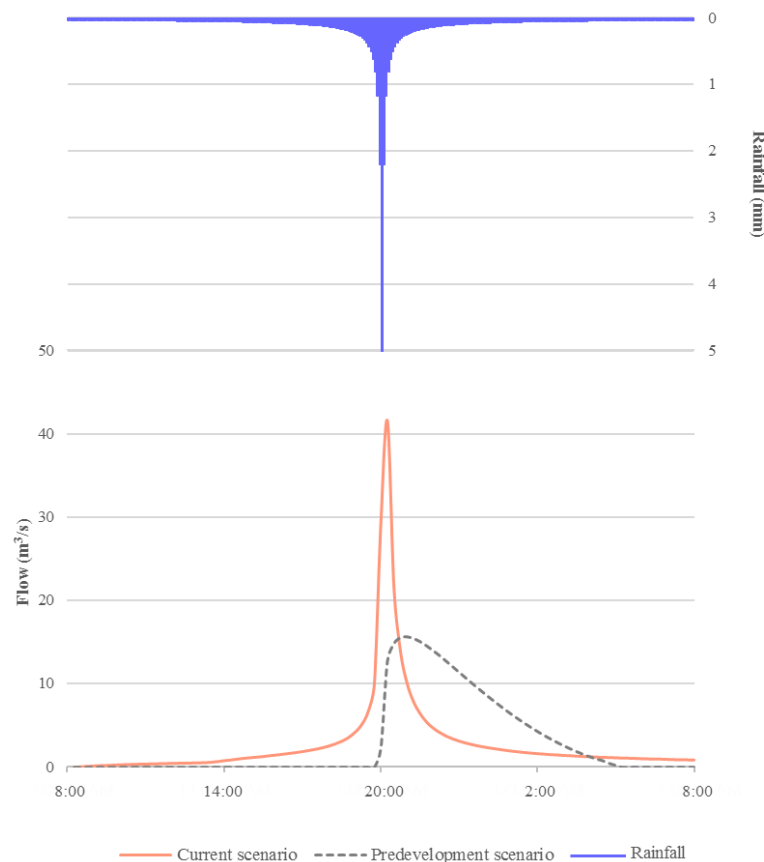


Figure 7-1: Rainfall and runoff in current and pre-development scenarios

7.3 SuDS scenarios

7.3.1 24-hour simulation results

7.3.1.1 Pollutant removal of each SuDS component

Different SuDS components are known to have varying pollutant removal capabilities; wetlands are known to have a generally high removal capacity, while filter strips have a limited treatment potential (Woods Ballard *et al.*, 2015). The pollutant removal for each modelled SuDS component for the hypothetical 24-hour storm are presented in Table 7-2.

Table 7-2: Modelled pollutant removal of SuDS components over the 1:6 month, 24-hour storm

SuDS intervention	Reduction in pollutant load (%)			
	SRP	TIN	TP	TSS
Constantia agricultural swales	50	46	49	48
Steenberg golf course swales	68	70	68	66
Westlake golf course swales	76	74	74	72
M3 freeway swales	45	43	52	43
Steenberg golf course bioretention	81	86	82	83
Westlake golf course bioretention	82	87	84	86
Blue Route Mall bioretention	54	54	54	55
Tokai industrial area bioretention	52	53	50	54
Pollsmoor bioretention	63	65	61	68
Dreyersdal wetland	49-66	50-54	45-52	52-68
Kirstenhof Greenbelt wetland	49-64	50-55	49-61	53-69
Keysers-Westlake wetland	81-88	80-83	81-87	80-89

Wetland pollutant removal was modelled based on HRT; as explained in Section 4.6.1, longer HRT typically results in greater pollutant removal. Since this simulation was only run for 24 hours, it does not represent the pollutant removal that might be expected with longer retention times. The 8-year simulation shows a larger range in wetland removal – as shown in the following sections.

The components displaying the largest removals are the bioretention systems in the Steenberg and Westlake golf courses. Each of these bioretention areas receive discharge from swales; the swales in the model would already have removed some pollutants, thus providing the bioretention areas with less polluted influent. The reason for their high removal rates could be

because they are preceded in the treatment train by a pre-treatment mechanism. This supports the idea that treatment trains provide more efficient treatment than standalone SuDS components.

CCT policy (Section 4.5.3) stipulates that for new developments, SuDS should remove 45% of TP and 80% of TSS. These figures are referred to as a guideline as this study investigates retrofitting parts of a catchment, not a new development. The modelled SuDS showed removal percentages that largely adhere to those stipulations, except for some cases of TSS removal. Where TSS removal is insufficient, additional components to assist with sediment control would be required. The SRP and TIN removal was mostly above 45%. The following section discusses SRP and TIN concentrations in relation to eutrophic limits based on South African guidelines (DWAF, 1996).

7.3.1.2 Pollutant removal of scenarios

For a 24-hour storm (1-in-6-month RI), each scenario reduced pollutant concentrations at the outfall. As expected, Scenario 1 produced much lower reductions than Scenario 2 and 3. The percentage reduction of pollutant concentrations at the outfall are displayed in Table 7-3.

Table 7-3: Percentage reduction in pollutant concentrations entering Zandvlei estuary

Scenario	Pollutant				CCT-stipulated reduction	
	SRP	TIN	TP	TSS	TP	TSS
Scenario 1	24%	24%	23% ¹	24%	45%	80%
Scenario 2	58-77%	57-63%	57-71% ²	58-77%	45%	80%
Scenario 3	80-86%	79-82%	82-85%	81-84%	45%	80%

¹ Figures shaded in red indicate non-compliance with CCT guidelines

² Figures shaded in green indicate compliance with CCT guidelines

The 24-hour storm showed eutrophic concentrations for SRP at the modelled outfall. TIN concentrations, however, were below the eutrophic threshold for all scenarios. None of the modelled scenarios reduced SRP concentrations to below eutrophic levels, but the upper treatment limit for Scenario 3 gave an SRP concentration of 0.03 mg/ℓ which is close to the threshold of 0.025 mg/ℓ. Scenario 2 and 3 were able to reduce modelled TIN concentrations more than any other pollutant. As shown in Figure 7-2, for the hypothetical 1:6 month, 24-hour storm the TIN concentrations were below the eutrophic threshold of 2.5 mg/ℓ, but this will not always be the case. The potential of Scenario 3 to remove 79-82% of the TIN concentration at the inlet to Zandvlei is a useful model output.

The concentrations at the outfall for each scenario are shown in Figure 7-2. The lighter-coloured bands on the Scenario 2 and 3 bars represent the removal difference in the wetland treatment equations.

The equations used to model the treatment of wetlands were taken from international literature, not local South African research. Furthermore, the measured nutrients were estimated to be greater than the international literature. Treatment capability is affected by many factors, including influent water quality, how well the SuDS are maintained, and – particularly for wetlands – temperature, residence time, etc. Given these factors, it is difficult to comment on how realistic these removal percentages are.

7.3.1.3 Volume and peak flow rate reduction

The SuDS scenarios had a similar effect on flow volumes and runoff rates as the pollutant loads. Figure 7-3 compares the 24-hour storm hydrograph of the three scenarios to that of the current

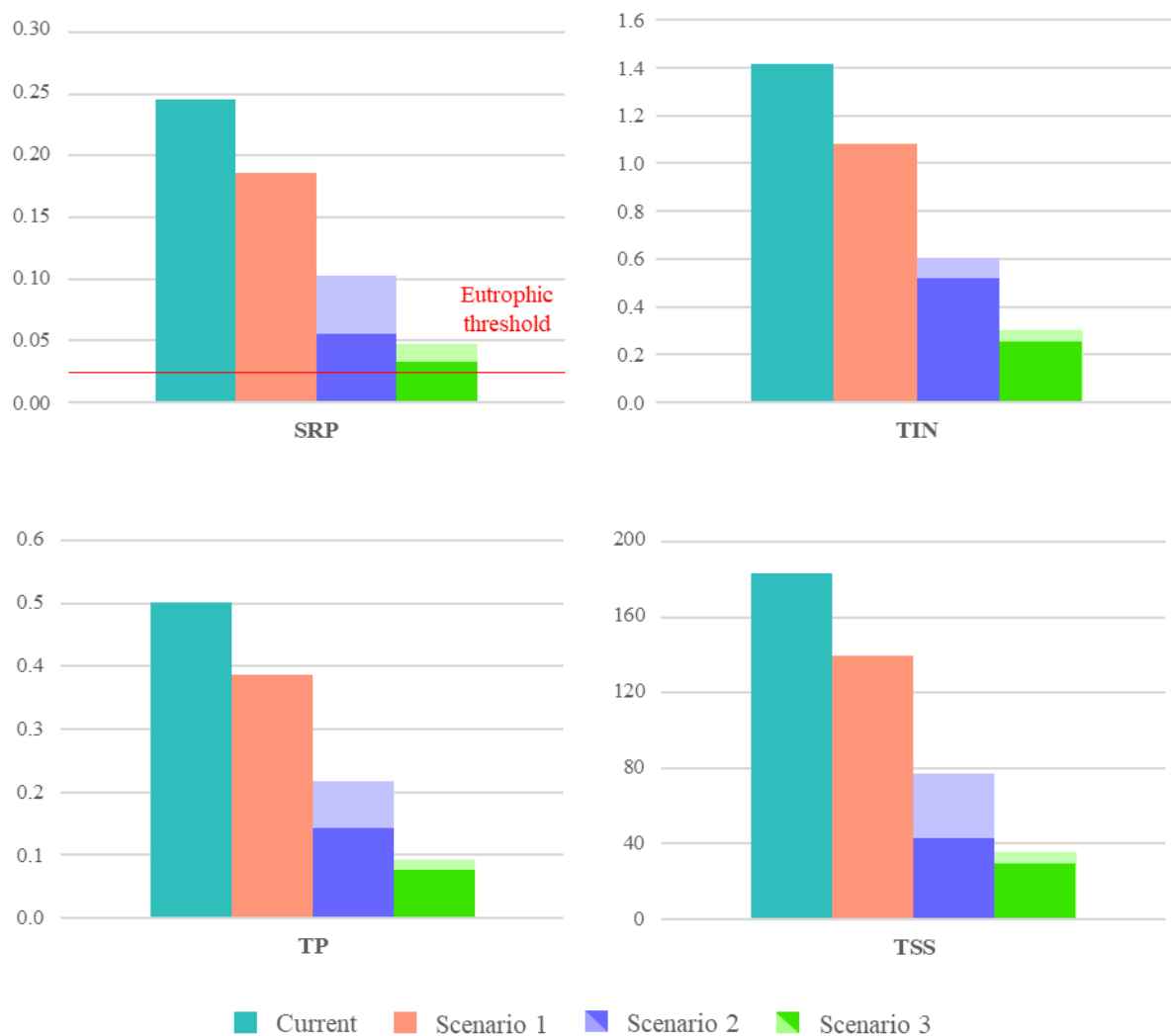


Figure 7-2: Pollutant concentrations (in mg/l) at the modelled outfall for each scenario

scenario. As expected, the combined SuDS scenario (Scenario 3) delivers the lowest peak runoff rate of 22.4 m³/s, while the peak runoff rate of Scenario 2 is slightly higher at 23.7 m³/s. The Scenario 1 modelled peak runoff rate is 32.1 m³/s, compared to the current scenario rate of 42 m³/s.

