UNIVERSITY OF THE WITWATERSRAND, JOHANNESBURG

CONVENTIONAL DESIGN AND WATER SENSITIVE URBAN DESIGN (WSUD) WITHIN A SOUTH AFRICAN GREENFIELDS DEVELOPMENT

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DECLARATION

I declare that this research project is my own unaided work. It is being submitted for the degree of Master of Science in Civil Engineering to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

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(Signature of Candidate)

...21st...day of ...September... year...2020...

ABSTRACT

Rapidly urbanizing areas present unique opportunities to implement alternative approaches to conventional water management as the Republic of South Africa (RSA) is a water-stressed country. An integrated systems approach that takes into account the different water users, sanitation and drainage have the potential to make urban areas more 'water-sensitive'. Some of the challenges facing RSA include inadequate supply, lack of accountability by those put in charge, failing of existing infrastructure, unsynchronized planning and environmental challenges brought about by climate change (e.g. drought). The main objective of 'water-sensitive' design is to decelerate runoff as well as reduce the quantity of surface water runoff. This is to allow for the effective management of downstream flooding, pollution risk, and potable water demand reduction. These objectives are achievable by harvesting, infiltrating, decelerating, storing, conveying, and treating runoff on site and, where possible, on the surface rather than underground.

In this research, the conventional design of storm water and potable water systems is compared to Water Sensitive Urban Design (WSUD) systems within a greenfield township development in South Africa. The key goal of this study is to determine if the benefits of applying WSUD methods to a greenfield urban development in South Africa is likely to outweigh the benefits of conventional design methods.

Prior to the detail design, an overview to the many conventional and WSUD interventions are provided. The study then focuses on the design of the water systems using the two approaches. Following the design of both systems, the cost of implementation is determined by measuring the quantities of the different elements of the solutions and applying the applicable market related costs.

The runoff generated in the post-development WSUD scenario is 30% less than the value of the post-development no control scenario. The cost benefit ratio is used to determine if the WSUD is economical as compared to the conventional design. A benefit cost ratio of 1.07 is calculated. As such the WSUD is considered better than the conventional design alternative. The study concludes that the implementation of WSUD within new developments is a starting point in achieving water sensitivity in greenfield developments in RSA.

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1. INTRODUCTION

1.1 Background

The Republic of South Africa (RSA) is a water-stressed country that is currently dealing with several challenges, including the security of constant water supply, environmental degradation, water resource pollution, access to secure and clean water supply, flood protection and mitigation of urban heat. Some of the challenges facing RSA include inadequate supply, lack of accountability by those put in charge, failing of existing infrastructure, unsynchronized planning and environmental challenges brought about by climate change (e.g. drought). This means that its scarce water resources require cautious management to ensure that every citizen has access to basic water while ensuring that the needs of economic growth are met in an environmentally sustainable manner (Department of Water Affairs, 2013).

Rapidly urbanizing areas present unique opportunities to implement alternative approaches to conventional water management due to constantly increasing water demands and inefficient water use. An integrated systems approach that takes into account the different water users, sanitation and drainage have the potential to bring about change in our current urban areas, taking them from 'water-wasteful' to 'water-sensitive' (Armitage et al., 2014). This notion is termed Water Sensitive Urban Design (WSUD). WSUD is loosely described as ensuring 'urban design' is done in a 'water sensitive' manner with the final goal being the holistic management of the urban water cycle to concurrently achieve the desired economic, environmental, and social targets and benefits (Armitage et al., 2014).

Conventional water systems have been developed using a linear design approach, where water is taken from a source, treated, transported, distributed to users, collected, treated and disposed of back into the environment (Armitage et al., 2014). This resource-intensive approach removes ownership and responsibility from the users it serves, resulting in the inadequate management of the urban water cycle. This linear design approach also produces high stormwater runoff resulting in high infrastructure cost. When urban surfaces are impermeable, surface runoff increases and so does pollution downstream. Controlling, conveying and disposing of runoff downstream is what informs the design of Conventional water systems. They are however limited because they limit groundwater infiltration and recharge and consequently, negatively impact other aspects of the water cycle e.g. reduced local evaporation. Water quality also comes into question as runoff discharged into receiving water bodies typically have high pollutant levels. This problem can be mitigated if local governments implement and enforce WSUD in urban developments (Armitage et al., 2014).

The WSUD approach in the South African context has the potential to additionally transform urban areas by connecting spatial divisions through the development of 'blue-green corridors', and ensuring greater equity in the availability of water services to a wider variety of urban areas. A study by Carden et al. (2018) outlining the challenges and opportunities for implementing WSUD in South Africa shows that WSUD has the potential to change how water is managed in RSA, increase sustainability and develop resilience through the implementation of reasonably modest WSUD interventions. The study, however, acknowledges that this new paradigm shift will take time, require local-level knowledge and 'champions' and the political buy-in to take this phenomenon forward.

In this research, conventional drainage systems (that typically dispose of runoff via concrete-lined channels and conduits to the nearest watercourse to prevent flooding to properties) will be compared to water sensitive solutions i.e. sustainable drainage solutions. Conventional drainage systems contribute to the degradation of receiving water bodies due to the increase in flows and pollutant loads compared to their natural conditions. Typically, pollutants in urban runoff such as sediments, hydrocarbons, metals, and nutrients degrade receiving water bodies. These pollutants normally originate from sources such as vehicle emission, sediment, brake linings, atmospheric deposition, pesticides and nutrients from lawns and gardens, viruses, bacteria and nutrients from pet waste and failing sewer systems (Armitage et al., 2013a). The increased flow peaks also cause erosion of the receiving water bodies. WSUDs offer a more holistic, environmentally safe alternative to conventional drainage systems. It manages stormwater runoff volumes and flow rates, reduces the pollutants of the runoff all the while offering various amenity functions and supporting biodiversity (Fisher-Jeffes et al., 2016).

1.2 Research Question /Problem Statement

This investigation raises the question: *Are the benefits of applying WSUD methods to a greenfield urban development in South Africa likely to outweigh the benefits of conventional design methods?*

1.3 Research Objectives

The purpose of this study is to compare the conventional design of stormwater and potable water systems to WSUD within a greenfield township development in South Africa. The results of both designs will be analysed against costs and benefits. The WSUD framework and guidelines for RSA as well as international best practices will be applied to a greenfield township development in Johannesburg. The specific objectives of this study are listed below:

- To identify applicable conventional and WSUD solutions for a greenfield township development in Johannesburg.
- To determine Pre- and Post- development run-offs using both methods.
- To calculate and compare the cost of conventional and WSUD stormwater and potable water infrastructure.
- To assess potential savings and/or losses (if applicable) from the implementation of recommended WSUD solutions in relation to conventional infrastructure solutions.

1.4 Case Study Application

This research paper presents a site specific comparison between the conventional design of stormwater and potable water systems to WSUD best practices. The methods used in the design of the interventions in this research require weather and geographic data which are site specific inputs.

The case study site is a high density, low-income, greenfields township development in South Africa. The selection of this area was motivated by the availability of design input information as well as the fact that there exists reliable bulk municipal services (water, sewer, and stormwater) in the vicinity making it easy for the design of conventional systems.

1.5 Expected outcomes

It is anticipated in this study that WSUD solutions will reduce peak runoff flows and as a consequence, improve the quality of runoff entering receiving water bodies and reduce the cost of required infrastructure.

1.6 Limitations of the research

The conclusions drawn from this research report will be limited to the urban context. Detailed construction drawings will not be produced.

1.7 Brief overview of the research report

Chapter 1 presents the rationale and objectives of the study. Chapter 2 presents a literature review explaining the benefits of WSUD, specifically highlighting the benefits of urban water management and a brief overview of local and international policies. Chapter 3 presents an explanation of the assessment method to be used and Chapter 4 presents the case study area. Chapter 5 outlines the data acquisition and analysis performed in the case study area. Chapter 6 presents the conventional and water sensitive designs. Chapter 7 outlines the key findings from the case study, including populated comparisons. Chapter 8 closes out the report with a summary, recommendations, and conclusions.

2. LITERATURE REVIEW

2.1 Introduction

Rapidly-urbanizing areas present a unique opportunity for the application of WSUD. Due to their fast economic growth, these areas are inclined to become major drivers for increased water demand if proper management of resources is not put in place. Due to this, it is apparent that an alternative to the 'business-as-usual' systems-based approaches to the conventional design of water supply systems are required (Armitage et al., 2014).

Green infrastructure (GI) is described as nature-based solutions that provide ecosystem services like water management, air quality management and climate regulation in urban environments (CoJ, 2019). Green roofs, infiltration trenches, vegetated swales, wetlands, bio-retention systems, sediment basins, and porous pavements are examples of green infrastructure. Conventional engineering, also referred to as grey infrastructure, is based on design principles that ensure that surface water is safely and quickly removed from a source to discharge or outfall. Grey infrastructure includes pipes, pits, pumps, ditches, and detention ponds engineered for the reliable management of stormwater. Though grey infrastructure has been successful in many instances, it is however described by many industry leaders as not resilient enough to handle the ever growing uncertainties surrounding climate change and ever-growing urbanization. Green infrastructure provides an alternative that is multi-functional and is considered more sustainable when compared to grey infrastructure. When adequately designed, it provides a similar level of service to some of the current grey infrastructure solutions (CoJ, 2019).

The World Bank reports that urban, peri-urban and rural environments can equally benefit from WSUD. Table 2.1 below indicates the general approach to how Green and Grey Infrastructure can work together to provide sustainable solutions (Browder et al., 2019).

Table 2.1 - Green infrastructure solutions to grey infrastructure components (Browder et al., 2019).

Service	Grey infrastructure components	Green infrastructure solutions/examples
Water supply and sanitation	Reservoirs, treatment plants, pipe network	Watersheds: Improves source water quality and thus reduces treatment effort Wetlands: Filter wastewater effluent and thus reduces wastewater treatment effort
Urban flood management	Stormwater drains, pumps, outfalls	Urban flood retention areas: Store stormwater and thus reduce drain and pumping effort
River flood management	Embankments, sluice gates, pump stations	River floodplains: Store flood waters and thus reduce embankment requirements

The perfect combination of green and grey infrastructure provides cost effective solutions, improves resiliency which contributes toward decreasing the financial and environmental crisis facing infrastructure systems globally. For example, New York City saved approximately 22%, or $1.5 billion, by combining green and grey infrastructure solutions to secure a reliable water supply to its residents (Browder et al., 2019).

2.2 Urban water management systems

2.2.1 A brief overview

According to Browder et al. (2019), almost half the world's population lives in a water-scarce area. By 2050, nearly 20 percent of the world's population will be living in flood risk areas. Cities are at the same time struggling to provide dependable, safe, efficient, and economical infrastructure to provide residents with clean water and resilience against drought. Conventional infrastructure systems provide solutions for the smooth and safe functioning of societies. The approach to these systems however no longer provides the climate resiliency and level of services required in the face of growing environmental threats.

Solutions that incorporate natural systems such as local vegetation, forests, floodplains, and soil conditions can potentially contribute to systems that provide clean, consistent water supply and protect against environmental threats such as floods and drought. Service providers such as municipalities, through the use of green infrastructure, can provide more cost-effective and resilient services to its residents (Browder et al., 2019). Green infrastructure aims to reduce the impact of urban development on water resources and has been adopted in many countries around the globe. Green infrastructure was adopted in the UK as SUDS (Sustainable Urban Drainage Systems), in the USA as LID (Low Impact Development), and in Australia as WSUD (Water Sensitive Urban Design). Developing countries such as South Africa, Brazil, and China, which are currently experiencing similar rapid growth of their metropolitan areas have adopted these principles (CoJ, 2019).

To gain the maximum benefits from WSUD, surface water management should be considered from the beginning of the development planning process (CoJ, 2019). As such, this will influence the site layout plan, design, and the characteristics of urban spaces i.e. the town planning process will be greatly influenced. To achieve this an interdisciplinary team (including town planners, architects, landscape architects, drainage, water, and environmental engineers) need to be appointed by the client and work together from the beginning. In so doing, site specific decisions are made with a broader understanding of the constraints and requirements of the wider catchment. This interdisciplinary design process essentially assures the optimum design of the development.

2.2.2 Terminology surrounding urban drainage

According to Fletcher et al. (2015), the development and use of terminology in urban drainage/stormwater management have occurred in an informal manner, driven by local context and regional perspectives. As such, there is a wide range of different terms and terminology used to define similar concepts in different parts of the world. This section aims to provide clear definitions for all the relevant terms potentially reducing the number of contradictions. In this research paper, the terms ‘urban drainage’, ‘urban stormwater’ and ‘urban runoff’ will be used synonymously.

The term **Low Impact Development** (LID) is most commonly used in North America and New Zealand. The term was first used in 1977 and aims to minimize the cost of stormwater management, by taking a "*design with nature approach*". LIDs are defined as "*an approach to land development (or re-development) that works with nature to manage stormwater as close to its source as possible. LIDs employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product*" (USEPA, 2012a). LIDs generally discouraged the use of large end-of-catchment solutions and aims to achieve 'natural' hydrology (equilibrium between pre-development runoff, infiltration, and evapotranspiration quantities) by use of the site layout and integrated runoff control measures (Fletcher et al., 2015).

The term **Water Sensitive Urban Design** (WSUD) has been most commonly used in Australia since 1992 and is described as an approach to urban planning and design that aims to minimize the hydrological impacts of urban development on the surrounding environment through the following (Fletcher et al., 2015):

- *"manage the entire water balance"*
- *"maintain and enhance water quality"*
- *"encourage water conservation and demand management"*
- *"maintain water-related environmental and recreational opportunities"*

Despite its origin in Australia, the term WSUD is now used interchangeably in the UK, New Zealand and South Africa (Fletcher et al., 2015). According to Carden et al. (2018) the integration of the water cycle into planning and design cuts across all geographic locations including rural settlements. As such, it was recommended that the reference to the word 'urban' be removed from the term WSUD to Water Sensitive Design (WSD). It is envisioned that by so doing, it will ensure that local authorities will implement green infrastructure to meet developmental goals in all types of settlements.

The term **Integrated Urban Water Management** (IUWM) has been used since the early 1990s and is derived from the larger term, Integrated Water Management (IWM). It relates to the integrated management of all aspects of the water cycle within a catchment and it encourages the management of water supply, wastewater, groundwater and stormwater in a holistic manner (Fletcher et al., 2015).

The term **Sustainable Urban Drainage Systems** (SUDS) or **Sustainable Drainage Systems** (SuDS) has been most commonly used in the UK since the late 1980s and in 1992 when the first set of guidelines was officially published. The term SuDS was officially coined in 1997 by Jim Conlin of Scottish Water. SuDS consist of a variety of interventions used to drain stormwater runoff in a manner that is more sustainable when compared to conventional solutions. This philosophy is aimed at replicating the natural, pre-development conditions of the site in the form of a management train (Fletcher et al., 2015).

The term **Best Management Practices** (BMPs) has been most commonly used in the United States and Canada since 1992 and is described as a type of practice or structured approach to prevent pollution. In the stormwater context, the performance of urban stormwater BMPs is grouped into four management categories i.e. detention, recharge, housekeeping practices and others (Fletcher et al., 2015). The term **Green Infrastructure** (GI) is also used in the USA and was first used in the mid-1990s. It is a notion that goes far beyond stormwater management. The GI concept is a concept that requires urban planners to maximize the inclusion of green space hubs and corridors in their designs. The term is frequently used interchangeably with BMPs and LIDs. In the stormwater management context, GI is defined as "*a network of decentralized stormwater management practices, such as green roofs, rain gardens and permeable pavement that can detain and infiltrate runoff which leads to the improvement in quality before discharge into surrounding waterways*" (Fletcher et al., 2015).

There are significant similarities and synergy between all the various terms described above. All the terms and principals are commonly underpinned by the following principles:

- They aim to maintain pre development/ natural hydrology and
- They aim to improve the quality of water systems by reducing pollutants.

2.2.3 Global trends

Globally, Europe, USA, Australia, and Singapore are the leading promoters of WSUD as the required legislation is extensively developed and implemented. Although they all have different regulatory structures and policy frameworks, they all fundamentally support sustainable development in water management (Hoyer et al., 2011). Through the use of WSUD, many urban areas have achieved water sensitivity (Armitage et al., 2014).

2.2.4 The South African context

Post-development runoff from Johannesburg's paved areas in 2011 was approximately 45% of the annual potable water purchased. The City of Johannesburg naturally doesn't have enough potable water for its consumers. Therefore, it imports the precious resource from as far afield as KwaZulu-Natal and Lesotho through inter-basin transfer schemes and international agreements. Alongside the scarcity of water, it has been reported that the Cities of Ekurhuleni and Johannesburg pollute the local streams to such an extent that the Department of Water & Sanitation and Rand Water are required to flush the Vaal River with some of this imported water. This is an effort to protect its downstream users such as the users of the Crocodile River and Hartebeespoort Dam (CoJ, 2019). This makes WSUD interventions more important as they play a significant role in the protection, management, and preservation of precious water resources.

RSA is currently in a peculiar position, it faces the challenge of delivering infrastructure to promote economic and social equity whilst needing to ensure the environmental sustainability of resources in its water stressed state (Armitage et al., 2014). From the water management viewpoint, the sustainable delivery of infrastructure is possible only through the effective integration of urban design, planning, and management in a water sensitive manner. In response to this, the Water Research Commission (WRC) in April 2014, released the WSUD framework and guidelines for RSA. The framework provides strategic guidance to urban water management, specifically WSUD for decision - makers in South Africa. According to Armitage et al. (2014), the framework, if adopted, has the potential to meet the challenges facing the urban water management sector in RSA. The framework aims to enable the transition to Water Sensitive Settlements by providing guidance on the various WSUD strategies that can be adopted to achieve water sustainability in RSA.

2.2.5 Legal and institutional framework supporting green infrastructure

The RSA government recognizes water as a critical role player in equitable social and economic development as such the constitution demands that every citizen has access to water. It is the basis of this constitutional right that led to the promulgation of South Africa's current water legislation.

The National Water Act (No 36 of 1998) legislates and provides guidance for how water resources in South Africa are to be managed. The purpose of the Act is to ensure that the nation's water resources are protected, used, developed, conserved, managed and controlled in a manner that fulfils the constitution (RSA, 1998). The Act recognizes that water is a scarce and unevenly distributed resource and that the national government's overall responsibility for the nation's water resources is to ensure its equitable allocation to all. The Act further states that the establishment of suitable institutions will lead to the successful implementation of the above.

The National Water Act (No 36 of 1998) requires the development of a National Water Resources Strategy (NWRS) by the minister. The NWRS is the main tool used to manage water across all sectors in South Africa to achieve the national government's development objectives. As per the Act, the NWRS is binding on all authorities and institutions implementing the duties stated in the Act (RSA, 1998). The first edition of the NWRS was published in 2004. The Act requires that the NWRS be reviewed at intervals not exceeding five years (RSA, 1998).

The first edition of the National Water Recourses Strategy (NWRS-1) of 2004 established the basics of integrated water resource management and clearly laid out the water situation in South Africa. The strategy also sets some critical actions to be achieved (Department of Water Affairs, 2013). The second edition of the National Water Recourses Strategy (NWRS-2) of 2013 builds on the progress that was made post NWRS-1. It also highlights the achievements and challenges of NWRS-1. One of the notable challenges pointed out was the inability to achieve targets that were set.

The NWRS-2 and the NDP propose the adoption of "*developmental water management*" to ensure that the scarce resource is available for all citizens (Department of Water Affairs, 2013 cited by Carden et al., 2018).

The National Climate Change Response White Paper also emphasizes the need for RSA to develop and implement WSUD principles in the urban landscape and to minimize pollution and erosion of receiving water bodies (RSA, 2011).

Despite the above, water resources to date are still not receiving the urgency it deserves. The RSA government is yet to provide and adapt an effective comprehensive approach to managing the total water cycle including, the impact of urban stormwater runoff and the use of SuDS in RSA.

2.2.6 Challenges and opportunities

Industry experts recently reported that although urban and peri-urban environments could benefit greatly from the water sensitive approach, some challenges are hindering its full implementation. The study by Carden et al. (2018) and Fisher-Jeffes et al. (2017) provides a comprehensive list of challenges inhibiting the implementation of WSD in RSA. The challenges by Carden et al. 2018 were concluded following a comprehensive case study of 21 developments across five metropolitan areas in RSA. The challenges are listed as follows:

- *Approval / legislation mechanisms:* there is currently a disconnect and lack of synergy between the approval process and the enabling legislation that promotes the inclusion of WSD in developments across certain municipalities. There has been a gradual increase in the presence of local legislation/guidelines available to guide and enforce the implementation of WSD, specifically stormwater policy, in the metropolitan municipalities. This is however very limited and mostly unavailable in others especially in District and Local municipalities. Additionally, these legislations/guidelines when available are almost entirely concerned with stormwater quantity management (Carden et al., 2018).
- *Institutional champions:* the lack of Institutional champions e.g. project leaders and consultants, that go beyond the basic approval / legislation requirements and

take it upon themselves to ensure that development level solutions are in line with regional/catchment level interventions (Carden et al., 2018).

- *Fragmented water management institutions:* The disjointed ‘silo-management’ of the different aspects of the urban water cycle in RSA currently allocates different responsibilities to different municipal divisions/departments. For example, the approval of stormwater management plans is often done by the roads departments, while water supply designs are often approved by the water and sanitation departments (Fisher-Jeffes et al., 2017).
- *Economic incentives*: the lack of economic incentives in the sense that the perceived benefit of WSD and its return on investments (RoI) is not entirely understood (Fisher-Jeffes et al., 2017).
- The lack of clear and consistent *objectives and targets for WSD* with specific regard to new urban developments and infrastructure. This constraint contributes to the lack of synergy between urban developments and regional/catchment level urban water management systems in municipalities (Fisher-Jeffes et al., 2017).
- The lack of a *consistent approach to WSD* across all relevant government policy departments and approving departments (Fisher-Jeffes et al., 2017).
- The lack of local *government and private sector support* to achieve the required standard to undertake WSD across RSA. This issue can be resolved through building capacity and skills through ongoing capacity building initiatives and peer mentoring and assessment (Fisher-Jeffes et al., 2017).
- Other socio-institutional challenges identified include underfunded government organizations; resistance to innovative approaches; and technical capacity problems across all stakeholders (Fisher-Jeffes et al., 2017).

The successful implementation of WSUD will require a combination of tools and strategies in order to realize the full potential of the opportunities provided by WSUD and gradually mitigate the challenges listed above (Fisher-Jeffes et al., 2012).

Carden et al. (2018) recommend the following as a way forward for the successful implementation of WSD across RSA:

- WSD should be incorporated into the entire regulatory structure of municipal and local authorities. These include its incorporation into Integrated Development Plans (IDPs), Spatial Development Plans (SDPs), Water Services Development Plans (WSDPs), bylaws and policies.

- An integrated 'Water Sensitive' strategy and targets are developed for all towns and cities, and must be linked to the WSD framework and guidelines by Armitage et al. (2014).
- The Guidelines for Human Settlement Planning and Design, the 'CSIR Red Book' widely unitized in the design of urban water systems, be amended to include the concepts of WSD / SuDS.
- The departments dealing with water and spatial planning, at both site and regional level in local areas, engage as early as possible in greenfield developments to ensure that WSD is adequately planned for. This will minimize the future need for retrofitting.
- Upon the completion of WSD / SuDS intervention, they were captured in the local authority's asset register and provision be made for monitoring. This will allow for continuous monitoring and enforcement of targets.
- A comprehensive design guideline is developed for WSD / SuDS implementation, operation, and maintenance. This guideline should be applicable across RSA.

2.3 Conventional urban water systems

In the conventional design of water systems, water is taken from a source, treated, transported, distributed to users. Wastewater is then collected, treated and disposed into the environment. In urban areas, the design of conventional WDS is therefore focused on meeting the demand for water, which is typically increasing.

During rainfall events in many urban catchments, runoff (containing pollutants such as sediments, nutrients and litter) is speedily evacuated, resulting in erosion, pollution and disturbance of in-stream ecology. (Water by Design, 2010).

When urban surfaces are completely impermeable, the natural water cycle in that area is adversely interrupted. The impermeable surface causes an increase in surface runoff. This then causes a notable reduction in groundwater recharge and evaporation. This increase in runoff is what informs the conventional design of stormwater drainage systems to ensure flooding does not occur (Hoyer et al., 2011). Conventional stormwater management systems (CSMS) have been reliable in the past as they quickly collect runoff generated and drain it from the city. Urban areas are heavily reliant on these systems; they, however, have many shortcomings that have led to the paradigm shift in design. A few of these shortcomings are listed below (Hoyer et al., 2011) :

- CSMS limit groundwater infiltration and reduce groundwater recharge rates. A decrease in groundwater recharge adversely affects cities that are heavily reliant on groundwater as a direct source of potable water.
- CSMS can negatively affect the local climate due to the reduced infiltration and evaporation rates. A cities' climate generally becomes warmer and dryer as compared to its surrounding areas. This phenomenon is commonly known as the Heat Island Effect.
- Heavy periods of rain cause an overflow to receiving water bodies, which in turn, increases the risk of downstream flooding.
- CSMS is rigid and cannot adapt to change when there is an increase in the city's development upstream of the network. In some instances, it is required that infrastructure be redesigned and constructed, making them capital intensive. On the other hand, forward thinking municipalities tend to overdesign systems making the initial cost of the infrastructure very high.

The conventional design of potable water systems focuses primarily on the construction of large scale water supply systems to meet the demand for water. Large scale water supply schemes entail the collection of water from the source (e.g. rivers and dams), treatment, transportation, and distribution to end users. Many South African urban communities are heavily reliant on conventional water distribution systems (CWDS) for their water supply. There is however no guarantee that the valuable resource will always be available especially now that the resource is scarce and many water boards do not have the budget required to treat and distribute current and future urban water demands. A few of the limitations associated with the CWDS are listed below:

- CWDS require several infrastructure elements (e.g. treatment plants, reservoirs, pipe networks) and these are capital intensive. Alternative water sources can be used to supplement potable water demand hence reducing the size and cost of required infrastructure.
- Aging distribution system infrastructure contributes to increasing numbers of leaks and pipe replacements hence negatively affecting the reliability and quality of supply to end users.

South Africa's scarce water resources require sustainable management to meet its growing water supply needs. For the country to reach sustainability, it is imperative that there be a move from the conventional supply model to water sensitive strategies. Water sensitive supply strategies entail the reduction of potable water supply requirements, minimizing wastewater generation and maximizing the use of alternative water sources (Armitage et al., 2014). For example, WSUD allows for stormwater drains to be viewed as a vehicle to carry precious water resources from source to control points for treatment and reclaim into the environment. Sustainable drainage systems not only allow for safe reuse by the environment but can also improve the quality of life in urban spaces by making them more visually attractive and vibrant. Its sustainability means it is more resilient to our changing environment. It improves urban air quality and regulates building temperatures. The stormwater design process allows for catchment planning, catchment recovery, and flood/hazard risk management in the design process (CoJ, 2019).

For this investigation, the following conventional designs will be undertaken:

- Water: treated water to a greenfields township development will be from the nearby municipal reservoir and transported by a water distribution network to end-users.
- Roads and Stormwater: runoff generated within and nearby the greenfields township development will discharge from the roads to full bore underground pipes and concrete channels for discharge into a nearby municipal stormwater system. Road pavement layers such as asphalt and block pavers will be incorporated into the design.

2.4 Water Sensitive Urban Design

2.4.1 Introduction

Brown et al. (2008) describe WSUD as "*an approach to urban planning and design that integrates land and water planning and management into urban design. WSUD is based on the premise that urban development and redevelopment must address the sustainability of water*" (Brown et al., 2008 cited in Armitage et al., 2014)). This, in essence, means that the method of WSUD is an urban planning and design method that ensures that urban development is sensitive to the environment. These methods consider conserving water supplies by minimizing treated water demand, minimizing wastewater, and managing stormwater quantity and quality (Water by Design, 2010).

Typical examples of WSUD solutions include erosion and sediment control, rainwater tanks, swales, porous pavements, bio-retention systems (or raingardens), constructed wetlands, infiltration systems and stormwater harvesting and reuse schemes (Water by Design, 2010). The main design objective of WSUD is to decelerate runoff as well as reduce the quantity of surface water runoff from a development. This is to allow for the effective management of downstream flood, pollution risk, and potable water demand reduction. These objectives are achievable by harvesting, infiltrating, decelerating, storing, conveying, and treating runoff on site and, where possible, on the surface rather than underground.

The adoption of any of the WSUD solutions listed above requires a proactive and holistic approach as it has the potential to reduce the negative effects of water scarcity; reduce water pollution; enhance social equity; increase ecological sustainability; and develop resilience within connected water systems in RSA (Armitage et al., 2014).

There are two main components of WSUD namely, Urban Water Infrastructure and Design and Planning. WSUD activities that make up the two main components are listed below (Armitage et al., 2014):

Urban water infrastructure activities include the following:

- **Stormwater management**: the application of sustainable drainage systems (SuDS) entails the incorporation of factors such as biodiversity and amenity enhancement and flood mitigation in the design process.
- **Wastewater / Sanitation minimization**: entails the consideration of factors such as effluent quality improvement and the reuse of treated wastewater in the design process.
- **Groundwater management**: entails the application of solutions such as artificial recharge and the use of groundwater.
- **Sustainable water supply**: entails the inclusion of water conservation, water demand management, non-revenue water reduction and alternative water sources, e.g. rainwater and stormwater harvesting in the design process.

Design and planning activities include the following:

- Inclusion of local community character and environment throughout the design process.
- Improving liveability and local amenity.
- Optimizing cost-benefit of infrastructure.
- Improving resource resilience and security.

Figure 2.1 provides an illustration of the different types of infrastructure-related activities described above.

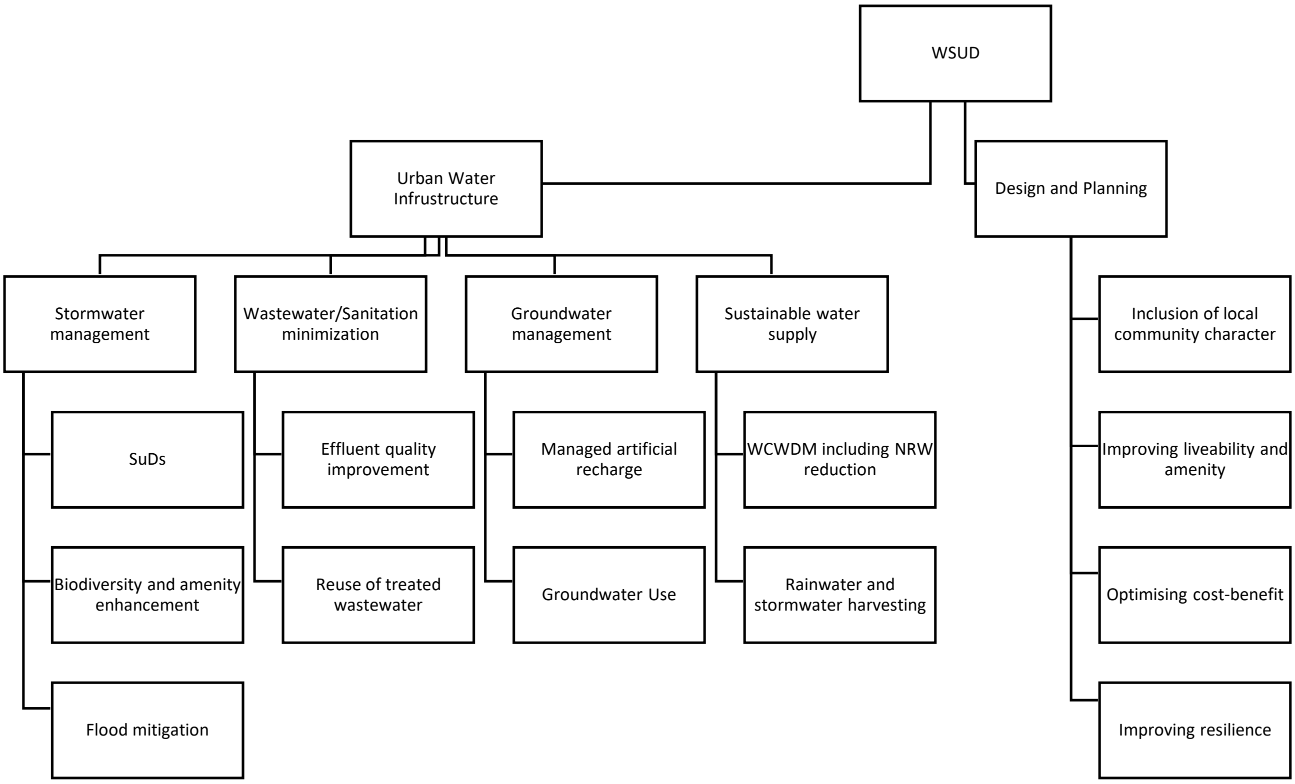


Figure 2.1 – An illustration of the different types of WSUD activities (Armitage et al., 2014).