The introduction of SuDS measures should ideally result in hydrographs similar to the pre-development hydrograph; this is not the case here since the modelled SuDS scenarios have steep recession curves. This is likely due the continued presence of connected impervious areas, as the inclusion of SuDS interventions could not address all impervious runoff in the study area. Scenario 1 reduces the current scenario peak runoff by 26%, while Scenario 2 and Scenario 3 reduce it by 43% and 46% respectively. The water “lost” in this reduction is infiltrated in the model.

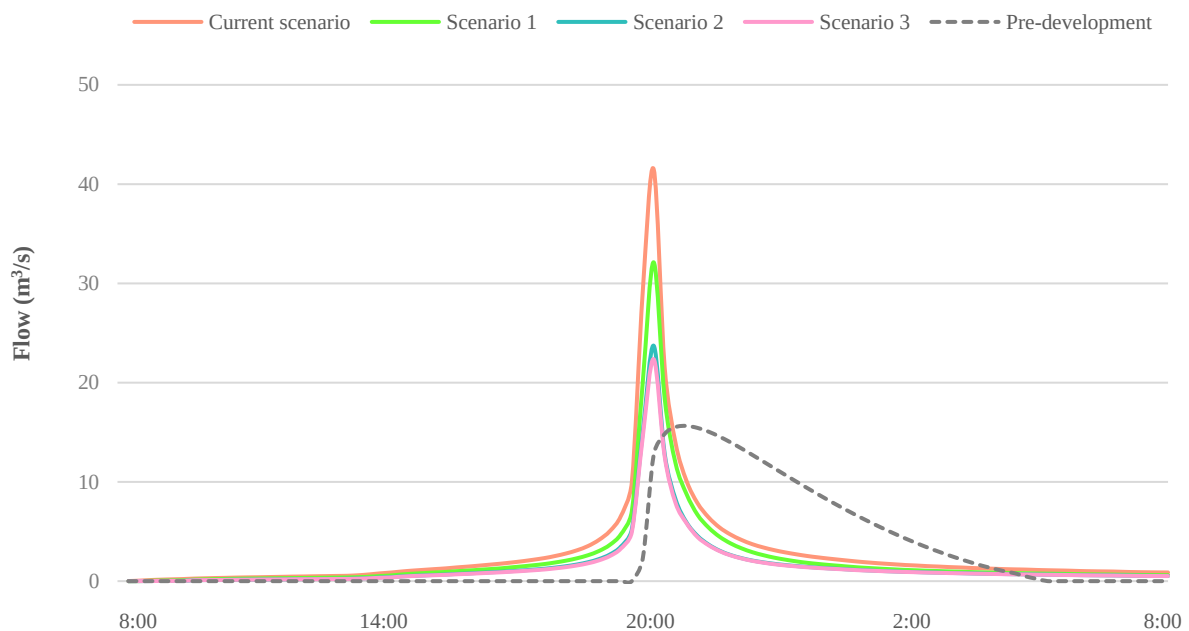


Figure 7-3: Runoff for the 24-hour storm in the current and three SuDS scenarios

7.3.2 8-year simulation results

7.3.2.1 Pollutant removal

Over the 8-year period the different scenarios showed the expected results in terms of relative pollutant reductions. Scenario 1, with its targeted local controls, showed the least pollutant removal, both in terms of concentrations and loads. Scenario 2, the scenario that modelled wetlands, showed improved reductions, but Scenario 3 (which combined the local controls and wetlands) generally showed the greatest potential removal with the exception of TSS. Scenario

2 and 3 showed similar pollutant reductions – showing that the local controls provide very little additional treatment when combined with the wetlands. The overall pollutant loads at the modelled outfall over the 8-year period are displayed in Table 7-4, followed by a graphical representation of the pollutant reduction percentages for each scenario in Figure 7-4.

Table 7-4: Pollutant loads generated by the study area from 2012-2019

Pollutant	Pollutant load at outfall (tonnes)			
	Current	Scenario 1	Scenario 2	Scenario 3
SRP	9	7	1.9 – 4	1.7 - 4
TIN	50	37	20 – 25	15 – 26
TP	19	14	3.7 – 9.1	3.1 – 9
TSS	6 450	4 830	1 220 – 3 230	1 170 – 3 060

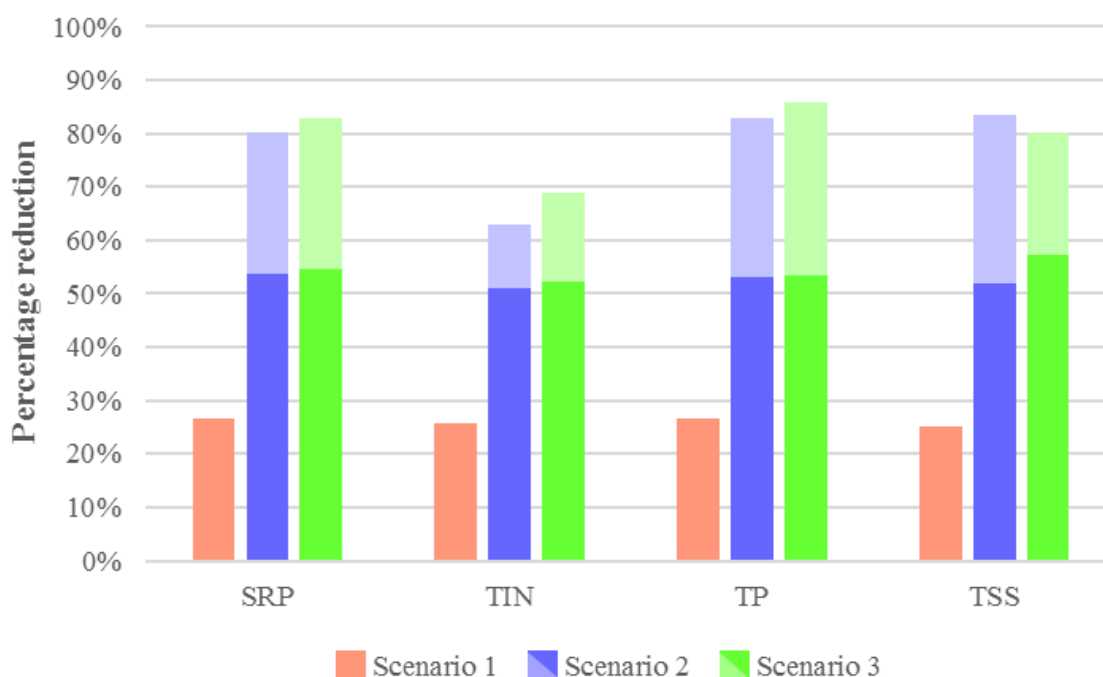


Figure 7-4: Percentage reduction in SRP, TIN, TP and TSS loads over 8-year modelling period

SRP and TIN concentrations are used as eutrophication indicators. In the current scenario model, the annual SRP concentrations at the outfall were above the eutrophic threshold every year, as shown in Figure 7-5. This corresponds to historical CCT data, which showed that SRP concentrations are frequently above the 0.025 mg/ℓ threshold.

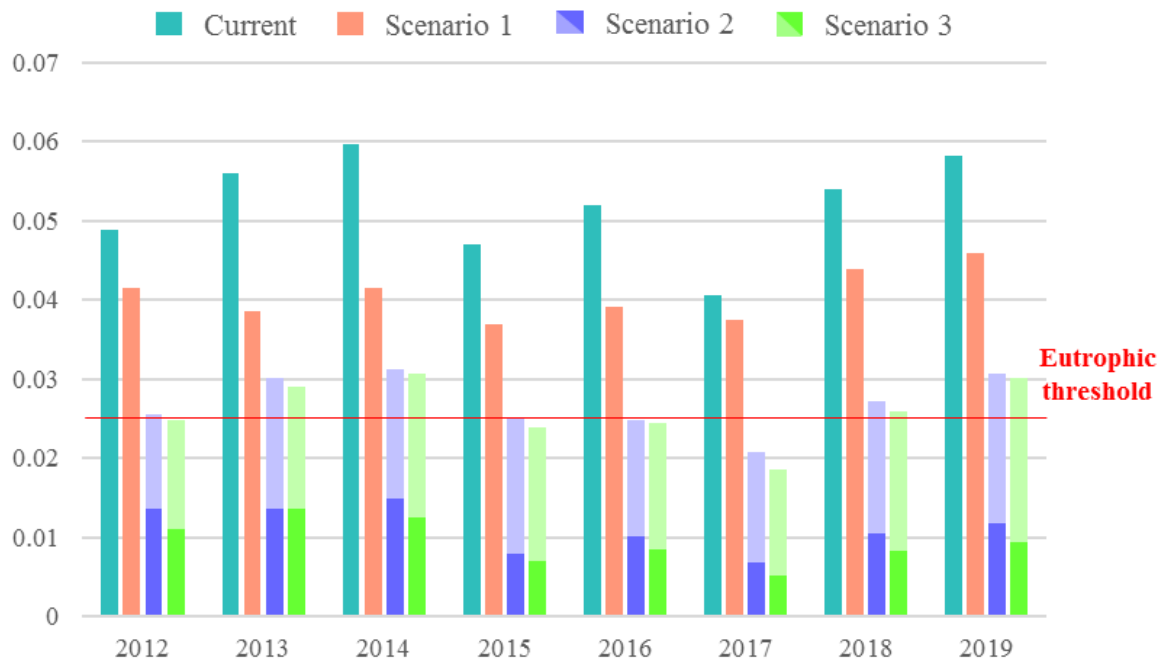


Figure 7-5: Annual SRP concentrations (in mg/l) at the outfall for each scenario

The Scenario 1 model showed reduced SRP concentrations, but not reduced below the eutrophic limit. Scenario 2 and 3 both include wetlands, so these scenarios have a range of treatment. For each scenario, the upper treatment limit shows concentrations below the eutrophic threshold every year in the simulation. The lower treatment limit, however, only shows concentrations below the eutrophic limit for the years 2015-2017. These years were also the years with the lowest rainfall and the lowest SRP loads over the 8-year period. This model shows that when the initial SRP concentrations are at, or below, 0.05 mg/l, even with low wetland treatment, it is possible to achieve SRP concentrations below the eutrophic threshold.