2.4.2 Stormwater management

According to Dong et al. (2017), there is a growing need for infrastructure that can handle the increased runoff in terms of quality and quantity but also reduce failures caused by adverse weather events. Figure 2.2 below provides an example of green vs grey infrastructure.

Figure 2.2 - Green vs Grey Infrastructure (Lagasse, 2015)

In addition to the benefits of WSUD relating to stormwater (i.e. volume reduction, aquifer recharge, energy dissipation, temperature reduction, peak delay and peak reduction), WSUD also provides added benefits such as improved biodiversity, carbon sequestration, faunal habitat restoration, improved urban aesthetics, and air pollution reduction. WSUD improves urban water management through integrated water cycle management. Conventional engineering aims to efficiently and quickly dispose of stormwater. This approach has been proven to have negative environmental consequences in that local water resources are unexploited and receiving waters are usually polluted causing a decline in the health of aquatic ecosystems in both local and receiving catchments (CoJ, 2019).

<u>Sustainable Drainage Systems (SuDS)</u>

Sustainable Drainage Systems (SuDS) is a component of WSUD that aims to manage surface water drainage systems holistically by simulating the natural hydrological cycle, often through a number of consecutive interventions referred to as a ‘treatment train’ (Armitage et al., 2014). The main goal of SuDS is to achieve the highest level possible on the ‘Stormwater Design Hierarchy’ depicted in figure 2.3. The hierarchy is a four-step pyramid which illustrates the following; any stormwater design has to meet the requirements of the base of the pyramid (managing water quantity by flow control and storage) before climbing to the second step (improving water quality), the third step (amenity) and finally the fourth step (biodiversity) (Fisher-Jeffes et al., 2016). Each level represents an improved and sustainable drainage system design (Armitage et al., 2014).

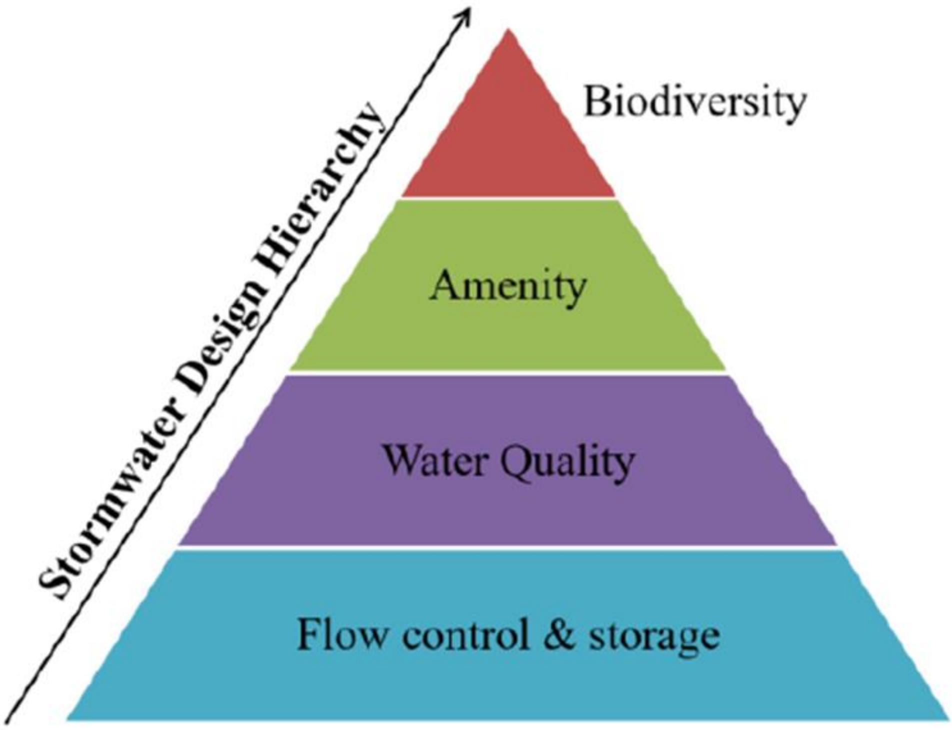
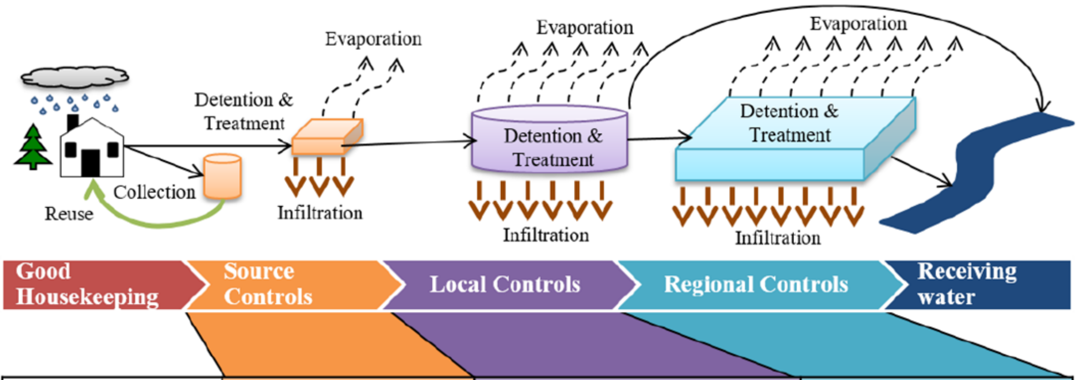


Figure 2.3 – The SW design hierarchy (Armitage et al., 2014)

The RSA SuDS Guidelines (Armitage et al., 2013b) categorizes the SuDS interventions into twelve groups. These interventions integrate a mixture of treatment processes with substantial overlaps in processes. The placement of interventions in succession or as a “treatment train” ensures the resilience of a SuDS drainage system. A treatment train comprises of several SuDS options which are implemented at a range of different scales from the source, local, to the regional scale (Armitage et al., 2013b).

It is highly unlikely that all interventions will be applicable to one site as the selection of interventions are determined by the unique characteristics of the site. There are four fundamental coaches in the SuDS treatment train (Figure 2.4) which have different combinations of SuDS interventions available to control runoff. SuDS interventions significantly improve water quality through the application of a range of processes listed in Figure 2.4. Appropriate use of certain SuDS removes particular pollutants. A combination of interventions can successfully control the full range of runoff impacts (Fisher-Jeffes et al., 2016).



		Source Controls				Local Controls					Regional Controls			
		Green roofs	Rainwater harvesting	Soak-a-ways	Permeable Pavements	Filter strips	Swales	Infiltration trenches	bio retention areas	Sand filters	Detention Ponds	Retention Ponds	Wetlands	Managed Aquifer Recharge
Quantity	Abstraction	X	P	X	S	X	X	X	X	X	X	P	S	P
	Infiltration	X	X	P	P/S/X	S	S	P	P	S	S	S	S	P
	Detention	P	S	S	P/S	S	S	S	S/P	P	P	P	P	P
	Conveyance	X	X	X	X	P	P	X	X	X	X	X	X	P/S
	Long-term storage	X	P	X	S	X	X	X	X	X	X	P	P	P
Quality	Sedimentation	P	PR	PR	X	P	S	PR	P	S	P	S	S	X
	Filtration	P	X	P	P	P	P	P	P	P	X	S	S	P
	Adsorption	P	X	P	P	P	P	P	P	P	X	S	P	P
	Biodegredation	P	X	P	S	P	S	S	P	S	X	P	P	S
	Plant-uptake	P	X	X	X	S	S	X	P	X	X	P	P	X
	Nitrification	P	X	X	X	S	S	S	P	X	X	P	P	X

Primary process (P) Secondary process (S) Pre-treatment Required (PR) Not applicable (X)

Figure 2.4 - SuDS 'treatment train' (Fisher-Jeffes et al., 2016)

Good housekeeping represents a group of interventions that minimize the amount of pollutants (e.g. solid waste) into the environment which would have otherwise ended up polluting runoff. Examples of these are public awareness campaigns and efficient operations and maintenance (Armitage et al., 2014).

Source controls represent a group of interventions that manage stormwater runoff as close as possible to its source. Examples of these are green roofs, rainwater harvesting, soakaways, and permeable pavements (Armitage et al., 2014).

Local controls represent a group of interventions that manage stormwater runoff in the greater local area, typically within the road reserves. Examples of these are filter strips, swales, infiltration trenches, bio-retention areas, and sand filters (Armitage et al., 2014).

Regional controls represent a group of interventions that manage the combined stormwater runoff from several developments at a common point. Examples of these are detention ponds, retention ponds, and constructed wetlands (Armitage et al., 2014).

Figure 2.5 provides an illustration of the interventions listed above.

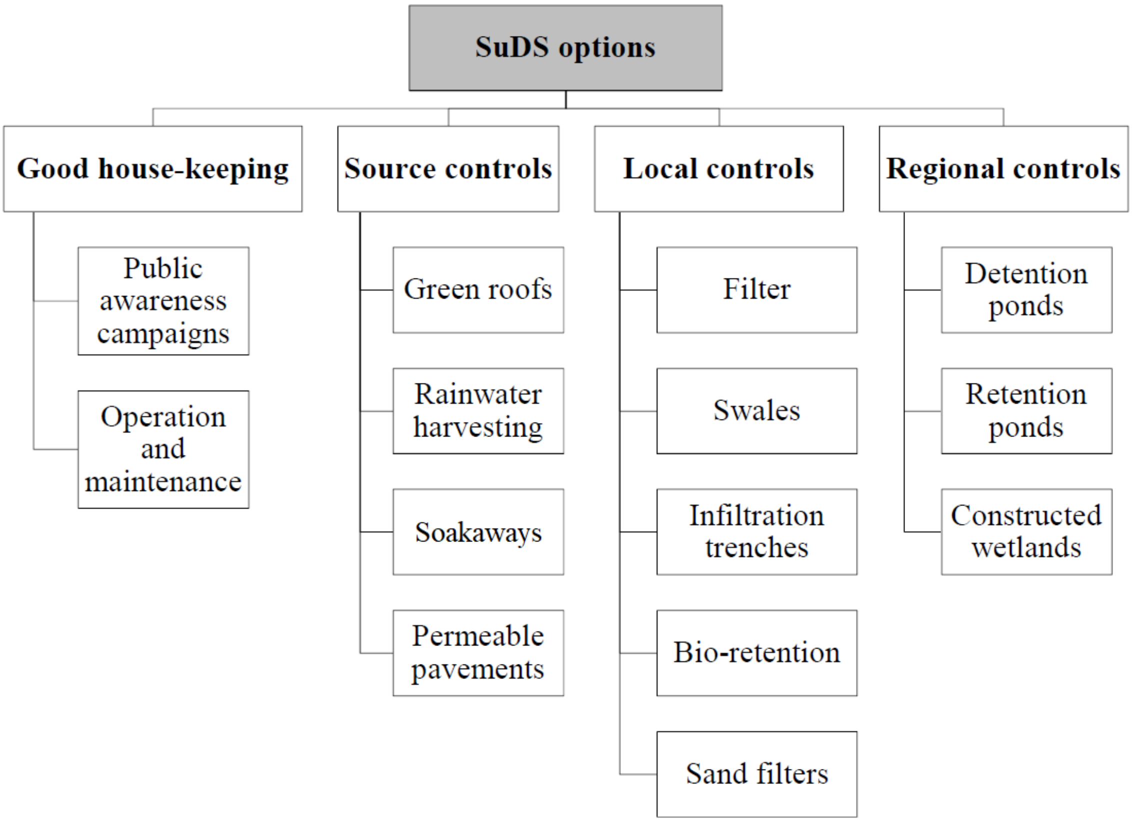


Figure 2.5 - Summary of SuDS interventions (Armitage et al., 2014)

Stormwater management in RSA has been historically dependent on the collection and disposal of runoff to the nearest water body. SuDS, however, has the potential to holistically manage runoff as a major component of WSUD. Table 2.2 provides a description of each of the SuDS options mentioned above.

Table 2.2 - SuDs Options

	SuDS options	Brief Definition	Example
Source Controls	Green roofs	A roof covered in living vegetation for reasons including reduction in runoff, visual benefit and enhanced ecological value (Woods Ballard et al., 2015) . Image A: eThekwini Green Roof Pilot Project, Durban CBD (Armitage et al., 2013b). Image B: Green roof at the University of the Witswatersrand (Source: Anne Fitchett)	 
	Rainwater Harvesting	The collection of runoff for temporary storage and use (Sharma et al., 2015). Kleinmond is a town outside Cape Town that has low cost houses (LCH) which are heavily reliant on the use of RWH. The Department of Science and Technology together with the Overstrand Municipality and the Council for Scientific Industrial Research (CSIR) in 2010 conducted a study to build 410 LCH in Kleinmond which are less reliant on municipal services. The project has been highly successful	

		and a good example to be replicated by other state entities providing social housing solutions Image: Rainwater harvesting tanks in Kleinmond township (O'Brien, 2014).	
	Soakaways	An excavated pit layered with coarse aggregate and other porous media, that are used to detain, filter and gradually discharge. Very similar to infiltration trenches (Armitage et al., 2013b). Image: Groundwater recharge of runoff from a single residential dwelling using soakaways (Armitage et al., 2013b).	
	Permeable Pavements	Permeable pavements allow runoff to infiltrate through the surface into the underlying sub-layers and/or strata (Woods Ballard et al., 2015). Image: Permeable pavements in the UK (Woods Ballard et al., 2015).	 

Local Controls	Filter Strips	Vegetated and maintained areas of land used to manage shallow overland runoff through a filtration process (Armitage et al., 2013b). Image: Vegetated filter strip in Cape Town (Armitage et al., 2013b).	
	Swales	Vegetated swales are shallow grass-lined channels that convey stormwater. They remove coarse and medium sediments and provide both a conveyance function for stormwater and treatment function through infiltration (Melborne Water, 2013). Fitchett (2017) conducted a study in Diepsloot (a low income settlement is Johannesburg) aimed at introducing SuDS in the settlement to enhance the existing surface-water quality and reduce the quantity of stormwater runoff in a low-cost manner. The study was successful and concluded that SuDS implementation in informal settlements are viable if community conditions are relatively stable and the community members are sufficiently incentivised. Image: Vegetated swale in Diepsloot, Johannesburg (Fitchett, 2017)	

	Infiltration Trenches	Excavated trenches filled with material such as rock, large granular material, or commercial void forming material. They are aimed at receiving runoff from neighbouring properties and links such as roads and walkways (Armitage et al., 2013b). Image: Infiltration trenches surround the parking lot in Kingston, USA (The University of Rhode Island, 2019).	
	Bio-retention Areas	They are landscaped depressions designed to manage the runoff from the first 25 mm of rainfall. They treat stormwater by passing the runoff through vegetated filter media (Armitage et al., 2013b & Melborne Water, 2013). Image: Bio-retention basin also referred to as Rain garden by roadside in Melbourne (Melborne Water, 2013).	
	Sand Filters	They are sedimentation chambers that are linked to underground filtration chambers. The chambers comprise of sand and other filtration media through which runoff passes. It removes coarse to medium sized sediments before release. (Armitage et al., 2013 & Melborne Water, 2013). Image: Sand filter in Melbourne (Melborne Water, 2013).	

Regional Controls	Detention ponds	Fairly large landscaped depressions that temporarily store stormwater runoff to decrease the downstream flood risk (Woods Ballard et al., 2015). Image: Detention pond or basin with landscaping also serving as an amenity space in Leicester UK (Woods Ballard et al., 2015).	
	Retention ponds	Designed by excavating below the natural groundwater level and lining the base to retain runoff. The construction of dam walls with a weir outlet structure allows stormwater coming into the pond to be mixed with the permanent pond water and released slowly over the weir at a much slower flow rate (Armitage et al., 2013b). Image: Retention pond in Cotswold Downs Golf Estate, Hillcrest, Durban (Armitage et al., 2013b).	

	Constructed Wetlands	Constructed wetlands are marshy areas that are shallow, heavily vegetated water bodies that remove pollutants through enhanced sedimentation, filtration and other pollutant removal processes. They provide a lively habitat for fish, birds and other wildlife (Armitage et al., 2013 & Melborne Water, 2013). Image : Grove Drive surface flow wetland in Singapore (PUB, 2018).	