Scenario 1 showed the greatest variation in SRP concentration reduction over the years – this is clearly shown in Figure 7-6 which displays the annual percentage reduction in SRP concentration for each scenario.

TIN concentrations in the current scenario model were all under the 2.5 mg/l eutrophic threshold. This corresponds to the CCT historical data which seldom displayed TIN concentrations above the limit in the lower reaches of each river system. Figure 7-7 displays the annual TIN concentrations at the modelled outfall over the 8-year period. The years with the lowest rainfall, 2015-2017, once again showed the lowest concentrations. Scenario 2 and 3 show similar outfall concentrations, but Scenario 3 displays the greatest potential treatment.

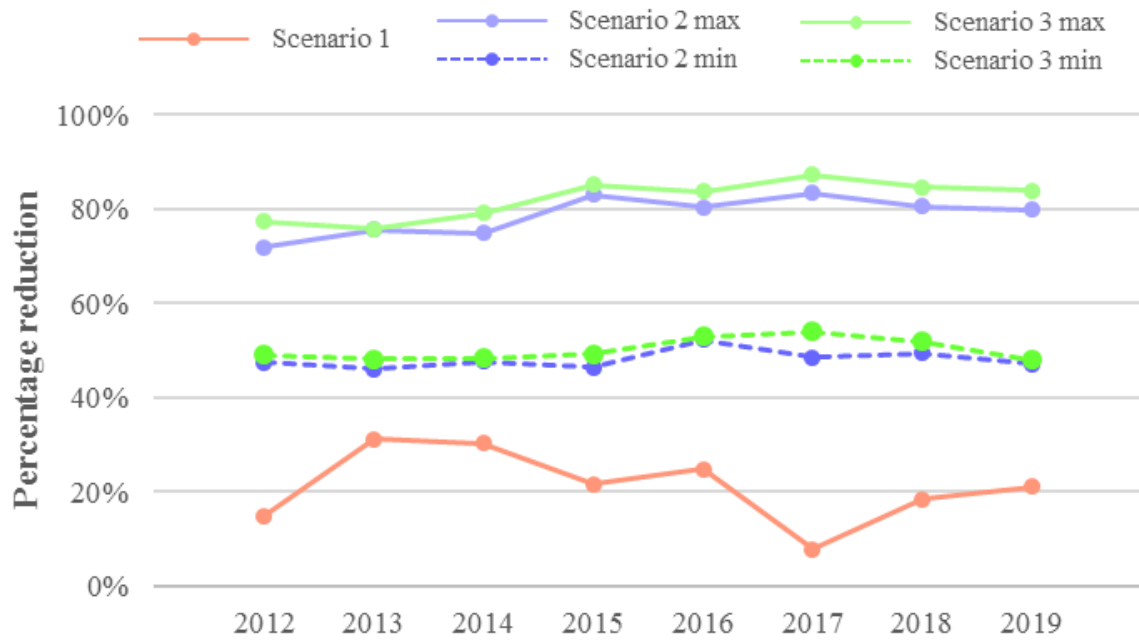


Figure 7-6: Annual SRP concentration reduction per scenario

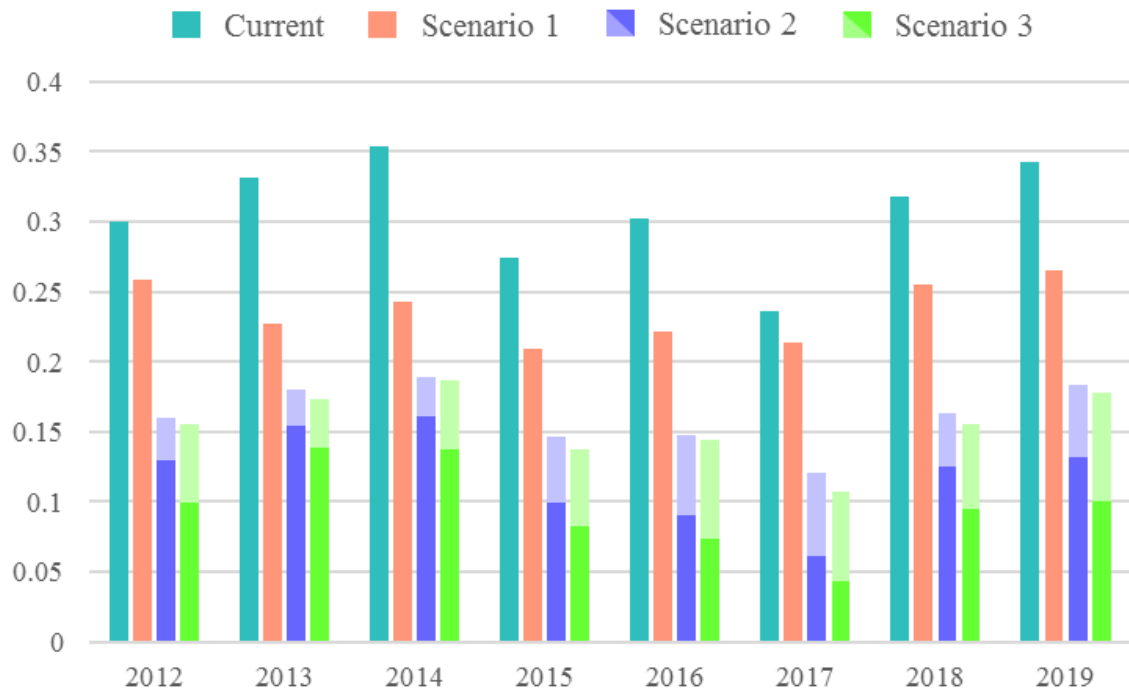


Figure 7-7: Annual TIN concentrations (in mg/l) at the outfall for each scenario

The CCT targets for SuDS development are a TP reduction of 45% and a TSS reduction of 80%. As shown in Figure 7-8, Scenario 1 did not manage to achieve these targets. For TP, the range of treatment for Scenario 2 and 3 achieved this target for most of the modelled period. In some years (2013, 2014 and 2019) the lower treatment limit removed approximately 40% of TP – just 5% short of the target. The TSS target is only achieved by the model when the wetlands in Scenario 2 and 3 are functioning at the upper treatment limit – and even then the target was only achieved from 2015 onwards.

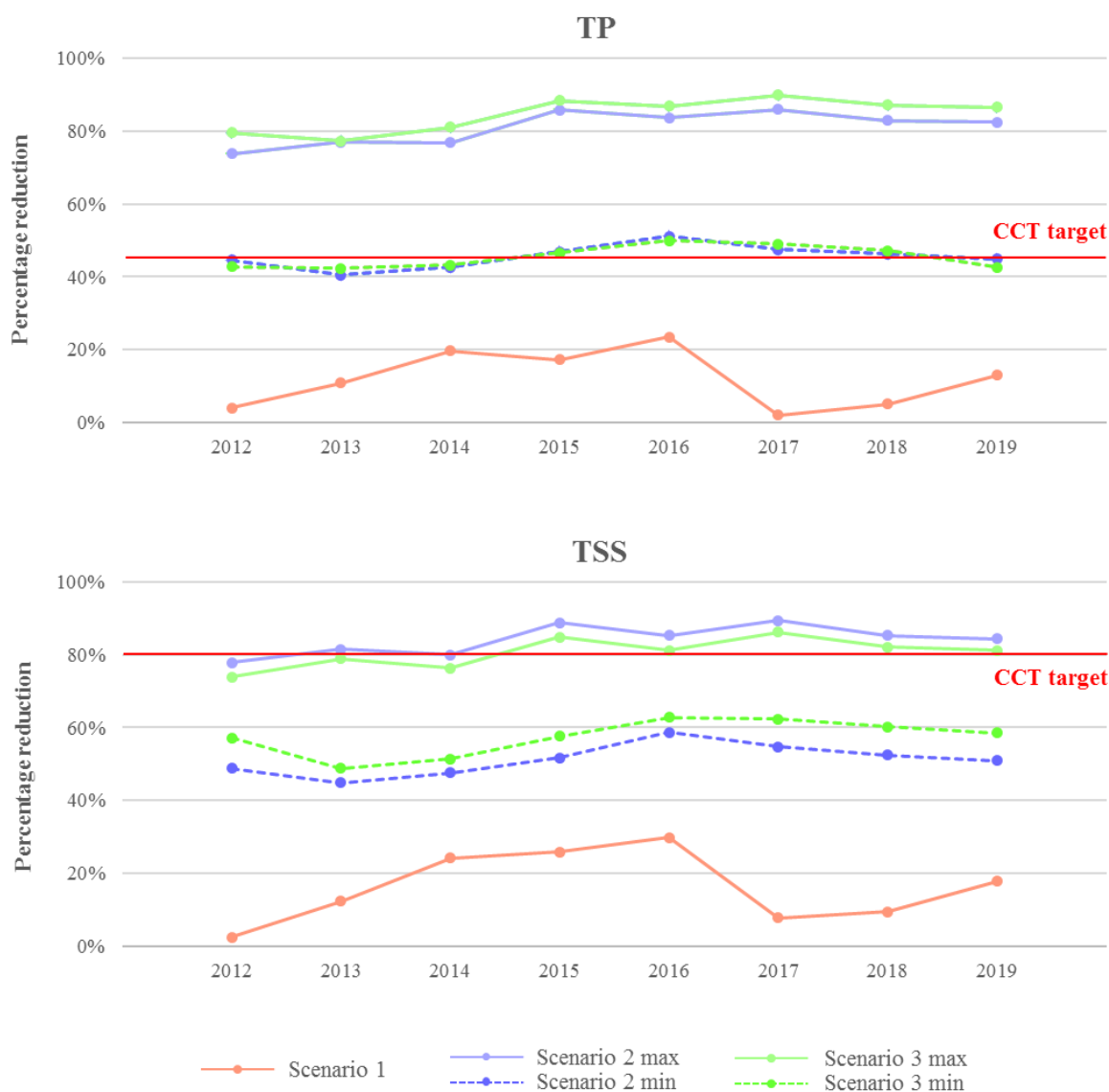


Figure 7-8: Annual percentage reduction in the TP and TSS loads at the modelled outfall

7.3.2.2 Volume and peak flow rate reduction

Over the 8-year period between 2012-2019, the various SuDS scenarios delivered peak runoff flows and total runoff volumes as displayed in Table 7-5. Overall, Scenario 1 reduced the highest peak by 18% and the total runoff by 17%. Scenario 2 reduced each by 47% and 45% respectively, while Scenario 3 showed reductions similar to Scenario 2, with 48% and 45% respectively. Each scenario had a similar effect on the peak runoff as it did on the total runoff.

Table 7-5: Peak and total runoff generated by the study area from 2012-2019

Scenario	Runoff peak		Total runoff	
	Peak (m ³ /s)	% reduction	Total (m ³)	% reduction
Current	73	-	28.7 × 10 ⁹	-
Scenario 1	60	18%	23.8 × 10 ⁹	17%
Scenario 2	39	47%	15.70 × 10 ⁹	45%
Scenario 3	38	48%	15.72 × 10 ⁹	45%

Scenario 2 and 3 showed similar results for runoff reduction; Table 7-5 shows that Scenario 2 gave the lowest total runoff of 15.70 × 10⁹ m³, while Scenario 3 showed a similar figure of 15.72 × 10⁹ m³ – a difference of 0.13%. The previous section showed that the additional treatment provided by the local controls in Scenario 3 is minor in comparison with the treatment potential of the wetlands; this appears to be the case for runoff reduction as well.

The total and peak runoff for each year in the simulated time period is shown in Figure 7-9. Each scenario reduced the annual total runoff volumes compared to the current scenario. Scenario 2 and 3 display almost identical total runoff volumes for each year, suggesting once again the minor contribution of the local controls in comparison with the wetlands.

The runoff peaks did not change as expected – in some years (2012 and 2013) Scenario 1 showed greater runoff peaks (4-6% higher) than the current scenario. The year 2013 experienced the greatest rainfall over the simulated period; the higher runoff peaks seen in Scenario 1 could be as a result of under-designed local controls. Scenario 2 and 3 showed similar runoff peaks to each other throughout the 8-year period.

When comparing total runoff to total rainfall, Figure 7-9 shows that the annual runoff across the simulated time period varied in tandem with annual total rainfall. The greater the total rainfall, the greater the reduction of total runoff between the current and SuDS scenarios. The year with the greatest modelled total runoff for each scenario was 2013, which was also the year of greatest rainfall depth. Annual runoff peaks did not always align with peak rainfall intensities. The greatest peak runoff occurred in 2014, and the highest rainfall intensity occurred in 2018.

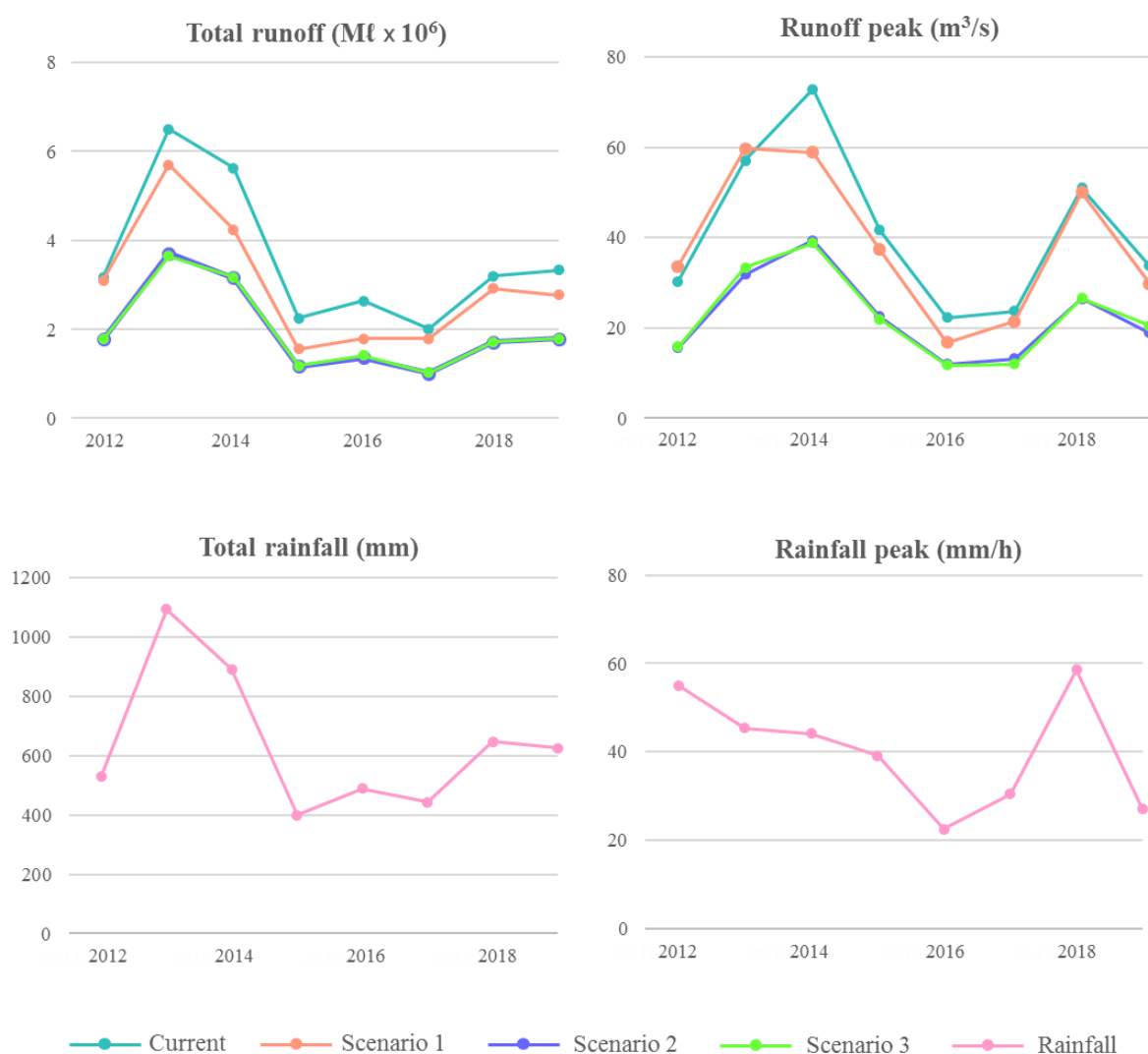


Figure 7-9: Runoff and rainfall values over across the 8-year period

7.4 Summary

This chapter displayed various outputs of the current, pre-development and SuDS scenario models. In terms of pollutant removal, Scenario 3 – the scenario that combined local controls and wetlands – showed the greatest pollutant load reduction and showed the lowest pollutant concentrations at the modelled outfall.

In terms of runoff reduction, Scenario 2 and 3 displayed similar results – possibly suggesting that the local controls in Scenario 3 contribute minimally in comparison to the wetlands.

8. Conclusions and recommendations

The objectives of this study were to investigate concentration and origin of nutrients in two of the rivers that drain into the Zandvlei estuary and to investigate the effect that SuDS interventions might have on water quality in the study area by creating a coupled hydrological, hydraulic and water quality model.

The initial objective – investigating water quality in the study area – was carried out via a sampling programme and examining historical data based on monthly grab samples. This exercise resulted in the identification of areas of concern suspected of raising nutrient levels. In both river systems SRP concentrations were frequently above the eutrophic threshold. TIN concentrations were higher in the Westlake system than the Keysers system; at Site CR03 in the Westlake system approximately half the historical samples over the 8-year period display above-eutrophic levels.

The second objective of this study was to investigate the effect that SuDS interventions might have on water quality in the study area by modelling various SuDS measures in PCSWMM – a widely available and well-supported software package. Three scenarios were modelled. The first scenario comprised smaller-scale interventions (local controls) that targeted the areas of concern. The second scenario employed the use of large-scale measures, using the existing, but disconnected, wetlands in the study area and reintroducing them into the river systems. The third scenario comprised a combination of components from Scenario 1 and Scenario 2.

The results of the modelling exercise showed that while all three scenarios delivered lower pollutant loads, smaller peak runoff rates and less total runoff, there was a clear difference between Scenario 1 compared to Scenario 2 and 3. The latter two scenarios achieved similar pollutant removal results and near-identical peak flow and volume reduction, even though Scenario 3 contained many more SuDS components. This suggests that the additional pollutant removal and runoff reduction achieved through targeting limited source areas is minimal compared to that achieved by constructing large-scale regional controls like wetlands.

Based on the modelling exercise, Scenario 2 would be recommended as the best option for implementation; since the study area already contains natural wetland infrastructure, it would arguably be the most cost-effective alternative for the given amount of pollutant reduction. The upper treatment limit equations showed annual SRP and TIN concentrations at the modelled outfall reduced to below the eutrophic threshold for three years of the 8-year simulation period; the years with lower rainfall coincided with the years in which concentrations were reduced to below the eutrophic limit. Further measures will be required to reduce SRP concentrations in years with high rainfall. The CCT requires a TP reduction of 45% and a TSS reduction 80% (for new developments) – for TP, the upper treatment limit (that delivers greater pollutant reduction) achieved the CCT guideline throughout the simulation period, while the lower treatment limit only achieved the CCT guideline for some years. However, the TSS requirement was only achieved by the higher treatment limit equation (in most years, but not all). This means an 80%

TSS load reduction is only possible in the model when the wetlands are at their most efficient. To achieve all the required targets, an additional wetland could be added to the system; alternatively, the efficiency of the modelled wetlands could be improved by accounting for the other factors that control wetland treatment which were not accounted for in these equations – such as changes in temperature, hydraulic loading rate, reed maintenance, etc. (Section 4.6.2).

There are many factors that were not considered because they fell outside the scope of this study. Were any of the modelled SuDS measures to be considered for construction, the relevant feasibility studies and site inspections would need to be conducted beforehand. The water quality study was brief and only considered the nutrients mentioned in this report – perhaps a more detailed water quality study would be required. Furthermore, point sources of pollution were not studied and accounted for.