2.4.3 Sustainable water supply

Sustainable water supply is defined as the consumption of water in a manner that does not deplete or permanently destroy the resource (Armitage et al., 2014). In addition, it describes the alternative approaches available to secure a community's water resource needs. The aim of sustainable water supply strategies in the context of the urban water cycle is to reduce potable water supply requirements, minimize wastewater generation, and maximize the use of alternative water sources. (Armitage et al., 2014) Figure 2.6 presents an illustration of how sustainable water supply strategies affects the urban water cycle.

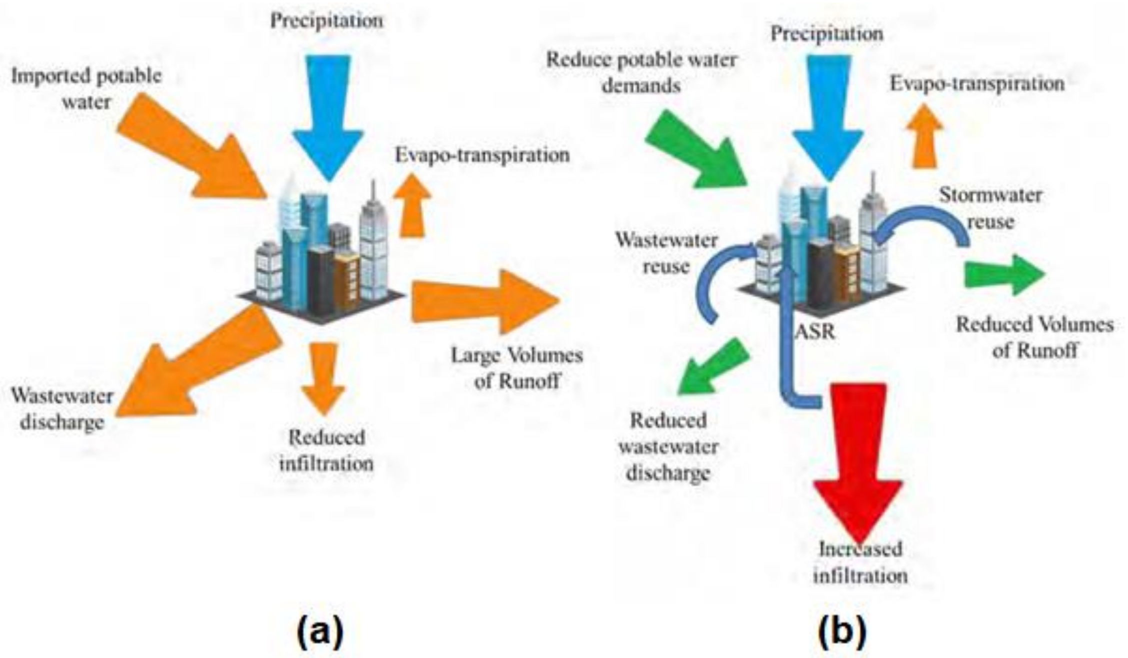


Figure 2.6 - The impact of sustainable water supply strategies (b) on a traditional urban water cycle (a) (Armitage et al., 2014 adapted from Hoban & Wong, 2006)

The aims of sustainable water supply strategies are achieved through the use of:

- Water Conservation and Water Demand Management (WCWDM) to reduce potable water demand and;
- Alternative water sources such as rainwater, stormwater, wastewater, and groundwater.

Water Conservation and Water Demand Management (WCWDM)

WCWDM strategies are developed and implemented to improve and optimize existing water resources. It emphasizes the effective management of water resources. Examples of WCWDM interventions are bulk metering, zoning, and accurate data collection among others. The NWRS2 outlines the following as the key benefits of WCWDM:

- WCWDM is the leading method that can be used to balance water supply and demand deficits.
- The development of new water infrastructure can be significantly postponed if WCWDM is successfully implemented.
- It has been proven that in many cases, the implementation of WCWDM can be more cost effective than new water infrastructure development.

WC and WDM have very similar yet distinct definitions and are describes as follows (Department of Water Affairs and Forestry, 2004 cited by Armitage et al., 2014):

Water Conservation (WC): "*decreasing water loss, protection of water resources, and the sustainable use of wate*r". WC essentially means doing more with less especially in drought conditions. Examples include reduction in garden water use, etc.

Water Demand Management (WDM): interventions that "*promote the sustainable use of water and reduces its demand"*. WDM concentrates mostly on the operational approaches that can be used to achieve the goals of sustainable water supply. Examples include reducing non-revenue water (NRW) due to losses in the distribution network, reducing water wastage at the point of consumption (policies that influence user behavior) and replacing potable water on a 'fitness for purpose' basis (the use of alternative water supply options such as rainwater, greywater, groundwater, and seawater)

Rainwater & stormwater harvesting

Rainwater harvesting refers to the collection, conveyance, and storage of precipitation from building roofs. This water is considered less contaminated than the water retained from stormwater harvesting (SWH). SWH refers to the collection of runoff, from ground surfaces such as roads and car parks. It typically occurs on a larger scale when compared to RWH as such requires larger infrastructure to contain the volume collected.

They are both great means of reducing the demand for potable water supplies (Armitage et al., 2014).

Fisher-Jeffes et al. (2016) recommend RWH & SWH as a means to substantially increase the supply of freshwater to urban areas, there are however a number of obstacles to its widespread use that needs to be addressed before it can become economical. These obstacles include the appropriate sizing of tanks, treatment of collected water, etc. In RSA, water for use in urban areas is conventionally supplied from the impoundments of rivers (dams) forming surface water schemes. These water schemes are of high quality water and are normally situated outside the urban areas they serve. SWH is described as the collection and storage of urban runoff for subsequent use and slow release into the environment. The planning and design of urban land use, as well as the design of the SWH collection systems, forms an important role in its success (Lim et al., 2011). In essence, RWH & SWH are a crucial aspect of WSUD as it contributes towards both stormwater management and sustainable water supply

2.4.4 Benefits of WSUD

The urban water cycle has four streams namely stormwater, wastewater, groundwater, and water supply. All four streams require different management solutions but are inherently linked. The application of WSUD to all four streams will result in the holistic management of the urban water cycle hence achieving the desired social, economic and environmental benefits. The main benefit of WSUD is to provide ecologically sustainable developments by carefully including all facets of the water cycle. Through these, sustainable developments can achieve sustainable urban water management (Armitage et al., 2014).

The key benefits of WSUD include the following:

It reduces and delays stormwater runoff volumes by utilizing the natural retention and absorption abilities of soils and vegetation. The systems are designed such that there is increased stormwater infiltration, thus reducing the quantity of runoff that would have otherwise entered sewer systems, and eventually into receiving bodies such as streams, rivers, and lakes (CoJ, 2019). It subsequently reduces the requirement for rehabilitation and maintenance of downstream waterbodies (Water by Design, 2010). It also improves the quality of surface water.

It enhances groundwater recharge by improving the rate and quality of groundwater aquifer recharge and replenishment. The significance of this is that groundwater provides an important percentage of the water required to maintain normal base flow rates into rivers and streams.

It reduces stormwater pollutants: it reduces sediments and nutrient loads associated with runoff flows by ensuring optimal infiltration of runoff close to its source as such prevents pollutants from being transported to receiving water bodies (Water by Design, 2010). Infiltration is significant because once in the soil, plants and microbes have an opportunity to naturally filter and break down common pollutants found in runoff (CoJ, 2019).

It reduces the demand for treated municipal water supply thereby delaying the requirement for infrastructure upgrade costs.

It limits the frequency of sewer overflow events by reducing runoff volumes and delaying peak discharges through infiltration.

Affordable food production: Urban agriculture such as community gardens, kitchen gardens, productive street verges, backyards, and school vegetable gardens are considered a form of green infrastructure that provides benefits such as providing a secure and healthy food supply close to its consumers as it has reduced transport cost. Urban agriculture is mostly successful in peri-urban areas (i.e. outskirts of the hinterland) providing more sustainable food sources for its residents.

Air Quality: Due to the extensive incorporation of trees and vegetation in WSUD, the greenery contributes towards improved air quality. Trees and vegetation absorb pollutants from the air thereby reducing the effects of climate change.

Social benefits: WSUD enhances the quality of life in urban areas by improving local aesthetics and providing urban amenity which contributes towards the ‘walkability’ of streets. According to Coutts and Hahn (2015), the presence of trees and green spaces allows for social interaction and community building thereby reducing the levels of inner-city crime, violence, improving academic performance, and reducing the symptoms associated with attention deficit and hyperactivity disorders (CoJ, 2019). In addition to passive recreational value due to the enhanced value of local amenity, it also enhances the quality of regional waterbodies downstream (Water by Design, 2010).

2.5 Conclusion

The literature review was undertaken to gather current and relevant information available on WSUD. Information on the benefits associated with the application of WSUD was also sought. The main focus of the literature review was peer reviewed journal articles over the past 0 -10 years that undertook WSUD research. International and local examples were reviewed. The review established the status quo of WSUD studies previously carried out. The findings of the literature review determined the approach to this case study.

3. RESEARCH METHODOLOGY

3.1 Approach

To address the four objectives of this study articulated in section 1.3, the following methods were employed:

- A desktop study involving the collection of existing information e.g. site layout plans, feasibility study reports, rainfall data, geotechnical reports and existing bulk infrastructure assessments.
- A visual assessment of the case study area.
- The calculation of water demand per land use. This was determined from first principles using engineering formulae and procedures set out in South African design manuals such the Neighbourhood Planning and Design Guide (commonly referred to as the Red Book) (DHS and CSIR, 2019) and the City of Ekurhuleni's guideline to water and sewer design (COE, 2006). The water network was designed to accommodate the peak hour domestic demands as well as firefighting water requirements. The analysis of the network was done using a computer design software called TECHNoCAD-WaterMate, to determine the optimal network.
- The calculation of Pre- and Post- development run-off was determined from applying engineering formulae and procedures set out in the South African National Roads Agency Limited (SANRAL) Drainage Manual (SANRAL, 2013) and the Neighbourhood Planning and Design Guide (DHS and CSIR, 2019). The Rational Method was used to determine pre- and post- development runoff. The computer software PCSWMM, was then used to model rainfall and runoff and determine pre- and post- development runoff based on a continuous model analysis. The analysis of the stormwater pipe network was done using the computer design software called TECHNoCAD-PipeMate.
- PCSWMM is a derivative of the US Environmental Protection Agency (EPA) Stormwater Management Model (SWMM). SWMM was developed by the EPA in 1971 and has had numerous improvements over the years. SWMM is a stormwater modelling software that allows for dynamic rainfall-runoff modelling, and can be used to model long term or single rainfall events. The software is also able to simulate the quantity and quality of runoff generated (Rossman, 2015).

- PCSWWM is a stormwater modelling software package developed by CHI International which significantly improves the usability and functionality of SWMM. PCSWMM is a computer based model that determines parameters based on the physical characteristics of the drainage area and utilizes continuous records of measured rainfall data. It is a valuable tool and will be utilized in this design. The main reasons for this are:
  - SWMM is the most widely used stormwater management software and the best tested in the world.
  - Most of the input variables such as soil properties, physical characteristics of the catchment, land use and topography are physically measurable.
- The Rational Method, as detailed in the SANRAL Drainage Manual was also used to determine runoff volumes. It is however not recommended/realistic for calculation of the effectiveness of stormwater management interventions because:
  - Its unable to determine the different runoff coefficients “C” corresponding to different storm durations. The value of the runoff coefficient should vary as the moisture conditions of the soil changes.
  - This method assumes that the recurrence interval of the runoff is the same as the recurrence interval of the storm.
  - It also assumes that the design rainfall is of uniform intensity and duration which is at least the time of the concentration of the drainage system.

  The use of the Rational Method will be compared to the computed methods that consider the effects of soil moisture conditions and design storm types, among others.
- TECHNoCAD is a computer based software that is used for the design of civil engineering urban infrastructure services such as Roads, Sewer reticulation, Stormwater reticulation, Water supply and Digital Terrain Modelling. The two packages to be utilized in this research are PipeMate and WaterMate. PipeMate is the urban sewer and stormwater design and draughting package. WaterMate is the pressurised water reticulation analysis/time simulation, design and draughting package. The software is able to produce final working layout and longsection drawings with minimum amount of manual input while also incorporating hydraulic design functions (TECHNoCAD, 2019).

3.2 WSUD design methods

The selection of WSUD options were determined by the unique characteristics of the site. It was highly unlikely that all options will be applicable and/or effective on any one site (Armitage et al., 2013a). The following were however undertaken in this study:

- Water: Rainwater Harvesting (RWH) is a potential source of water supply and an essential part of WSUD. It can be used effectively by domestic households to replace some or all non-potable water demand and reduce runoff volumes entering storm water drainage systems (Water by Design, 2010). In this case study, the hydrologic design and assessment of RWH systems was investigated to augment the required non-potable water demand. A dual household water network is envisioned for this development as rainwater quality may not always meet drinking water standards. The required potable water from the nearby municipal reservoir will be transported by a water distribution network to end-users in the development.
- Stormwater: Sustainable Urban Drainage Systems (SuDS) generally includes a number of options and combinations that make a treatment train. The methodology followed is briefly summarised below:
  - Hydraulic determination of the post-development storm water volume to be treated
  - Mapping out of preferred flow path based on the township development plan
  - Determination of the type and location of the various SuDs options
  - Determination of the performance of the identified SuDS for the different design storms
  - Quantification of the contribution of each SuDS in the reduction of stormwater run-off.

3.2.1 Design principles

WSUD utilizes the physical characteristics as well as the hydrological characteristics of the site to provide interventions that use infiltration, runoff storage, runoff conveyance, and filtration to manage urban runoff. Detailed below is a list of these approaches (CoJ, 2019):

Infiltration: Interventions in this group include infiltration basins, rain gardens, porous paving, and rainwater spouts. They reduce runoff by capturing and allowing it to infiltrate the soil. This reduces the amount of runoff and consequentially replenishes groundwater resources.

Runoff storage: Interventions in this group include rainwater harvesting systems, depression storage in medians and landscaped islands, green roofs and swales with hydraulically controlled flow structures. These interventions harvest runoff from impervious areas such as roofs and paving for future infiltration, release or storage. They reduce peak flows and control the volume of discharge into receiving water bodies.

Runoff conveyance: Interventions in this group include swales and grass-lined channels they reduce runoff velocity and lengthens the conveyance route thus reducing and delaying peak flow. Conventional inlet curbs and gutters can be replaced with alternatives with rougher surfaces that will reduce velocity allowing for evaporation and settlement of sediment to take place.

Filtration: Interventions in this group include bio-retention ponds, rain gardens, vegetated swales and vegetated filter strips. They filter the runoff through a medium similar in process to infiltration. It allows for the distillation of solid particles and dissolved pollutants.

It is recommended that retention storage, combined with infiltration and evapotranspiration, are the ideal methods for runoff volume reduction. Wet ponds (Detention Basins) and biofiltration are ideal for managing the quality of stormwater runoff (CoJ, 2019).

3.2.2 RWH design guidelines

Rainwater Harvesting (RWH) in urban South Africa is generally viewed as a temporary water supply measure for houses yet to get access to a municipal water supply. As such the government mostly focuses on providing RWH systems to rural communities and agriculture efforts (Department of Water Affairs, 2013). The high cost, lack of aesthetic appeal of RWH tanks, concerns around water quality, consistency of supply and the limited water savings that can be achieved are stated as some of the main limitations on the use of RWH tanks in the domestic setting (Mpofu, 2019). Fisher-Jeffes (2015) however stated that there is considerable potential for RWH in South Africa as it has the potential to significantly reduce potable water demand. Rainwater is a commodity that can provide both potable and non-potable water to communities. This section aims to explain the methodology utilized in sizing the RWH tanks.

The Ndiritu et al. (2018) generalized guidelines form the basis for this section. The guidelines are based on the relationships between storage size, catchment area, rainfall yield and reliability. Rainfall was obtained using data from daily time-step simulations of RWH systems. The concept of hydrologic optimality was then applied. The most hydrologically optimal RWH system falls on the Pareto front. The Pareto front is described as a point at which allocation is optimal. An alternative allocation that improves one variable cannot be made without reducing any other variables (Legriel et al., 2010 cited by Ndiritu et al., 2018). At the Pareto front, the storage size is minimized while yield and reliability are maximized. Rainfall availability and roof area are the main variables that contribute to the quantity and variability of water available to collect. Reliability is defined as the percentage number of days in a year when the sized rainwater tank is able to supply the non-potable water demand (Imteaz et al., 2011). Yield is defined as the expected number of days of supply per year (Ndiritu et al., 2018).

The Ndiritu et al. (2018) generalized guidelines were developed such that they are independent of location by averaging the variables obtained from the simulations from the 8 rain gauge stations. This was done by performing daily time step simulations for each rain gauge station data. The simulations were carried out for available supply to demand ratios of 0.1, 0.3, 0.5, 0.7 and 0.9 and the optimal tank volume obtained for reliabilities of 50, 66.7, 80, 90, 95 and 98 %. The probability of exceedance i.e. respective levels of assurance of days of supply for the above reliabilities is 1 in 2 years, 2 in 3 years, 4 in 5 years, 9 in 10 years, 19 in 20 years and 49 in 50 years.

RWH systems are usually not the main water supply source and tend to be complementary. As such, high reliabilities of 90% and above will not typically be required. The generalization consequently includes reliabilities as low as 50%. This is also beneficial for establishing realistic life cycle cost benefit analyses (Ndiritu et al., 2018). The generalization was carried out in two steps: *"(i) finding appropriate non-dimensionless ratios of the optimal hydrologic components obtained from the* *simulations, and (ii) graphically presenting the relationships of these non-dimensionless ratios and other variables efficiently to obtain design charts"* (Ndiritu et al., 2018).

The design charts of the Ndiritu et al. (2018) generalized guidelines are presented in Figure 3.1. Ndiritu et al. (2018) concludes that the design charts, when correctly used, can accurately size RWH systems. They can also be used to assess the performance of an existing system and thus establish alternatives for its enhancement.

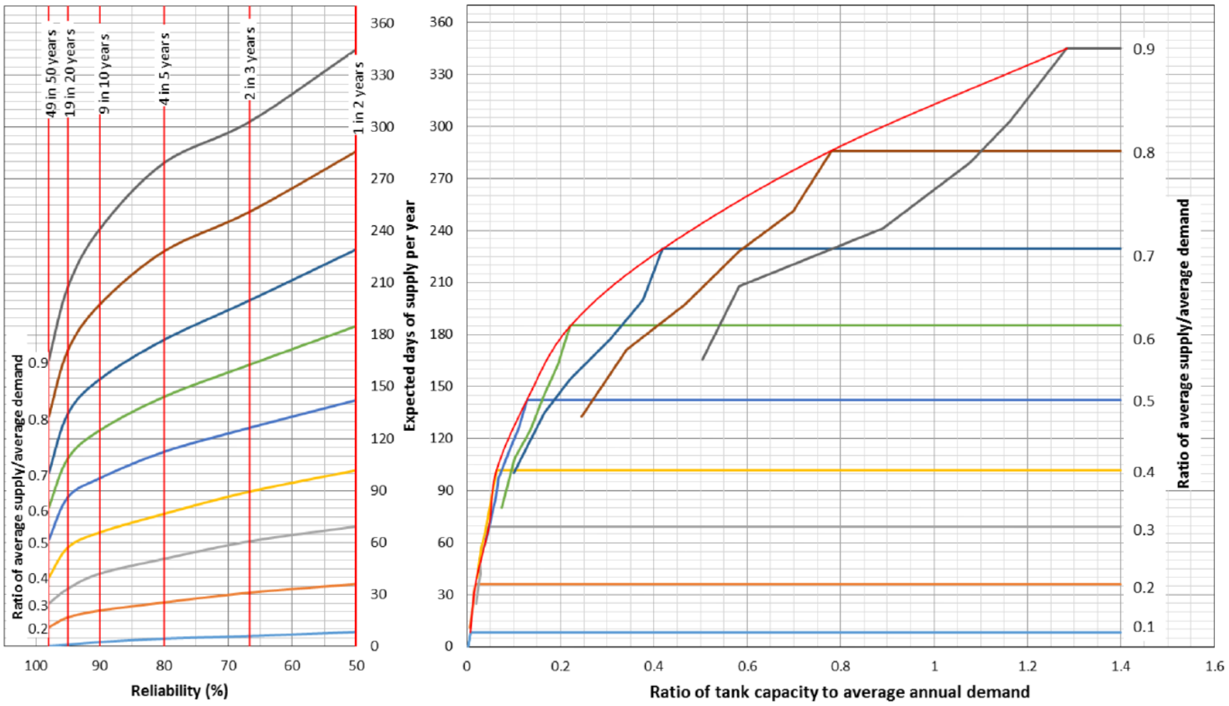


Figure 3.1 - Generalised rainwater harvesting design and assessment charts for Johannesburg (Ndiritu et al., 2018).

The Ndiritu et al. (2018) generalized guidelines formed the basis for the development of the Mpofu (2019) generalised Storage-Yield-Reliability relationships for the sizing of Rainwater Harvesting System (RWHS). The RWH design in this case study was conducted using the generalized RWHS models by Mpofu (2019). The model follows the same hydrologic design and assessment format by Ndiritu et al. (2018).

According to Mpofu (2019), there are several parameters influencing the accuracy of behavioural models. Whilst they are the most dependable method of sizing RWHS, they, however, require a lot of input data in order to obtain reliable results. The RWHS design parameters required to determine the optimal size of RWHS components include the mean annual precipitation (MAP), non-potable demand, rainwater losses (i.e. water collection efficiency coefficient and first flush devise), reliability of supply and roof catchment area. With rainfall being the most critical hydrological parameter used in the model. The accurate determination of non-potable water demand impacts the yield and reliability of an RWHS.

Gould and Nissen-Petersen (1999) describe rainwater collection efficiency coefficient as the ratio of volume of runoff to volume of rainwater collected. It is a coefficient that accounts for the various rainwater losses that can occur before the tank. Examples of these rainwater losses include evaporation, the influence of roof shape and slope, leakage of gutters, filter screens and depression storage. The accurate estimation of losses and its inclusion in an RWHS model leads to more accurate system performance designs (Mpofu, 2019)

Ndiritu et al. (2018) presents the above inputs into a dimensionless coefficient denoted as the ratio of average supply to average demand ($\boldsymbol{R_{SD}}$) as shown below.

$$R_{SD} = \frac{\eta \text{ x } A \text{ x } r_t}{D_t} \tag{3.1}$$

Where: $\boldsymbol{\eta}$ is the efficiency of rainwater collection, $\boldsymbol{A}$ is the roof area contributing to runoff (m^2), $\mathbf{r_t}$ is the average rainfall over a period (m/y) or MAP, $\boldsymbol{D_t}$ is the average non-potable annual demand (m^3/y). The dimensionless nature of the supply to demand ratio allows for an unlimited number of combinations of catchment area and demand calculations to be done without being user specific.

In this case study analysis, water users are grouped into several categories including residential, commercial, educational and institutional. Many studies accurately analyse end use water demand characteristics that are vital in the determination of non-potable demand. This has however mostly focused on residential/domestic users. The limited availability of end use information on the other water user groups meant that alternative information sources needed to be used to develop the required data for use in the case study. The non-potable water uses identified in this study include toilet flushing, air conditioning cooling towers, irrigation, and laundry. Most of these users are in part dependent on the number of people in a household and the frequency of the event.

Various studies were considered when determining the non-potable demands. Table 3.1 presents the typical end use of non-potable demands for residential users.

Table 3.1 - Typical end use non-potable demands for residential users (O'Brien, 2014)

Tank Water End Use Demand Data for Residential 2			
Parameter	**Household Size (PPH)**	**Value (ℓ/day)**	**Source**
Toilet = 3 events/c/d Volume per event = 9ℓ/event	4	108	There exist one standard toilet in each household. According to Jacobs et al 2017 one flush event is equivalent to 9 ℓ.
Laundry = 0.143 events/c/d Volume per event = 62ℓ/event	4	35.464	Rudin (2008) cited by O'Brien (2014) indicates the laundry without a washing machine demand at 62 ℓ per wash. Jacobs et al 2017 indicates washing machine demand as 80 ℓ/event.
Cleaning = 2 events/c/d Volume per event = 6ℓ/event	4	48	Jacobs et al (2017) indicates a typical volume for kitchen sink events of 6ℓ/event which is adapted for this study, as it is the most realistic value.
Gardening = 1 events/c/d Volume per event = 5ℓ/event	N/A	5	Jacobs & Haarhoff (2004a) cited by O'Brien (2014) indicates an outside tap demand of 5 ℓ/event, which can be adopted for garden use. The gardens for the Res 2's are very small, making Jacobs et al 2017 estimate of 82 ℓ/day too large an estimate as it applies to irrigated areas of $75m^2$
Tank Water End Use Demand (ℓ/day)		196.464	

Rainwater losses are grouped into two categories, general losses, and first flush losses. General losses are described as water losses to the system that occur as a result of evaporation, leakage of gutters, initial abstraction through depression storage, loss of water in filters, etc. These losses are expressed as a water loss coefficient or the water collection efficiency coefficient. The water collection efficiency coefficients for different types of roof materials and alignments range from 0.6 – 1 and are presented in Table 3.2 (Mpofu, 2019). Ndiritu et al. (2018) recommend a median Figure of 0.8.

Table 3.2 - Water Collection Efficiency Coefficients (Mpofu, 2019)

Reference	Water Collection Efficiency Coefficient	
(Gould & Nissen-Petersen, 1999)	Sheet-metal: Cement tile: Clay tile (machine made): Clay tile (handmade):	0.8 - 0.85 0.62 – 0.69 0.30 – 0.39 0.24 – 0.31
(Fewkes & Warm, 2000)	Pitched roof without filtration (tile/slate): Pitched roof with filtration (tile/slate): Flat roof with impervious membrane: Flat green roof:	0.9 – 1 0.75 – 0.9 0 – 0.5 0 – 0.5
(Liaw & Tsai, 2004)	Inverted V (metal): Flat (cement): Parabolic (metal): Saw-tooth (metal):	0.82 0.81 0.81 0.83
(UN-Habitat, 2005)	Tiles: Corrugated metal sheets:	0.8 - 0.9 0.7 – 0.9
(Woods Ballard et al., 2015)	Pitched roof with profiled metal: Pitched Roof (tile): Flat roof (no gravel): Flat roof (gravel): Intensive green roof: Extensive green roof:	0.96 0.9 0.8 0.6 0.3 0.6

Table 3.3 from Armitage et al. (2013b) indicates commonly used runoff coefficients.

Table 3.3 - Typical runoff coefficients for rainwater harvesting off roof (Armitage et al., 2013b).

Roof classification	Runoff coefficient C
Pitched roof, tiled	0.85
Flat roof, tiled	0.6
Flat roof, gravel	0.4
Extensive green roof	0.3
Intensive green roof	0.2

A 'first flush' system is considered in an RWHS as it serves as a preliminary treatment system for improving rainwater quality. In more sophisticated systems, a first- flush system can then be followed by a filtration process which then ensures the effective removal of suspended impurities. In RHW, a first flush device discards the first amount of roof runoff before it enters the tank. It serves as a contaminant mitigation measure to improve water quality during rainfall events (Sharma et al., 2015). Mpofu (2019) presents a list of recommended first flush quantities. Several assumptions underlie the recommended values making it prudent that caution is taken when selecting the appropriate depth to use in an analysis. Most recently, the World Health Organization

(WHO, 2017), recommends an initial roof cleaning wash volume of 20 – 25 litres. In this study, a minimum first flush depth value of 0.2mm was used.

Mpofu (2019) developed generalized relationships using data obtained from the daily time-step simulation of inflows and outflows through a typical RWHS. These simulations were then evaluated using regression analysis with the aim of establishing the best fit mathematical form between the previously identified dimensionless variables of Ndiritu et al. (2018). The reliabilities considered included: 50%, 66%, 80%, 85%, 90% and 95%. These translate to levels of assurance of 1-in-2 years, 2-in-3 years, 4-in-5 years, 5.7 in 6.7 years, 9-in-10 years, and 19-in-20 years respectively.

The following generalized models, for the case of constant demand, were proposed:

The Supply-to-Demand Ratio represented in equation 3.1 was employed.

Below is the proposed generalized model relating the yield to Supply-Demand (S-D) ratio & reliability:

$$\boldsymbol{S_{p-r}} = \frac{\boldsymbol{N_r}}{\mathbf{365.25}} = \boldsymbol{\alpha R_{SD}}^{\boldsymbol{\beta}} \qquad \textit{(3.2)}$$

Where:

α = 1.362(1−r)0.207	$\boldsymbol{S_{p-r}}$ = Yield ratio
β = 1.769e−0.445(1−r)	$\boldsymbol{N_r}$ = Expected number of days of rainwater supply per year
$\mathbf{0.5 \leq \boldsymbol{r} \leq 0.95}$	$\boldsymbol{R_{SD}}$ = Supply to demand ratio
	$\boldsymbol{\alpha}$ and $\boldsymbol{\beta}$ = Coefficients of regression
	$\boldsymbol{r}$ = Reliability

Below is the proposed generalized model relating the yield to storage-demand & reliability.

$$\boldsymbol{R_{TD-r}} = \boldsymbol{\gamma e}^{\varphi(\boldsymbol{S_{p-r}})} = \frac{\boldsymbol{K_r}}{\boldsymbol{D_t}\ \mathbf{x\ 365.25}} \qquad \textit{(3.3)}$$

Where:

γ = 0.009e1.005(1−r)	$\boldsymbol{R_{TD-r}}$ = Storage-Demand ratio

$\varphi = 8.207e{-}0.996(1{-}r)$	$\boldsymbol{\gamma}$ and $\boldsymbol{\varphi}$ = Coefficients of regression
$\mathbf{0.5} \le \boldsymbol{r} \le \mathbf{0.95}$	$\boldsymbol{r}$ = Reliability
	$\boldsymbol{K_r}$= Storage required to achieve reliability r
	$\boldsymbol{D_t}$ = Average annual demand

Flush Depth:

$$\boldsymbol{FF} = (\boldsymbol{ff_d}) \times \boldsymbol{n_r} \tag{3.4}$$

$\mathbf{r_t}$ which is the average rainfall over a period (m/y) or MAP is then reduced by the value of the $\boldsymbol{FF}$. Substituting the above equation into $\boldsymbol{R_{SD}}$ equation 3.1 gives the adjusted S-D ratio.

$$\boldsymbol{R_{SDadj}} = \frac{\boldsymbol{\eta A(r_t - FF)}}{\boldsymbol{D_t}} \tag{3.5}$$

Where:

$\boldsymbol{FF}$ = mean annual first flush depth
$\boldsymbol{ff_d}$ = Desired first flush depth (0.2mm, 1.5mm or 3mm)
$\boldsymbol{n_r}$ = average number of rain days expected in a year at a particular location (93 Days)

3.2.3 The conceptual model of the SuDS process in PCSWMM

The modelling of SuDS in PCSWMM allows for the use of a combination of detention, infiltration and evapotranspiration. SuDS controls are represented by a combination of layers or zones in SWMM. Figure 3.2 shows a cross section of a bio-retention cell which is the only SuDS control that utilizes all four zones illustrated below. Other SUDS are usually represented by a subset of the zones (CHI, 2019).

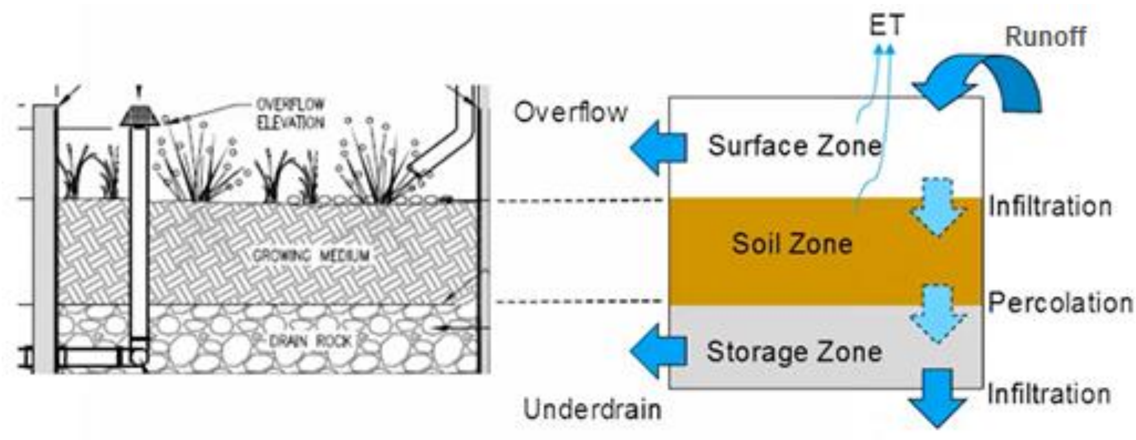


Figure 3.2 - SuDS Zones (Rossman, 2015 & CHI, 2019)

The **surface zone** represents the ground surface that receives rainfall and runoff from upstream. It stores excess inflow in depression storage and generates surface outflow that either enters the drainage system or flows onto downstream sub-catchments.