Various challenges were faced in this research study; in relation to furthering this research, the following recommendations are made:

- Conducting studies on the existing SuDS measures in the study area to assess treatment capacity – such as the Pollsmoor Pond, various smaller ponds in the Keysers river system and the existing wetlands mentioned in this study (during high flows).
- Establishing permanent water level sensors in the study area; this data would prove useful for future model calibration exercises.
- Creating models that focus on smaller areas in the catchment; this will allow for the incorporation of greater detail, as there will not be a trade-off between model resolution and achieving reasonable run times.

The following are recommended in terms of creating a larger pool of locally-relevant data for similar research in the future:

- Studies on EMC values in local urban catchments.
- Studies into the modelling of SuDS and the best methods to use for a South African context with particular reference to which mechanisms/tools in specific software to use for which purposes – with the aim of creating standards/best practice for SuDS modelling in South Africa.
- Monitoring and modelling studies on the removal capacity of wetlands and other SuDS measures, and how removal is affected by different variables – for the purpose of developing a range of applicable methods of modelling the treatment of different SuDS measures in South Africa.

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Appendix A: List of rain gauge data collected

Table A-1: Collected rainfall records

Rain Gauge	Source	Interval	Record length (years)	Record period
KEYS05FR	CCT	5-minute	8	2012-2019 ¹
DIEP05AR	CCT	5-minute	8	2012-2019
Groot Constantia	ARC	Daily	8	2012-2019
Corsair Crescent	Private	Monthly	43	1977-2020
Vineyard Road	Private	Monthly	41	1979-2020
Yorkshire Close	Private	Monthly	26	1994-2020
Clunie Road	Private	Monthly	26	1994-2020
William Frederick Road	Private	Monthly	18	2002-2020
Bryn Road	Private	Monthly	12	2008-2020
Gilmour Circle	Private	Monthly	7	2013-2020
Wagtail Close	Private	Monthly	7	2013-2020
Cnr Aliwal & Ebenezer	Private	Monthly	2	2018-2020
Drosdty Road	Private	Daily	11	2009-2020
Bunker Road	Private	Daily	2	2018-2020

¹ Shaded items were included in the analysis and shown in Figure 4-6.

Appendix B: Preliminary water quality data

Electrical conductivity around the study area was tested during the preliminary testing phase prior to water quality sampling. The testing points are shown in Figure B-1 below, and the EC readings at these locations are displayed in Table B-1.

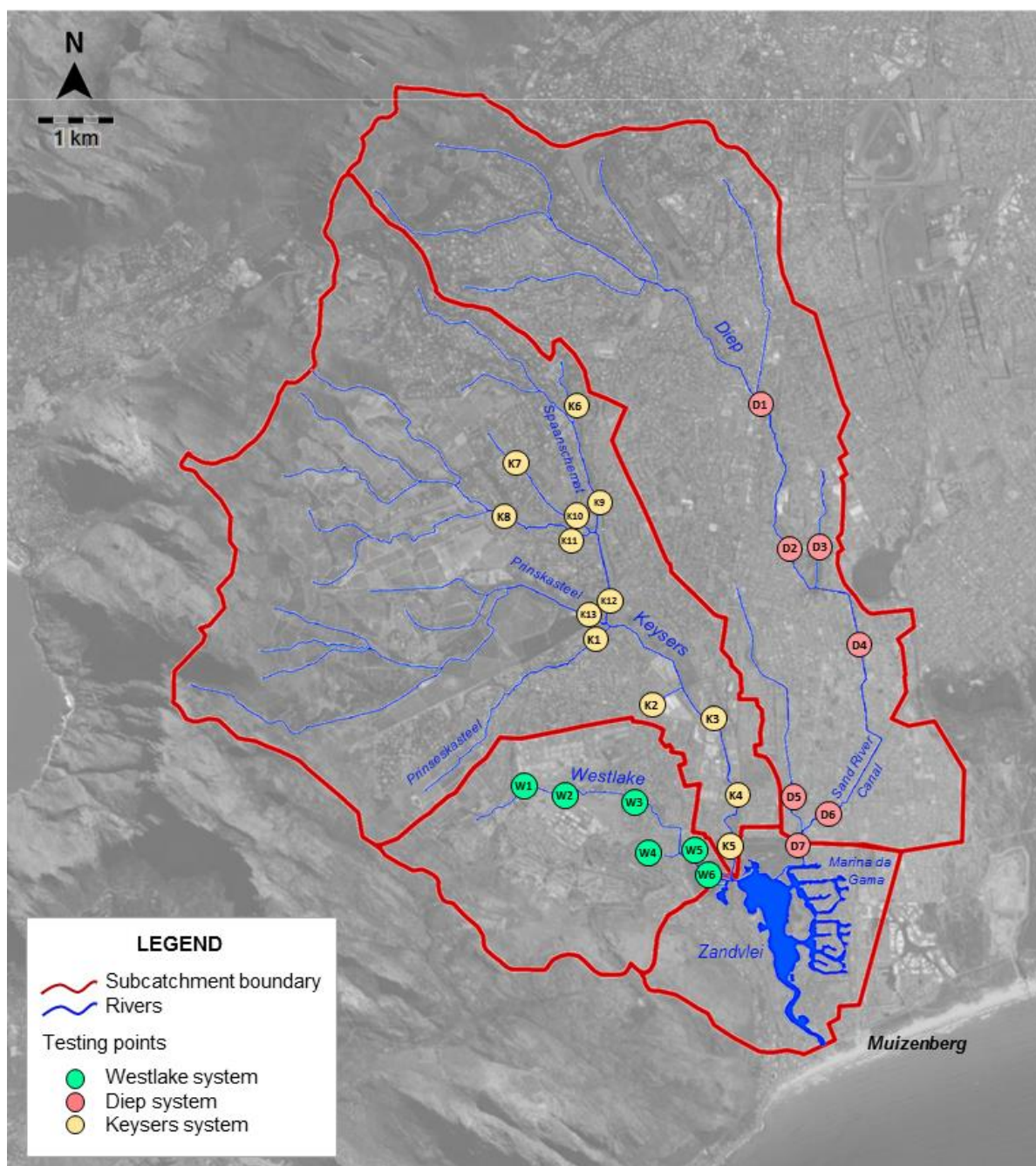


Figure B-1: Preliminary testing points for electrical conductivity (EC)

Table B-1: Minimum, maximum and mean EC measurements for the Zandvlei catchment

River system	Testing point	EC measurements ($\mu\text{S}/\text{cm}$)		
		Minimum	Max	Mean
Diep	D1	376	486	431
	D2	408	451	430
	D3	594	671	633
	D4	419	422	421
	D5	688	787	738
	D6	423	501	462
	D7	508	522	517
Keyzers	K6	453	596	525
	K7	412	608	510
	K8	405	632	519
	K9	382	638	510
	K10	403	615	509
	K11	415	611	513
	K12	449	693	591
	K13	568	674	612
	K1	564	857	699
	K2	635	843	726
	K3	442	1070	618
	K4	561	1005	832
	K5	1167	1434	1268
	W1	930	1300	1053
Westlake	W2	823	1239	986
	W3	661	1000	743
	W4	1009	2000	1425
	W5	772	1917	1170
	W6	945	1296	1116

Appendix C: Published EMC data

As stated in Section 4.4.5, EMC values were derived from a variety of literature sources. These values and respective sources are listed in Table C-1.

Table C-1: EMC values from literature (in mg/l)

	Guerard & Weiss (1995)	Lin (2004)	Line <i>et al.</i> (2002)	Kayhanian <i>et al.</i> (2003)	King <i>et al.</i> (2007)	Edwards (2009)	DEFRA (2013)	Mean
TOxN								
Residential	0.59	0.44	0.79				0.98	0.70
Golf course			1.02					1.02
Industrial	0.88	0.48	0.46				0.6	0.60
Agricultural			1.30					1.30
Commercial	0.73	0.65					0.60	0.66
Open space	1.20	0.91					0.84	0.98
Freeways				1.10			0.81	0.96
NH3-N								
Residential	0.49	0.46	0.55					0.50
Golf course			0.79					0.79
Industrial	0.43	0.38	0.25				0.56	0.41
Agricultural			0.31					0.31
Commercial	0.60	0.30					0.56	0.49
Open space	1.30	0.11					0.10	0.50
Freeways				1.10				1.10
TIN								
Residential	1.08	0.90	1.34				0.98	1.08
Golf course			1.81					1.81
Industrial	1.31	0.86	0.71				1.16	1.01
Agricultural			1.61					1.61
Commercial	1.33	0.95					1.16	1.15
Open space	2.50	1.02					0.94	1.49
Freeways				2.20			0.81	1.51

TP								
Residential	0.75	0.25	0.59				0.41	0.50
Golf course			1.07					1.07
Industrial	0.30	0.50	0.27				0.30	0.34
Agricultural			2.14					2.14
Commercial	0.28	0.40					0.30	0.33
Open space	0.60						0.22	0.41
Freeways				0.30			0.28	0.29
SRP								
Residential							0.19	0.19
Golf course					0.15			0.15
Industrial							0.16	0.16
Agricultural						0.40		0.40
Commercial							0.16	0.16
Open space							0.06	0.06
Freeways				0.10			0.18	0.14
TSS								
Residential	229	72	73					125
Golf course			202					202
Industrial	220	404	231				50	226
Agricultural			151					151
Commercial	284	49					50	128
Open space	846						126	486
Freeways							132	132

Appendix D: Model input parameters

Construction of the hydrological model requires the input of various parameters, as described in Section 4.3. Model inputs derived from various datasets is shown in the tables below.

Table D-1: Typical ranges of sub-catchment characteristics per land use
(from Palla & Gnecco, 2015; Krebs *et al.*, 2014; McCuen *et al.*, 1996)

Land use	Depression storage (mm)	Manning's <i>n</i> for overland flow	
		Pervious	Impervious
Commercial	0.5-1.5	0.15-0.24	0.011-0.015
Conservation and nature	5-7.5	0.05-0.41	N/A
Institutional (Education, Health, etc)	0.5-2.5	0.15-0.24	0.011-0.015
Golf course	5-7.5	0.15-0.41	0.012-0.015
Industrial	0.5-1.5	N/A	0.012-0.015
Freeway/road	0.5-1.5	N/A	0.011-0.013
Residential – high density	1.5-5	0.15-0.41	0.015-0.02
Residential – low density	1.5-5	0.15-0.41	0.015-0.02
Agriculture	4-7.5	0.05-0.25	N/A
Undeveloped urban land (empty sand areas)	0.5-2.5	0.05-0.06	N/A

Table D-2: Properties of soil types in the study area (from Rawls *et al.*, 1983)

Soil type	Hydraulic conductivity (mm/hr)	Suction head (mm)	Porosity (fraction)	Field capacity (fraction)	Wilting point (fraction)	Initial moisture deficit (mm)
Sand	120.34	49.02	0.437	0.062	0.024	0.413
Loam	3.30	88.90	0.463	0.232	0.116	0.347

Table D-3: Initial and calibrated groundwater equation coefficients

Coefficient	A1	A2	A3	B1	B2
Initial estimate from CHI (2014b)	1	0.1	0	2	0.1
Calibrated value	0.006	0.0006	0	1.318	2.223

Appendix E: SuDS design standards and assumptions

This appendix lists the design specifications adhered to during the basic SuDS design process, based on guidelines set out by Woods Ballard *et al.* (2015) and Armitage *et al.* (2013).