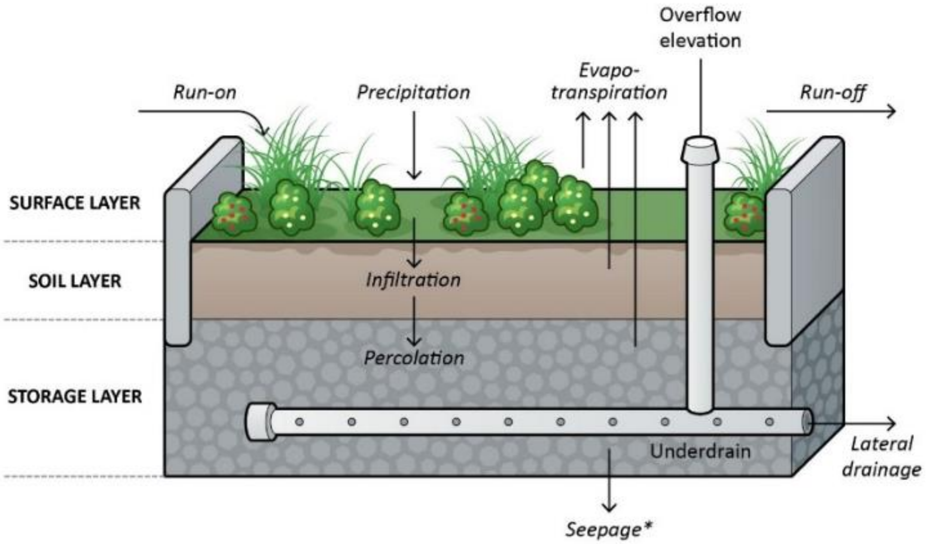
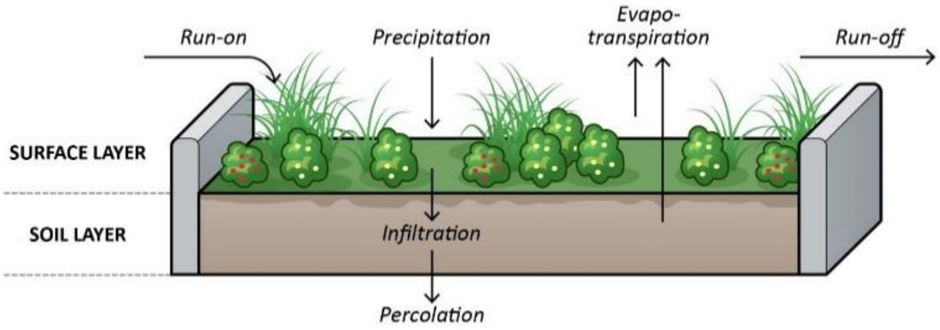
The **soil zone** is the engineered soil layer used in bio-retention cells and rain gardens to support vegetation growth. This layer also percolates water into the underlying storage zone.

The **storage zone** is a bed of crushed rock or gravel that provides storage in bio-retention cells, permeable pavement, and infiltration trench systems.

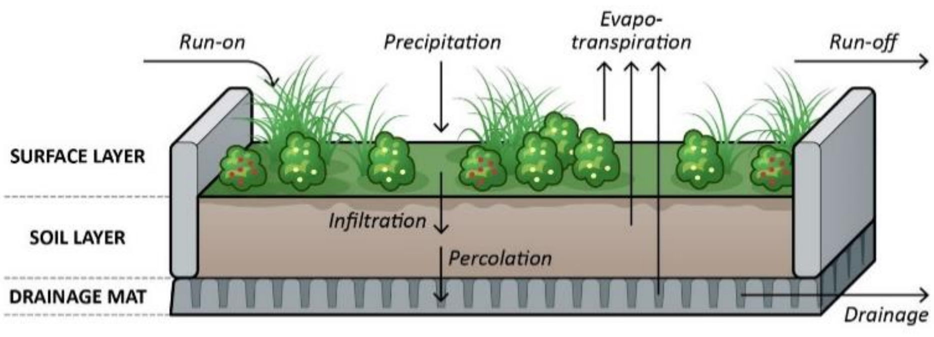
The **underdrain system** conveys water out of the gravel storage layer of SuDS into a common outlet pipe or chamber. These underdrain systems are usually made of slotted or perforated pipes.

All of the SuDS interventions are designed such that they are able to provide some degree of runoff storage and evaporation of stored water with the exception of RWH. Infiltration into the natural soil most commonly occurs in vegetated swales, bio-retention cells, porous pavement systems, and infiltration trenches (Rossman, 2015 & CHI, 2019).

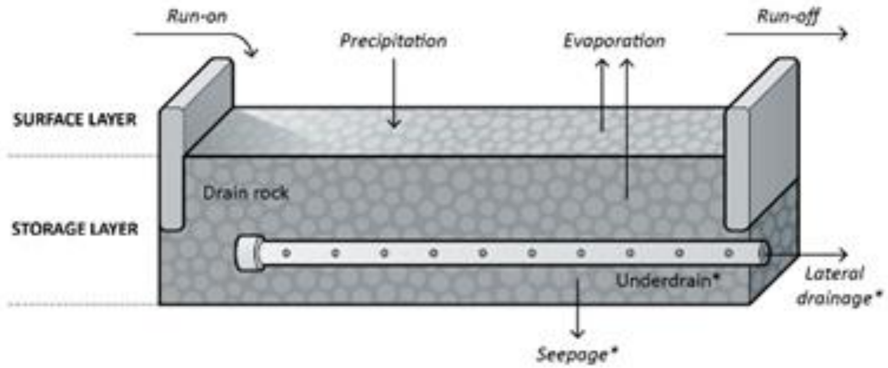
The PCSWMM software can model eight generic types of SuDS interventions, illustrated below (CHI, 2019):

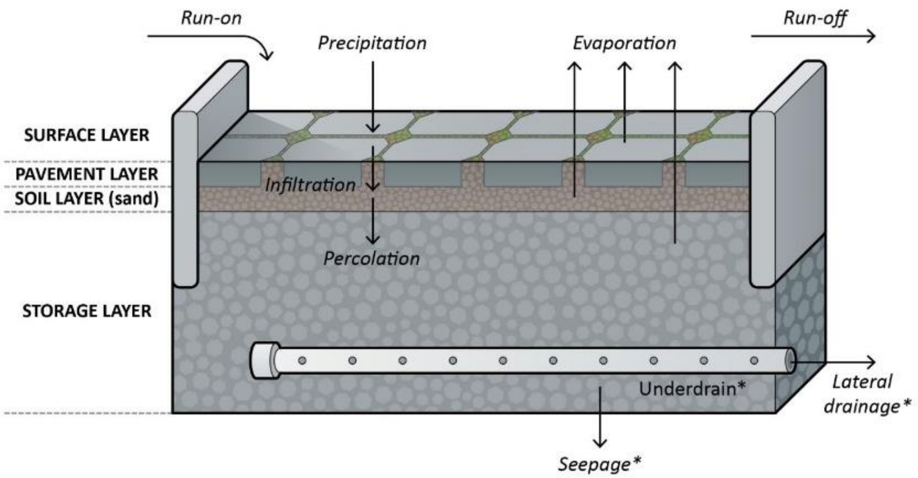
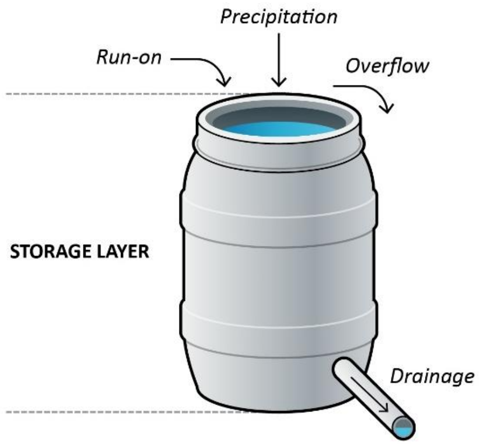
Bio-retention cells modelled to represent vegetation grown in engineered soil over a gravel drainage bed. It provides storage, infiltration, and evaporation.  	**Rain gardens** modelled similar to bio-retention cells with vegetated surface storage and engineered soil layer but without the underlying drainage layer. 

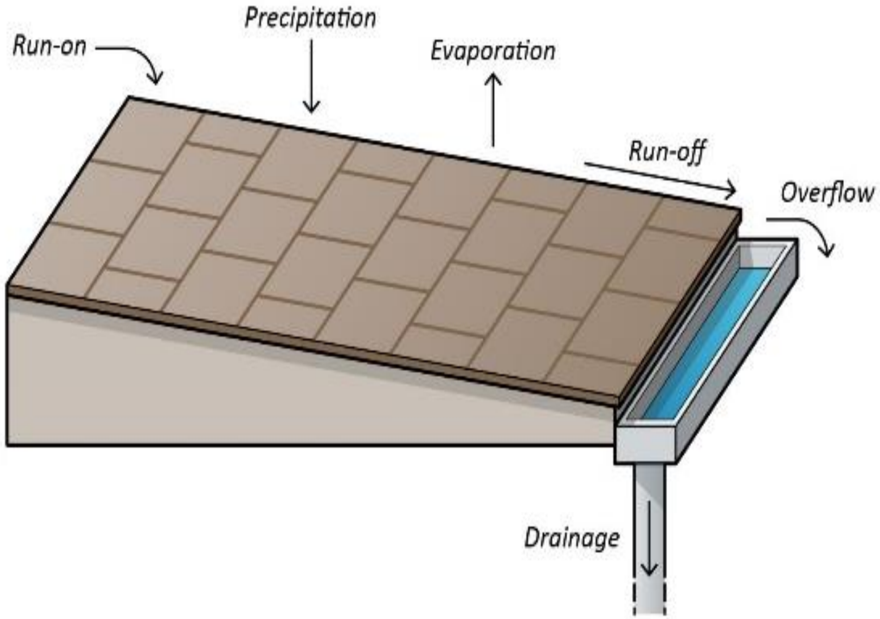
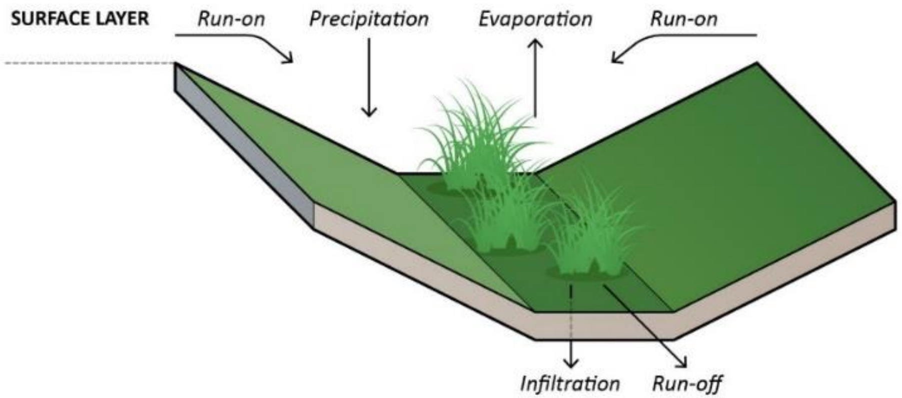
Green roofs similar to bio-retention cells. Usually, shallow soil layer laying above a drainage mat that collects and conveys excess infiltrated rainfall to the roof drainage system.



Infiltration trenches are storage facilities underlain by granular media that capture runoff. They are designed such that they capture and hold runoff for a period of time to enhance infiltration into the insitu soil below.



Pervious pavements are underlined by gravel such that rainfall percolates the pavement into the gravel storage layer below where it can infiltrate at somewhat natural rates into the insitu soil.  	**RWH tanks** are modelled as rain barrel in PCSMM. The tanks collect roof runoff during storm events and release for reuse. The input parameters are tank size and outflow rate. 

Rooftop disconnection are downpipes that liberation to pervious landscaped areas instead of directly into stormwater drains. It allows for the interception of the rainfall. They slow down the flow rate of collected runoff. It can be modelled to represent roofs with directly connected downpipes that overflow onto previous areas. 	**Vegetative swales** are modelled with sloping sides covered with grass and/or other vegetation. They intercept and slow down the flow rate of runoff and allow for infiltration and evapotranspiration. 

4. THE CASE STUDY SITE

4.1 Description of the site

The proposed development, Villa Lisa Extension 4, is located south of Boksburg, within the City of Ekurhuleni in Gauteng, South Africa. The image below shows the locality Map of the site.

Figure 4.1 - Locality map of development (Aseda, 2017)

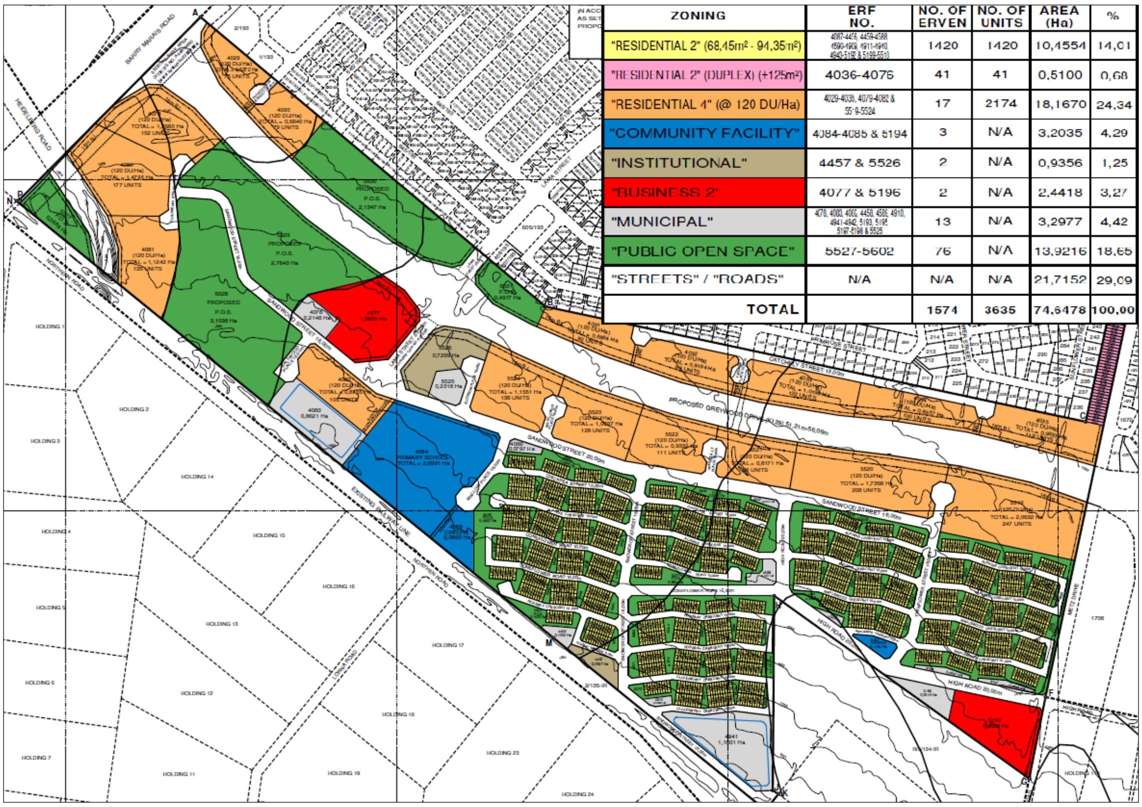
Villa Lisa Extension 4 was chosen as a case study for this research, due to the fact that it is a greenfield project making its features suitable for this study. The data required to perform the proposed methodology was also available. The greenfield development also allows for the comparison between conventional design and WSUD. This housing development is planned to include low cost social housing (commonly called RDP's - Reconstruction and Development Programme) and RDP Walk-ups.

The case study area is on topography with a gentle slope and covers an area of approximately 75 ha. There are 1574 erven in total. The erven are zoned as:

- 1461 erven for free standing houses
- 17 erven for high rise residential buildings
- 3 erven for community centres
- 2 erven for institutions e.g. schools.
- 2 erven for business or commercial use
- 13 erven for municipal use

- 76 erven for public open spaces.

The figure 4.2 shows the approved site development plan.



ZONING	ERF NO.	NO. OF ERVEN	NO. OF UNITS	AREA (Ha)	%
"RESIDENTIAL 2" (68,45m² - 94,35m²)	4087-4456, 4459-4588, 4590-4909, 4911-4940, 4943-5192 & 5199-5510	1420	1420	10,4554	14,01
"RESIDENTIAL 2" (DUPLEX) (±125m²)	4036-4076	41	41	0,5100	0,68
"RESIDENTIAL 4" (@ 120 DU/Ha)	4029-4035, 4079-4082 & 5519-5524	17	2174	18,1670	24,34
"COMMUNITY FACILITY"	4084-4085 & 5194	3	N/A	3,2035	4,29
"INSTITUTIONAL"	4457 & 5526	2	N/A	0,9356	1,25
"BUSINESS 2"	4077 & 5196	2	N/A	2,4418	3,27
"MUNICIPAL"	4078, 4083, 4086, 4450, 4589, 4910, 4941-4942, 5193, 5195, 5197-5198 & 5525	13	N/A	3,2977	4,42
"PUBLIC OPEN SPACE"	5527-5602	76	N/A	13,9216	18,65
"STREETS" / "ROADS"	N/A	N/A	N/A	21,7152	29,09
TOTAL		1574	3635	74,6478	100,00

Figure 4.2 - Site Development Plan (Aseda, 2017)

4.2 Objectives of the case study

The case study intends to compare the conventional design of stormwater and potable water systems to WSUD best practices in greenfield development. The results of both designs will be analyzed against costs and benefits. The specific objectives of this study are listed below:

- To identify applicable conventional and WSUD solutions for a greenfield township development in Johannesburg.
- To determine Pre- and Post- development run-offs using both methods.
- To calculate and compare the cost of conventional and WSUD stormwater and potable water infrastructure.
- To assess potential savings and/or losses (if applicable) from the implementation of recommended WSUD solutions in relation to conventional infrastructure solutions.

4.3 Technical Surveys

4.3.1 Existing Services

The proposed township development site and its environs currently fall within the Dawn Park Tower water distribution zone. Water supply to the proposed development will be via the Dawn Park Reservoir. An existing 500mm uPVC water pipe is currently located through the development, from the northern to southern boundary of the development.

The Villa Lisa area is currently serviced by the Vlakplaat Waste Water Treatment Plant currently at full capacity and cannot accommodate the new development. It is however currently being upgraded to ensure it can service all future developments that fall within its drainage area. An existing 300mm uPVC sewer pipe is currently located through the development, from the north to the south of the boundary of the development. The topography allows the runoff generated to drain towards the south eastern boundary of the development.

4.3.2 Topographical Survey

The site generally slopes from North to South, towards Northern Road on the southern boundary. A topographical survey concludes a gentle slope of approximately 1% across the site. The detailed topographical map of the area showing all the existing features is indicated in figure 4.3.

Figure 4.3 - Topographical survey (Aseda, 2017)

The survey was used for designing the stormwater and potable water network. The general direction of drainage across the site is in a southwest direction.

4.3.3 Geotechnical information

Villa Lisa Ext 4 is underlaid by chert and weathered derivatives of the Chuniespoort Group intruded by Karoo age dolerites. The near surface residual material is principally dolerite soils and dolerite outcrops on the extreme south-eastern corner of the site. Variable thicknesses of transported soils cover the site. It comprises of a silt clayey hillwash overlying a nodular ferricrete sometimes with scattered gravels. These materials overlie either chert residuum or clayey silt, residual dolerite (Geogroup, 2013).

The groundwater level was measured on site at 12m to 24m below the ground surface. The dolomite bedrock was intercepted at depths of 6m to 51m (Geogroup, 2013).

5. DATA ACQUISITION AND ANALYSIS

5.1 Overview

Urban water design utilizes the physical characteristics as well as the hydrological characteristics of a site to provide interventions that use infiltration, runoff storage, runoff conveyance, and filtration to manage urban runoff. The key physical factors that affect the function, design, and performance of WSUD are climate, geology, and soils. These will be discussed in detail below.

5.2 Weather Records

A consistent and reliable historical weather information is a dependable tool required for urban stormwater design. South Africa is a semi-arid country, with significant variation in rainfall across the county. As such it is vital that data from the nearest weather station is utilized. The nearest weather station to the case study site (weather station number 0476399 0), as illustrated in Figure 5.1 is OR Tambo International Airport weather station. The station has been active for the past 66 years; it however only provides 30 years of data for analysis. The information from this weather station was obtained from the South African Weather Services (SAWS). The annual average rainfall for this area is 750mm and the rainfall season commences in October and runs up to April. Details of the station are as follows:

Table 5.1 – Weather station details

SAWS station name	Lat	Long	Start	End	Active Years	Recording
JOHANNESBURG_INT_WO	-26.1430	28.2346	1953	current	66	5 min

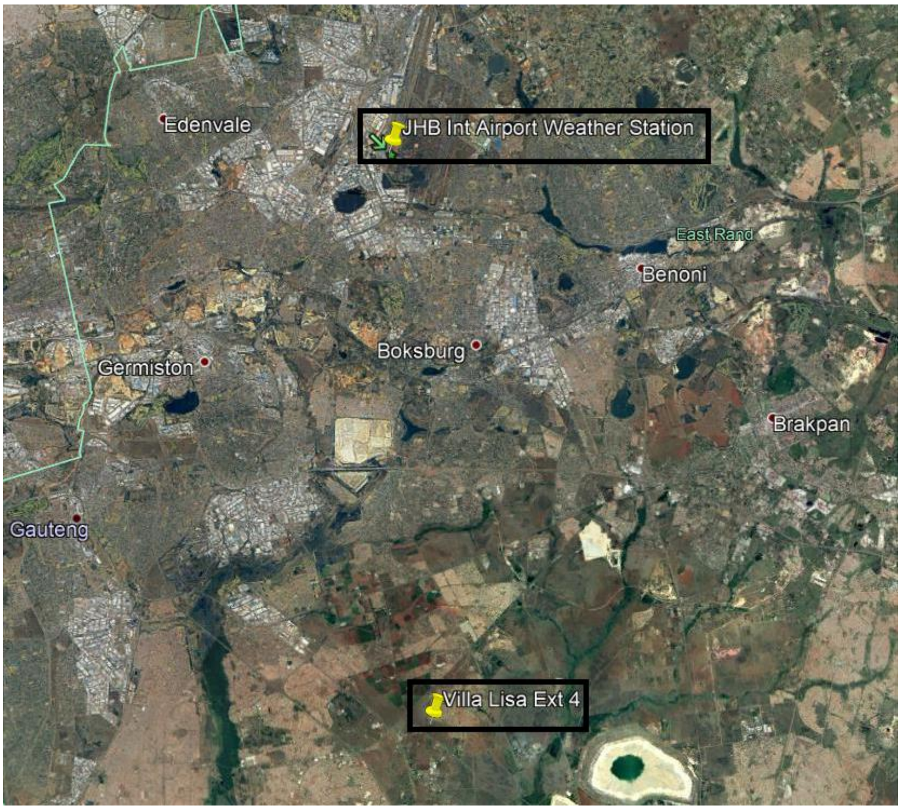


Figure 5.1 - Google Earth Image of Villa Lisa and Nominated Rainfall Station (Google Earth, 2019)

Rainfall is not stationary or constant as such it is vital that flood risk analysis accurately predicts rainfall as this affects the reliability of the design. Research shows that climate change influences storm frequency, magnitude and antecedent conditions. A study conducted by Fatti and Vogel (2011) analysed historical rainfall data (1960-2008) from the OR Tambo International Airport weather station. The study analysed 30-day totals, daily rainfall, number of rain days, and number of rain days greater than 10mm. The study concluded the following:

1. the seasonal totals suggest a general increase in summer rainfall from 1960 to 2008.
2. the average rainfall per thunderstorm event increased, with the average storm rainfall in 2008 being approximately 70% greater than that in 1960.
3. the number of “heavy storm” events, meaning the top 10% of the historic storms, has been consistent over the period 1960 to 2008.
4. It was reported that over the period, there were fewer storms with more rainfall per event, even though there was no significant change in the number or magnitude of larger events.

5. The above implies that there is a notable shift in the size of the lower order events i.e. the 2 and 5 year events but not the larger events.

It is thus important that climate change and extreme weather conditions be considered in the design of urban water systems.

5.2.1 Rainfall

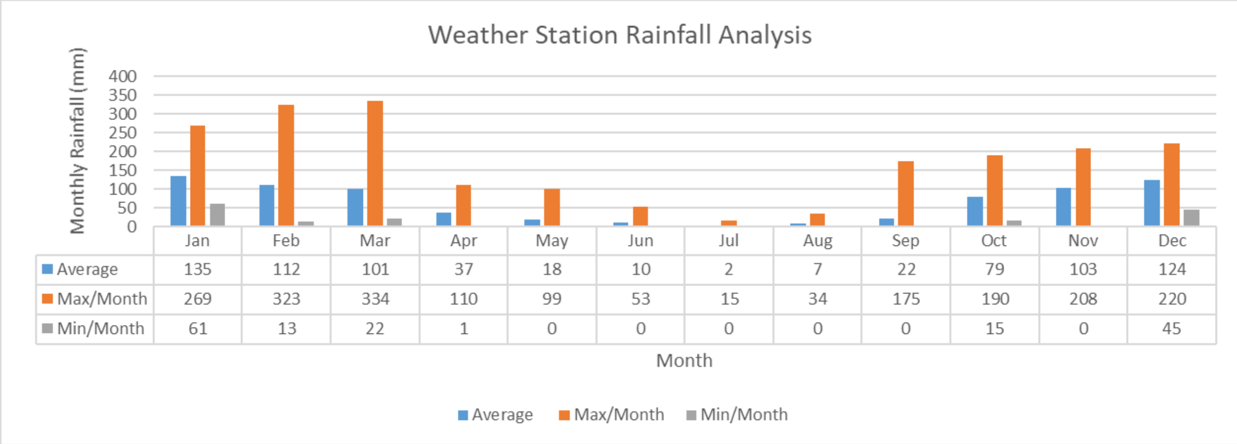
The design rainfall depth/duration/frequency was obtained from the Design Rainfall Estimator of Smithers and Schulze (2012). OR Tambo International Airport weather station records 5-minute data records and these were available for use in the analysis.

The rainy season lasts from November to April with an annual precipitation of 750mm. The resilience of infrastructure is greatly affected by rainfall, therefore, making it one of the key components of climatic conditions that cause the failure of infrastructure. December and January are the wettest months and are depicted in Table 5.2.

Table 5.2 - Precipitation Data

	Rainfall per month (mm)			**Rain days per month**		
Month	**Average**	**Max/Month**	**Min/Month**	**Average**	**Max Days**	**Min Days**
Jan	135	269	61	14.6	22	8
Feb	112	323	13	11.1	19	4
Mar	101	334	22	11.1	20	6
Apr	37	110	1	6.5	15	1
May	18	99	0	2.6	9	0
Jun	10	53	0	1.9	6	0
Jul	2	15	0	0.7	4	0
Aug	7	34	0	1.9	10	0
Sep	22	175	0	3.3	14	0
Oct	79	190	15	10.3	18	5
Nov	103	208	0	13.3	24	0
Dec	124	220	45	15.8	20	7

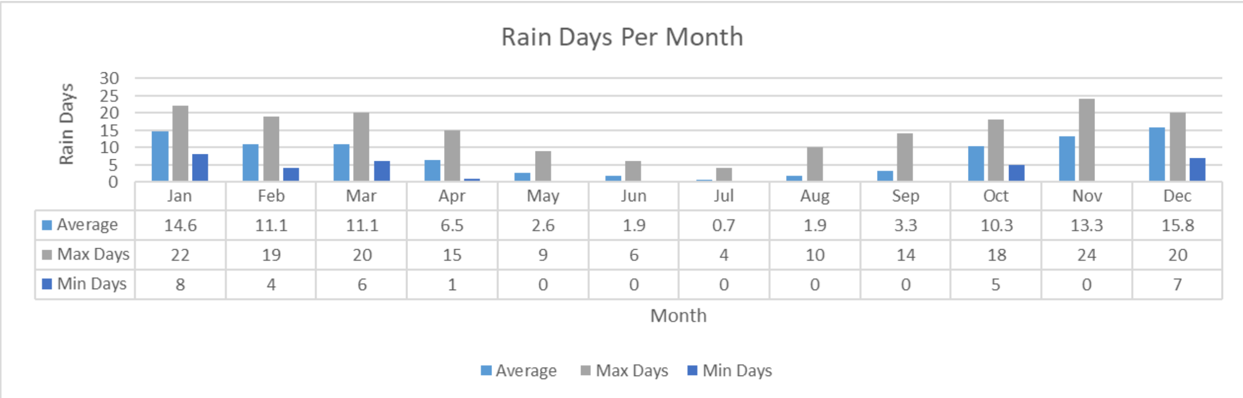
Figure 5.2 illustrates the maximum and minimum rainfall per month.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average	135	112	101	37	18	10	2	7	22	79	103	124
Max/Month	269	323	334	110	99	53	15	34	175	190	208	220
Min/Month	61	13	22	1	0	0	0	0	0	15	0	45

Figure 5.2 - Graph indicating rainfall pattern

Figure 5.3 illustrates the maximum and minimum rain days per month.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Average	14.6	11.1	11.1	6.5	2.6	1.9	0.7	1.9	3.3	10.3	13.3	15.8
Max Days	22	19	20	15	9	6	4	10	14	18	24	20
Min Days	8	4	6	1	0	0	0	0	0	5	0	7

Figure 5.3 - Graph indicating rain days per month per year

5.3 Soils & Infiltration

Pre-development runoff coefficients depend on the undisturbed soil conditions obtained from land type reports and the geotechnical report. Other influencing factors such as slope will be determined from site measurements obtained from the topographical survey.

In 1986, Rutherford & Westfall defined six biomes areas in South Africa based on bio-climatic and growth form information. This was improved by Low & Rebelo (1996), who further sub-divided the savanna biome to include the category "Thicket" which occurs predominantly in the river valleys of the eastern and south eastern coastal region of the country. Figure 5.4 indicates that the case study area is within the 'Grassland' zone.