E-1: Water Quality Volume (WQV)

Armitage *et al.* (2013) describe three ways of calculating the WQV; this research used the 1 in 6-month recurrence interval 24-hour storm based on historical CCT MAP data. The calculation process resulted in rainfall depths varying between 24-47 mm across the study area. SuDS components were sized to carry the peak runoff flow rate/volume associated with this storm depth.

E-2: Swale design

E-2.1 General design requirements

- Minimum base width: 500 mm
- Maximum side slopes: 33%
- Maximum longitudinal slope: 6%
- Swales with longitudinal slopes > 3% will require check dams
- Maximum water flow height: 100 mm
- Maximum velocity for WQV storm: 0.3 m/s
- All swales should be able to carry the peak flow delivered by the WQV storm

E-2.2 Equations used

The Manning equation was used to calculate velocity (Equation E-1) and flow (Equation E-2).

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \quad (\text{E-1})$$

$$V = \frac{R^{2/3}S^{1/2}}{n} \quad (\text{E-2})$$

where Q = flow (m^3/s); V = flow velocity; A = cross-sectional flow area; R = hydraulic radius; S = slope and n = Manning's roughness coefficient.

E-2.3 Design assumptions

The Manning roughness coefficient (n) was assumed to be 0.25 for all swales, except the swales with additional vegetation near to streams where $n = 0.35$ (based on recommendations by Woods Ballard *et al.* (2015)).

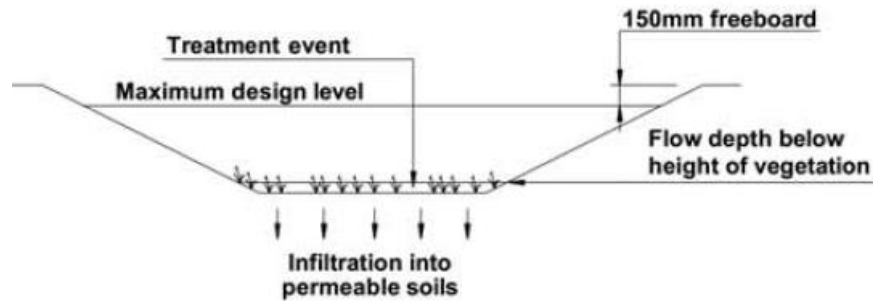


Figure E-1: Cross-section of a typical conveyance swale
(Woods-Ballard *et al.*, 2007)

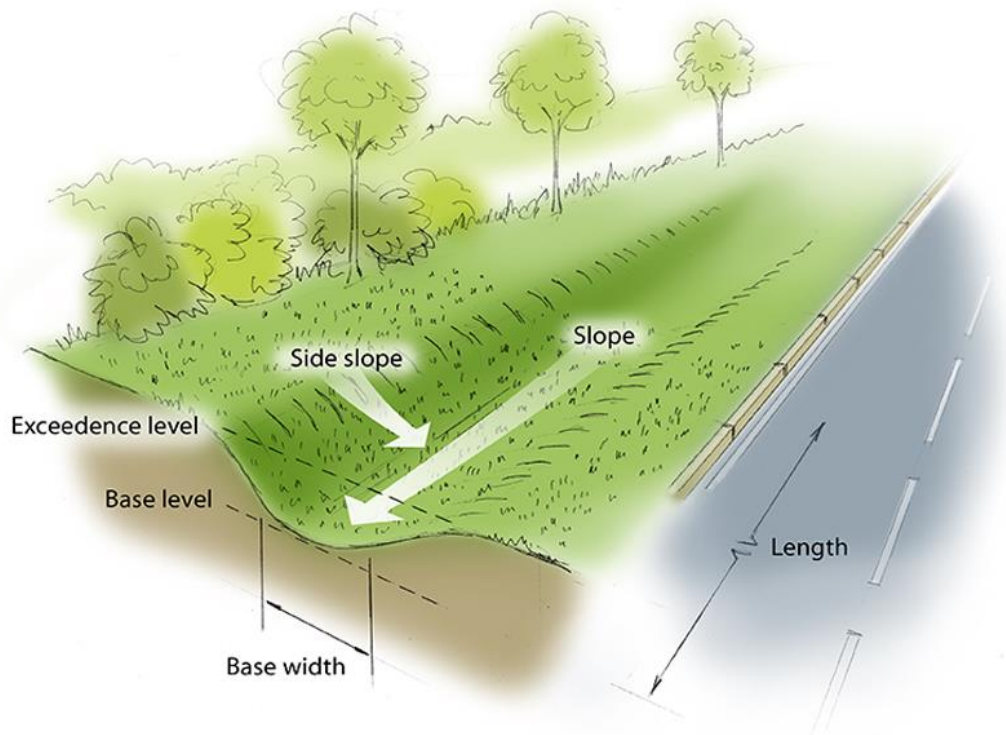


Figure E-2: Schematic of a typical conveyance swale
(XPDrainage, 2018)

E-3: Bioretention system design

E-3.1 General design requirements

- Sufficient area temporarily to store WQV at a depth 150 mm on the surface
- WQV should be able to half drain in 24 hours
- Minimum soil bed depth: 1 m
- Minimum width: 3 m
- Minimum depth to groundwater: 1 m (if unlined)

E-3.2 Equations used

The minimum required bioretention surface area was calculated with Equation E-3 and checked with Equation E-4, which is used to calculate the maximum water height for a given bioretention surface area, both from Woods Ballard *et al.* (2015).

$$A_f = \frac{V_t L}{k (h + L)t} \quad (\text{E-3})$$

$$h_{max} = \frac{D (R i - q)}{n} \quad (\text{E-4})$$

Where A_f = surface area of filter bed (m^2); V_t = volume of water to be treated (m^3); L = filter medium depth (m); k = coefficient of permeability (m/s); h = average height of water (m); t = time required for WQV to percolate through; and

h_{max} = maximum depth of water (m); D = storm duration of design storm; R = ratio of drained area to infiltration area; q = infiltration coefficient (m/h); i = rainfall intensity (m/h); and n = porosity of fill material.

E-3.3 Design assumptions

Since sand is the underlying soil in all areas where bioretention systems have been implemented, soil properties used for the equations listed above were assigned based on the properties of sand, as follows:

- Coefficient of permeability (k): Das (1990) list the range of sand permeability as between 0.01 and 1 m/s. For this study, a conservative estimate of 0.01 m/s was used (the lower limit of the range).

- Infiltration coefficient (q): Woods Ballard *et al.* (2015) suggest that on site infiltration tests be conducted to determine the infiltration capacity of the filter medium. This was not possible for this study; instead the infiltration coefficient specified by the Minnesota Stormwater Manual for sand/loamy sand was used: 0.02 m/h (MPCA, 2020). Using a safety factor of 10%, the value used in the calculation was $q = 0.002$ m/h.

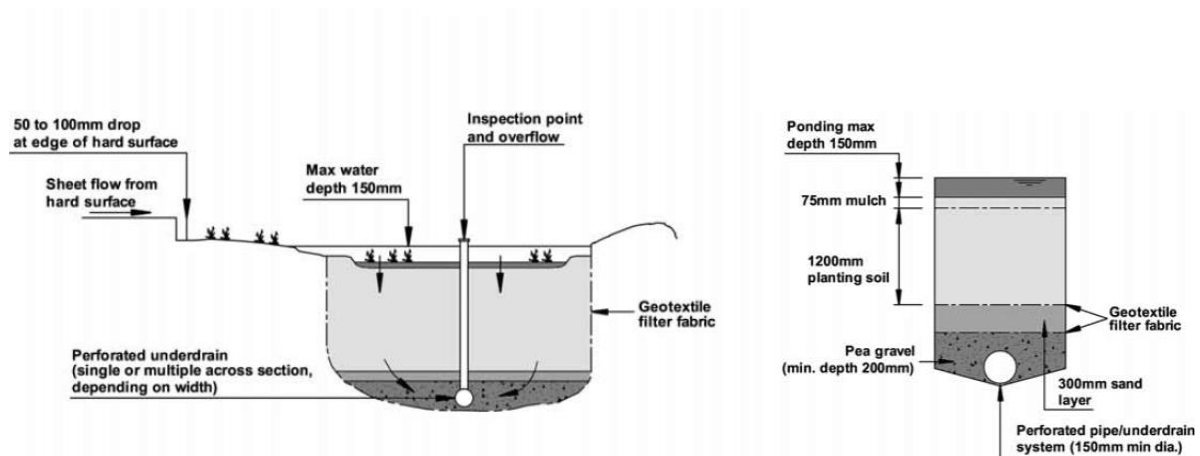


Figure E-3: Profile and section views of typical bioretention areas
(Woods-Ballard *et al.*, 2007)

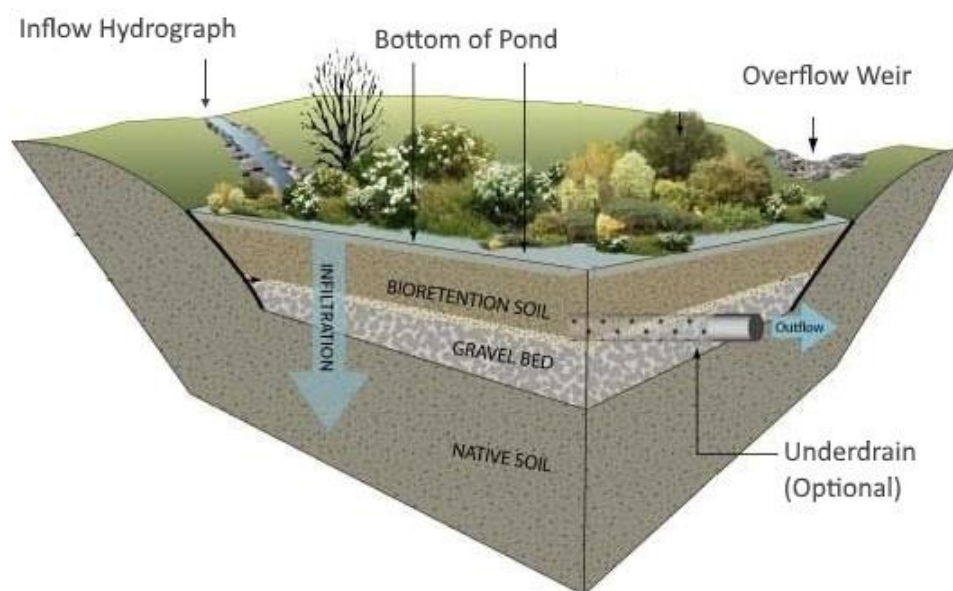


Figure E-4: Schematic of a bioretention system
(Hydrology Studio, 2020)

E-4: Wetland design

E-4.1 General design requirements

- Minimum length:width ratio: 3:1
- Shallow sides slopes
- WQV may be managed by splitting it between several components in series
- Recommended bathymetry (for temporary storage above permanent water level):
 - 0.0-0.5 m – 50% of wetland
 - 0.5-1.0 m – 30% of wetland
 - 1.0-1.5 m – 20% of wetland

E-4.2 Sizing and modelling assumptions

The existing wetlands are not large enough to each manage the influent WQV, but they were checked to ensure that the total WQV could be accommodated by all the wetlands together. According to DEM data, depths in the wetland areas vary between approximately 0.5-2 m deep – wetland capacity calculations were based on these depths and the recommended bathymetry described above.

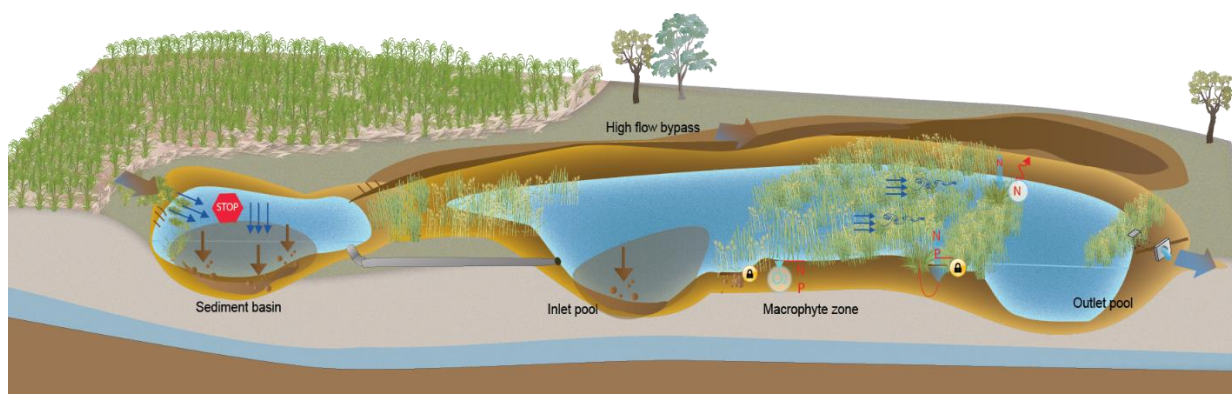


Figure E-5: Schematic of a typical constructed wetland
(Queensland Department of Environment and Science, 2018)

Appendix F: Inlet and outlet structure examples

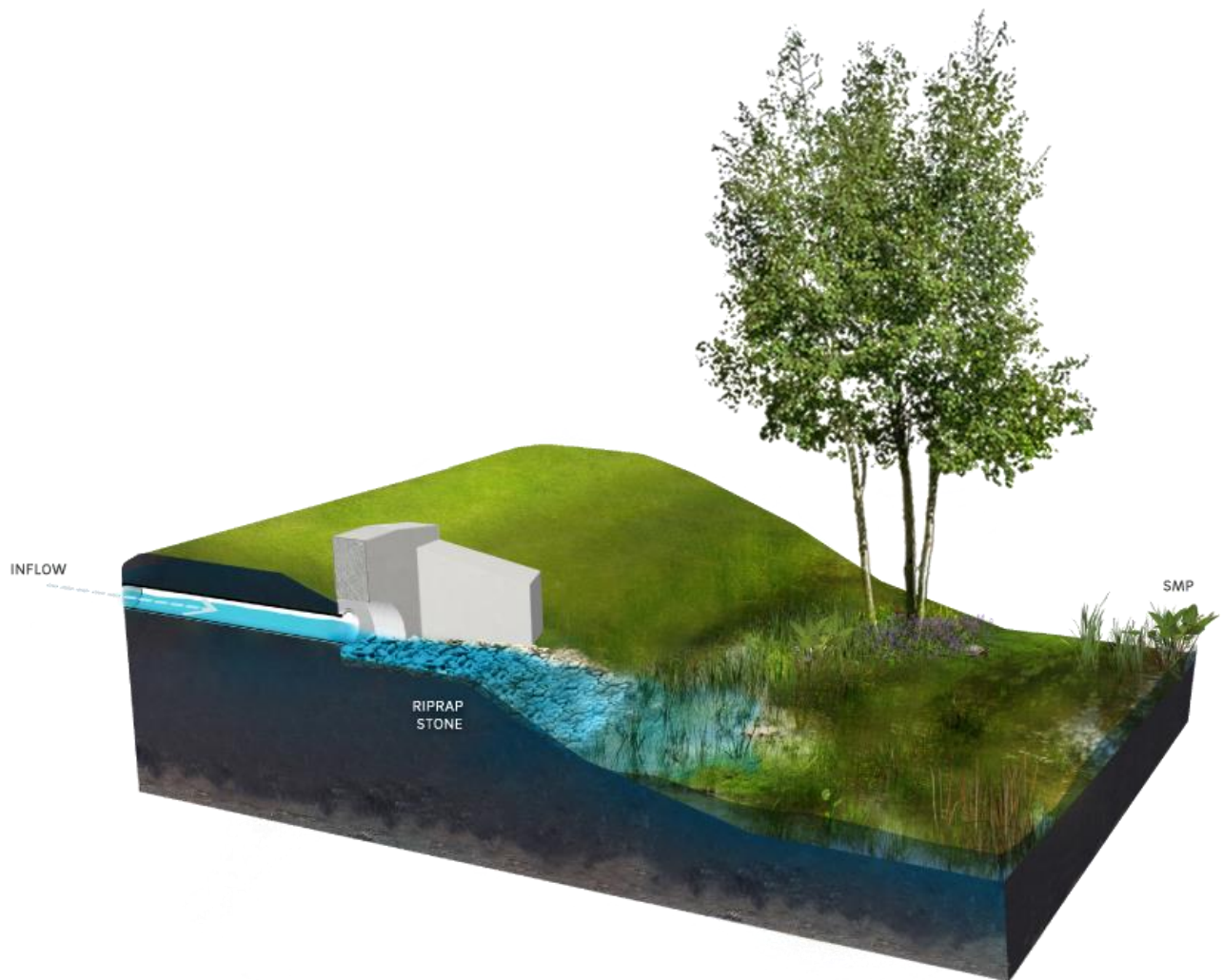


Figure F-1: Schematic of a typical inlet, here shown conveying flow into a bioretention system. The riprap stone is used to protect the inlet against scour and erosion (PWD, 2020)



Figure F-2: A typical inlet structure leading into a grassed swale
(Susdrain, 2020)

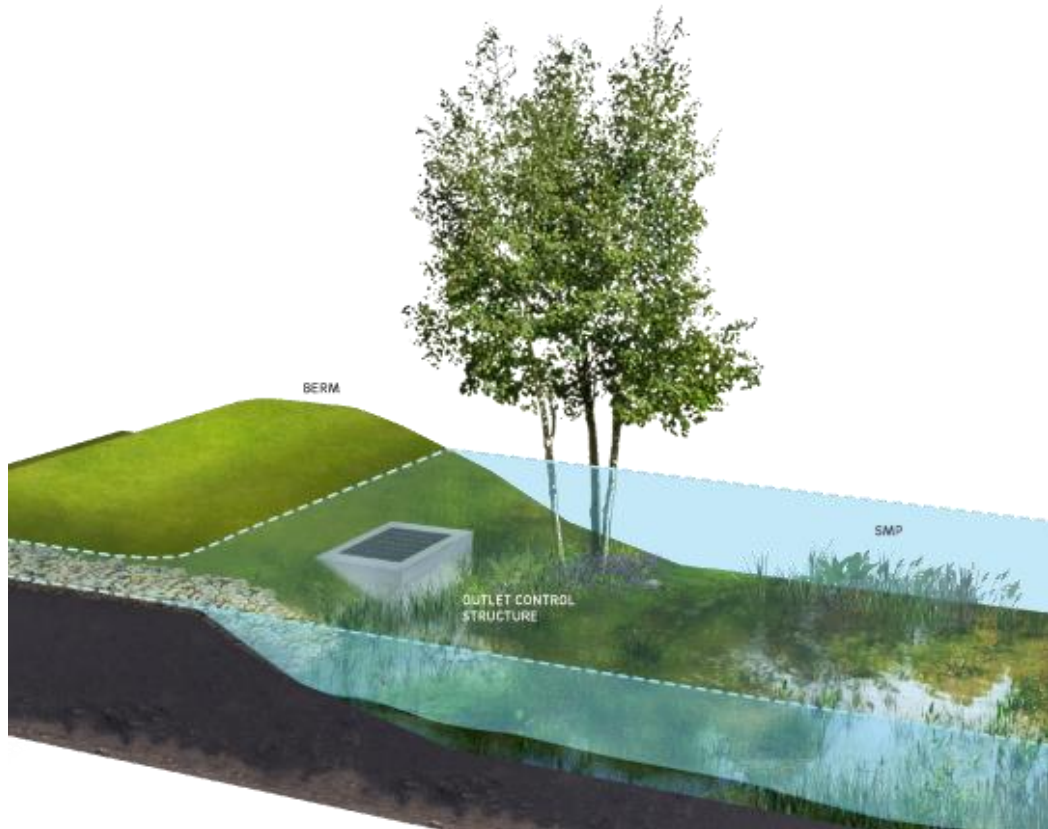


Figure F-3: Schematic of a typical outlet control structure in a depressed drainage area. When the water level reaches the top of the outlet, it drains through the grid usually into an underground pipe. (PWD, 2020)



Figure F-4: An outlet control structure in a constructed wetland (SEES, 2021)



Figure F-5: Schematic of a riser, a type of outlet control usually used in bioretention areas (PWD, 2020)



Figure F-6: Top view of a domed riser (PWD, 2020)

Appendix G: Local & regional SuDS controls: some advantages and disadvantages

G1. Local controls

G1.1 Filter strips

A filter strip is an area of grass that is used to manage shallow stormwater runoff through various filtration processes.

Besides the benefits of pollutant removal and flow peak attenuation, there are various other advantages associated with filter strips. According to Armitage *et al.* (2013), some of these are:

- Since filter strips are usually early in the treatment train, they usually trap pollutants close to the source.
- A wildlife habitat corridor can be created with the use of appropriate indigenous vegetation.
- They are unobtrusive and integrate well into natural open spaces and recreational areas.
- Installation and maintenance costs are relatively low.

There are various factors that need to be considered before deciding to construct filter strips – since adequate time is required for runoff to infiltrate, they ideally need slopes between 2-6% (Debo & Reese, 2003), they would thus be unsuitable in steep areas. The attenuation function of filter strips works best when runoff is spread out – this can be promoted by including additional infrastructure such as shallow weirs, check dams, perforated pipes, rip-rap mattresses or stilling basins. Optimal filtration processes require dense and sustainable vegetation growth, which also reduce the potential for erosion (Environment Protection Authority – Melbourne Water Corporation, 1999). Regular maintenance is required to ensure that the subsurface drainage medium remains unobstructed.

G1.2 Swales

Swales are shallow channels usually with vegetative linings and sloped sides.

Swales offer many advantages: they are less expensive and more aesthetically appealing than traditional roadside drainage infrastructure such as kerbs and concrete-lined channels. Their natural grass linings can completely absorb the runoff from adjacent impermeable areas. They also retain particulate pollutants as close to the source as possible.

A major consideration regarding the implementation of swales is their maintenance requirements. With an adequate standard of maintenance, they can manage stormwater indefinitely (Armitage *et al.*, 2013). In order to function effectively, grassed surfaces of swales

need to be regularly mowed, watered during extended dry periods, and re-seeded when uncovered. Frequent clearing of litter, debris and visible blockages are essential and weed control needs to take place (Melbourne Water, 2005).

Additional considerations include site topography (swales may be impractical in steep areas where increased flow velocity would hinder infiltration, unless check dams are used), available space (they require large surface areas to function effectively) and pollution requirements (they have limited removal capability for soluble pollutants and fine sediment). Sufficient freeboard level is required to ensure that flooding does not occur (Armitage *et al.*, 2013).

G1.3 Infiltration trenches

Infiltration trenches are shallow excavations lined with a geotextile layer and filled with large granular material, such as rock or a synthetic void-forming material.

The configuration of infiltration trenches offers various advantages – their narrow cross-section makes them suitable for implementation in urban areas, including brownfield and retrofit sites. Because they are below ground, their visual impact need not be a design consideration. Compared to other infiltration-based SuDS measures, the construction costs of infiltration trenches are relatively low.

As with other SuDS interventions, one of the key considerations is maintenance. The top layers of the trenches require regular cleaning to prevent sediment build-up. Clogging can also be avoided by incorporating a vegetated swale or buffer before infiltration trenches in the treatment train which would reduce sediment flow to the trench. While the construction costs of infiltration trenches are relatively low, their maintenance costs tend to be relatively higher, particularly in areas with fine-grained soils (Taylor, 2003).

G1.4 Bio-retention areas

A bio-retention area or cell can be described as a miniature garden in a depression designed to capture and absorb stormwater.

Bio-retention areas are versatile in their application, which offers many advantages: they can be incorporated into many other SuDS measures, such as swales and detention ponds, and can fit easily into a broad range of landscapes. They are suitable as retrofit options and can provide great amenity, biodiversity and aesthetic value.

There are a range of considerations regarding the implementation of bio-retention; they are typically suited to relatively small catchments, around 1000-4000 m², but are usually impractical in steep areas or regions where the water table is shallower than 1.8 m (Endicott & Walker, 2003). The construction costs are relatively higher than other SuDS options, and consistent maintenance

is required to preserve the bio-retention area's aesthetics and ensure that clogging does not occur. Sediment flow from adjacent areas contributes to clogging, so maintenance requirements are affected by housekeeping upstream.

G1.5 Sand filters

A typical sand filter setup consists of two linked sections: a sedimentation chamber, in which suspended particles and heavy metal are separated from stormwater, and a filtration chamber, where sand and/or other filtration media remove smaller particulate pollutants.

Sand filters provide many benefits: they can be constructed above or below ground, and filtered effluent can be repurposed and can fulfil domestic non-potable needs – such as toilet flushing and garden irrigation. Sand filters are easy retrofit options – they can typically be easily integrated into existing impervious developments and conventional stormwater drainage systems (Melbourne Water, 2005).

Sand filters have a range of limitations; they are prone to clogging and require a fairly robust maintenance programme. Surface material needs to be regularly screened to prevent litter, coarse sediment and debris from entering the system. They generally have higher construction and maintenance costs than most other SuDS interventions. In contrast to their pollutant removal ability, sand filters are typically ineffective in peak flow attenuation. Furthermore, they generally have little aesthetic appeal, unless covered with vegetation (Armitage *et al.*, 2013).

G2. Regional controls

G2.1 Detention ponds

A detention pond, also called a detention basin, is an empty surface depression/excavated area that can hold water for short durations following a storm event..

Among the advantages of detention ponds are the relatively low construction costs and simple maintenance requirements. They require periodic mowing and inspection for accumulated sediment and may occasionally need to undergo de-silting – approximately every 5 years, depending on the design (Woods Ballard *et al.*, 2015; Armitage *et al.*, 2013). They invite fewer safety concerns than wet ponds, since they are empty during dry periods. With some landscaping they can provide great aesthetic value to their sites.

The use of detention ponds is limited to appropriate sites: they require adequate space availability, are not suitable in areas with a high water table, on unstable ground or in regions with coarse soil (Taylor, 2003). The ponds need to be designed with certain considerations in mind – they are most effective with small magnitude, high frequency storms, and potentially dangerous site characteristics, such as steep slopes near the pond, should be avoided. Stormwater is often contaminated by sewage in areas with dysfunctional sewage infrastructure/power supply, in such cases the pond should be lined to prevent contamination of groundwater.

G2.2 Retention ponds

A retention pond is similar to a detention pond, except that it holds water throughout the year. It can be formed by using an existing natural depression, excavating a depression or creating embankments – usually in areas with high water tables.

The treatment capability of retention ponds is one of their greatest advantages; they can be a source of non-potable water for various irrigation and domestic non-potable purposes. These ponds are one of the most cost-effective SuDS measures and are typically easier to maintain than detention ponds. They are also an effective retrofit alternative (Woods Ballard *et al.*, 2015).

Similar to detention ponds, the implementation of retention ponds is limited to areas with enough space and are not suitable on unstable ground; they are also impractical on steeply sloped sites. Public health and safety are of particular concern with filled ponds – easy access to the pond can be unsafe, and the possibility of contaminated water entering the pond poses a health risk. Certain wildlife attracted to the pond can increase phosphorus levels in the water through their excrement. Maintenance considerations include sustaining the water level in the pond (either by adding water or through baseflow), regular removal of any litter, debris or scum appearing on the water surface, managing nuisances associated with open waterbodies like odours and mosquitoes and the need for the pond to be drained when necessary (Armitage *et al.*, 2013).

G2.3 Constructed wetlands

Wetlands are shallow waterbodies partly or completely covered in aquatic vegetation.

Wetlands offer the advantage of greater pollutant removal ability compared to detention and retention ponds of the same volume (Armitage *et al.*, 2013). They have immense amenity and biodiversity potential, and with creative use of vegetation can easily be incorporated into urban landscapes. They can also be retrofitted into existing detention ponds.

One of the most significant limitations of constructed wetlands is their space requirement. Furthermore, wetlands attract undesirable insects such as mosquitoes and birds whose excrement adds to the nutrient load in the water; biological controls such as dragonflies can be introduced to the wetland to manage the mosquito population (Armitage *et al.*, 2013).

There are various maintenance requirements to ensure optimal wetland functionality: baseflow is vital to the wetland's organisms and their habitat; in dry periods, supplementary water supply will be necessary to sustain the organisms. Litter and debris can easily enter wetlands, damaging both the wetland's aesthetic and natural processes; mitigation measures – such as a sediment forebay, trash racks on the inlet and easy access to the wetland to remove any debris – should be incorporated into the design to manage these aspects. Adequate maintenance is necessary to avoid the growth of invasive plant species which threaten indigenous wetland vegetation and in turn, the sustainability of the system (Woods Ballard *et al.*, 2015).

Appendix H: Photographs of sampling points

H1. Keyzers sampling points



Figure H-1: Site K1 on the Prinseskasteel River, just upstream of its confluence with the Keyzers River



**Figure H-2: Site K2, as the Keyzers River flows under Tokai Road
(downstream is left)**



**Figure H-3: Site K3, the beginning of the stream behind Blue Route Mall
(facing downstream)**



**Figure H-4: Site K4, just after the Military Road bridge
(facing downstream)**



Figure H-5: Taking a sample at Site K5, just upstream of the confluence with the Westlake River. The reed growth alongside and the aquatic growth on the river become increasingly more dense the closer one gets to the confluence of the Westlake and Keyzers Rivers.

H2. Westlake sampling points



Figure H-6: Site W1, just downstream of the US consulate. The river is shallow at this point and flows in the dense reeds beneath Reddam Avenue.



Figure H-7: Site W2, just downstream of the Westlake Primary School and upstream of the Pollsmoor Prison grounds (facing downstream)



Figure H-8: Site W3 in the Kirstenhof Greenbelt (facing upstream)



Figure H-9: Site W4, the channel in Orchard Park draining the Westlake Golf Club (facing upstream)



Figure H-10: Site W5, just after the bridge over the Westlake River leading to Camargue housing complex (facing downstream and Main Road). This picture was taken 2 days after the river was dosed with pesticide (according to CCT employees) to clear the water hyacinth overgrowth.



Figure H-11: Site W6, just after the Main Road bridge over the river



Figure H-12: Just upstream of Site W7, facing downstream



Figure H-13: Water hyacinth growth and scum floating on the water surface near Site W7