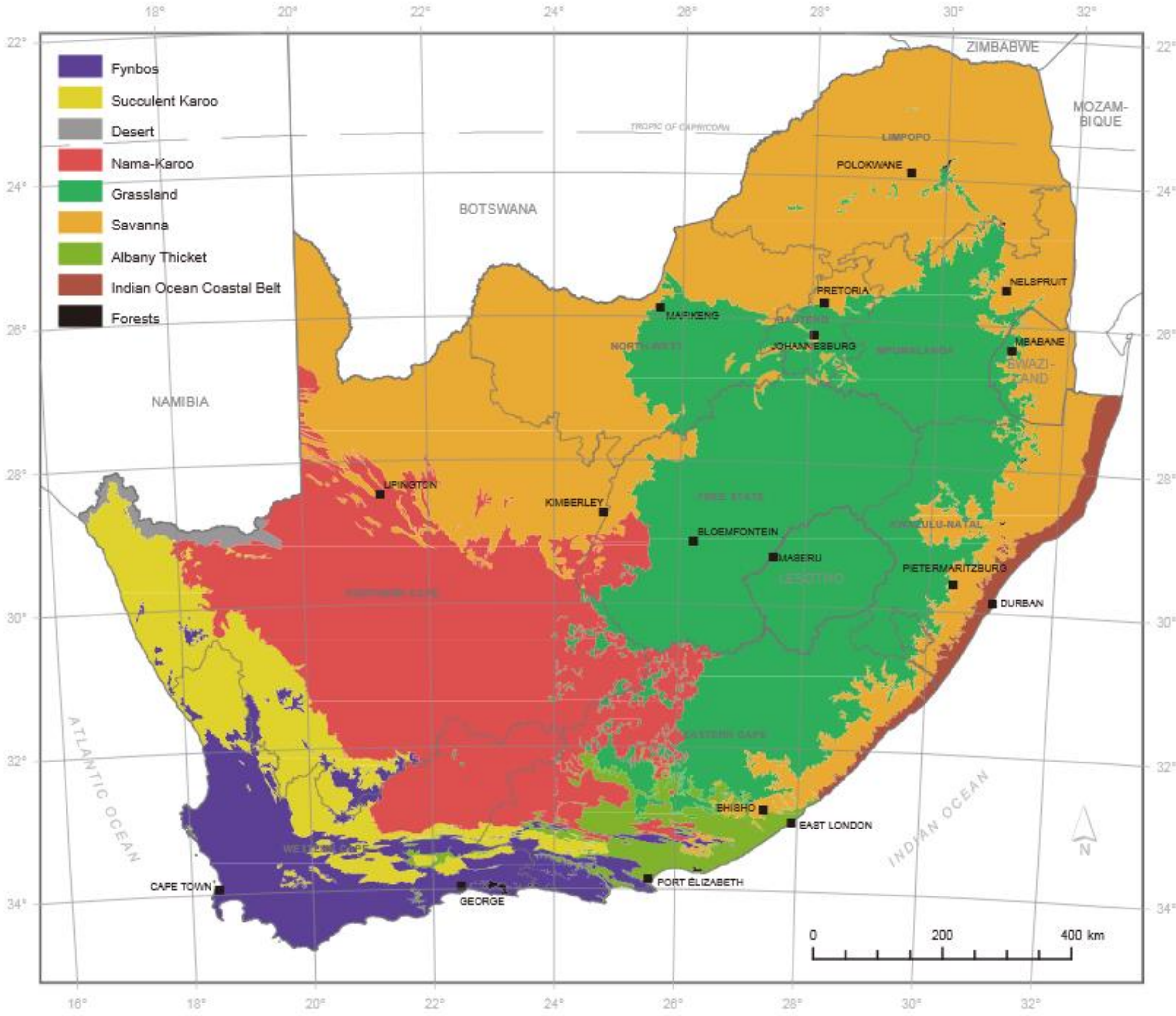


Figure 5.4 - Biomes of South Africa, Lesotho and Swaziland. (After Rutherford & Westfall, 1986 and Low & Rebelo, 1996 cited by Mucina and Rutherford, 2006)

The geotechnical investigations for the site and tests conducted provide information on specific soil conditions. This is needed for the assessment of infiltration potential, risk of groundwater contamination, subsurface stormwater flows, erosion control and sediment management of the site. Information on the local geology indicates the depth to bedrock, impervious layers, water table conditions, etc. which are important in designing infiltration and recharge systems. This information is particularly important as it influences the selection of stormwater management facilities which ensure the local groundwater is protected and that no damage is caused to the foundations of adjacent structures and services. The typical tests that need to be considered are summarised in Table 5.3.

Table 5.3 – Tests to be performed

Test	Purpose of Test
Soil profiling	SUDS planning & design.
Soil grading analysis	Sediment & erosion analysis
Bulk density	Porosity (soil water storage potential)
Specific gravity	Sediment trap efficiency
Plasticity Index	Erosion risk,
Permeability test of undisturbed sample	Infiltration design, pollution risk.
Water table (level and quality sample)	Resource risk, structural protection, recharge

5.3.1 Infiltration Methods

Infiltration is described as the process of rainfall penetrating the natural ground surface into the soil zone of the area. The two main methods utilized to model infiltration in PCSWMM are Horton's method and Green-Ampt method (Rossman, 2015).

Horton's Method *"is based on empirical observations that show that infiltration generally decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required are the maximum and minimum infiltration rates, a decay coefficient (how fast the rate decreases over time) and the time it takes a fully saturated soil to completely dry"* (Rossman, 2015). The following parameters in Table 5.4 and Table 5.5 are suggested by Green (1984) for the Horton's infiltration method in SWMM5 (Rossman, 2015).

Table 5.4 -Typical Values for the Parameters in Horton's Method (Green, 1984 cited in CoJ, 2019)

Soil Type	f_o (dry / Initial f)	f_c (equilibrium / Final f)
	mm/h	mm/h
Sandy Soil	125	15
Loam Soil	50 - 75	5 - 10
Clay Soil	5 - 25	0 - 5

Table 5.5 - Rate of Decay of Infiltration for different values of k (Green, 1984 cited by CoJ, 2019)

k			**Percent of decline of infiltration capacity towards equilibrium value f_c after 1 hour**
(sec^{-1})	**($hour^{-1}$)**	**(day^{-1})**	
0.00056	2.02	48.4	76
0.00083	2.99	71.7	95
0.00111	4.00	95.9	98
0.00139	5.00	120.1	99

Green-Ampt Method "*assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters include the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity"* (Rossman, 2015). The following parameters in Table 5.6 are suggested for the Green-Ampt infiltration method in SWMM5.

Table 5.6 - Suggested Green-Ampt Parameters (Rawls et al., 1983 cited by Rossman, 2015).

USDA Soil-Texture Class	**Hydraulic Conductivity (K_1)**	**Wetting Front Suction Head (Yf)**	**Porosity**	**Water Retained at Field Capacity**	**Water Retained at Wilting Point**
	mm/h	**mm**	**m^3/m^3**	**m^3/m^3**	**m^3/m^3**
Sand	120.40	49.02	0.437	0.062	0.024
Loamy Sand	29.97	60.96	0.437	0.150	0.047
Sandy Loam	10.92	109.22	0.453	0.190	0.085
Loamy Sand	3.30	88.90	0.463	0.232	0.116
Silt Loam	6.60	169.93	0.501	0.284	0.135
Sandy Clay Loam	1.52	219.96	0.398	0.244	0.136
Clay Loam	1.02	210.06	0.464	0.310	0.187
Silty Clay Loam	1.02	270.00	0.471	0.342	0.210
Sandy Clay	0.51	240.03	0.430	0.321	0.211
Silty Clay	0.51	290.07	0.479	0.371	0.251
Clay	0.25	320.04	0.475	0.378	0.265

5.4 Water Demand

Water consumption demand for different end users typically varies for several reasons. Domestic water demand is normally constant throughout the year while demand for users like day schools is higher on weekdays and lower on weekends and non-schooling months of the year. Shopping malls, on the other hand, experience higher demands over the weekends and the school holiday periods of the year. As such, it is vital that water demand is accurately determined.

The determination of the potable water demand was based on *The City of Ekurhuleni (COE) guidelines* (COE, 2006) and the design standards and criteria as set out in *The Neighbourhood Planning and Design Guide* (DHS and CSIR, 2019). The potable water network was designed to accommodate peak hour domestic demands as well as firefighting requirements as stated in SANS 10090 (SANS, 2003).

5.4.1 Non-potable demand

The probable non-potable uses of RWH systems include toilet flushing, air conditioning and irrigation (Jacobs et al., 2017). This demand is distinct and varies at different timescales across the different end users. As such, it is important to consider the effects of this variability in the design of the RWH system.

The results of the 2017 report by Jacobs et al. conclude that the largest 4 end-uses are toilet, bath-shower and washing machine and they contribute 81.5% to total indoor water use. This validates earlier reports by other researchers. The study further recommends that Water Demand Management (WDM) measures be targeted towards the largest 4 end-uses i.e. toilet, shower, bath and washing machine for fairly smaller properties. For larger properties, it is recommended that WDM be targeted towards reducing garden irrigation (Jacobs et al., 2017).

A study by Basson et al. (2018) on water intelligence report by GreenCape provides a figure adopted from the Environmental Protection Agency (EPA), indicating the typical end uses of water in various types of commercial and institutional facilities (Basson et al., 2018).

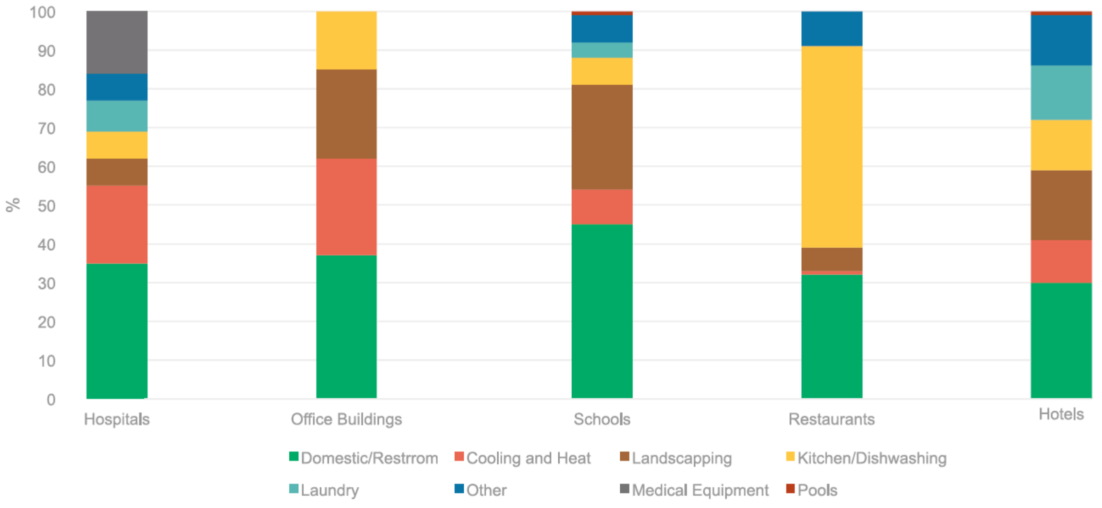


Figure 5.5 - Typical end uses of water in various types of end users (Basson et al., 2018)

Below is a breakdown of some of the specific users (USEPA, 2012c).

Office Building

The largest consumers of water in office buildings are restrooms, heating and cooling, and landscaping (Figure 5.6).

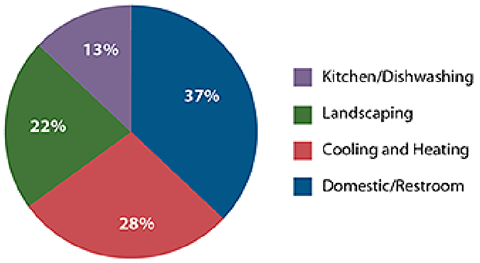


Figure 5.6 – End uses of water in office building

Hospitals

The largest consumers of water in hospitals are cooling equipment, plumbing fixtures, landscaping, and medical process rinses (Figure 5.7).

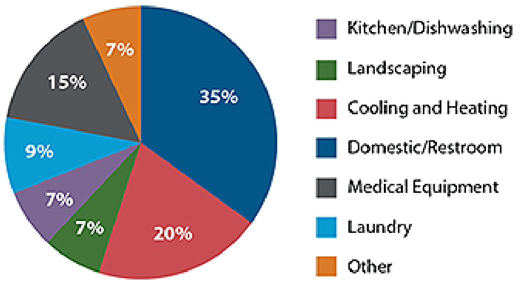


Figure 5.7 - End uses of water in hospitals

<u>Hotels</u>

The largest consumers of water in hotels are restrooms, laundry operations, landscaping, and kitchens (Figure 5.8).

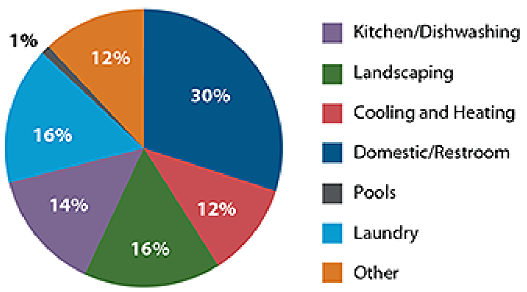


Figure 5.8 - End uses of water in hotels

<u>Educational Facilities</u>

The largest consumers of water in educational facilities are restrooms, landscaping, heating and cooling, and cafeteria kitchens (Figure 5.9).

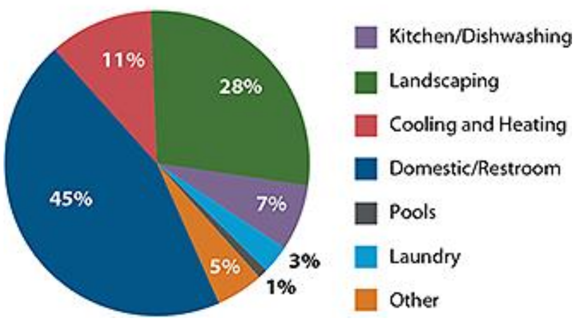


Figure 5.9 - End uses of water in schools

5.5 Stormwater runoff

The Rational Method as detailed in the SANRAL Drainage Manual will be used to determine runoff volumes. PCSWMM, a computer based model that determines runoff based on the physical characteristics of the drainage area, utilizes continuous records of measured rainfall data, considers the effects of soil moisture conditions and design storm types which will also be used to determine runoff. Results from the Rational Method will be compared to the computed results.

Pre-development condition *“means the condition of any property or portion of a property as it existed in its unaltered natural state prior to any development on that property”* (CoJ, 2019).

Post-development condition *“means the condition of any property after the conclusion of development”* (CoJ, 2019).

The runoff volumes of the pre and post development conditions will be compared.

6. CONVENTIONAL DESIGN VERSUS WSUD

6.1 Assumptions

Listed below are the assumptions made during the design of this development.

The conventional design scenario assumes the following:

- the conventional design of stormwater drainage management systems where runoff is conveyed in a system of pits and pipes.
- the conventional design of water supply systems where municipal water is conveyed in a system of pipes to consumers.

The best practice WSUD design scenario will assume:

- all the conventional design methods i.e.:
  - typical stormwater drainage management systems where runoff is conveyed in a system of pits and pipes and
  - typical water supply systems where municipal water is conveyed in a system of pipes to consumers.
- the use of bio-retention systems to address runoff quality, quantity and frequent flow objectives
- the use of detention storage to address receiving waterbodies stability and erosion objectives
- the use of rainwater tanks to address water consumption objectives

6.2 Conventional Design

6.2.1 Potable Water

<u>Demand Estimation</u>

As earlier indicated, the determination of the potable water demand was based on The City of Ekurhuleni (COE, 2006) guidelines and DHS and CSIR (2019) guidelines. The results of the Annual Average Daily Demand (AADD) and peak flows are presented in Tables 6.1:

Table 6.1 - Domestic water demand calculations

					COE (2006) guideline				DHS and CSIR (2019)			
Zoning	**Area (Ha)**	**No. of Erven**	**No. of Units**	**Unit**	**AADD**	**Total AADD (Kl/day)**	**Peak Factor**	**Peak Flow (Kl/day)**	**AADD**	**Total AADD (Kl/day)**	**Peak Factor**	**Peak Flow (Kl/day)**
Residential 2	10.46	1420	1420	Kl per erf	1	1420.00	2.4	3408.00	0.6	852.00	2.4	2044.80
Residential 2 (Duplex Units)	0.51	41	41	Kl per unit	1	41.00	2.4	98.40	0.6	24.60	2.4	59.04
Residential 4	18.17	17	2174	Kl per erf	0.6	1304.40	2.4	3130.56	0.4	869.60	2.4	2087.04
Church	0.12	1		Kl per $100m^2$	0.1	1.18	-	-	0.3	3.55	-	-
Crèche	0.39	1		Kl per hectare	1.5	0.59	-	-	12	2.34	-	-
Primary School	2.70	1		Kl per hectare	1.5	4.05	-	-	12	16.19	-	-
Institutional	0.94	2		Kl per $100m^2$	1.5	140.34	-	-	20	748.48	-	-
Business 2	2.44	2		Kl per $100m^2$	0.8	195.34	-	-	21	2051.11	-	-
Municipal	1.28	11		Kl per $100m^2$	1	127.55	-	-	20	1020.40	-	-
Public Open Spaces	13.92	76		Kl per hectare	-	11.60	-	-		0.00	-	-
Streets and Roads	21.72											
Total	**72.63**	1574	3635			**3246.05**		**6636.96**		**5588.28**		**4190.88**

Conveyance infrastructure are designed such that they have sufficient capacity for peak demand conditions and fire-flow requirements (DHS and CSIR, 2019). The estimation of the total peak flow of this development was calculated by summing only the values for the residential zones and applying the peak factor as specified and not by combining all the AADD values for all the zones.

Fire Demand

	DHS and CSIR (2019) guideline	
Description	**Value**	**Comments**
Fire-risk categories		Low-risk – group 3
Fire Demand(l/min)	350	From Table 9.20
Number of fire Hydrants	20	located at convenient points in the area
Fire Flow (l/hour)	420000	No. of Hydrants x Fire demand
Duration(hrs)	1	Table 9.19
Fire flow (l/day)	420000	Assuming that each hydrant is used once a day for four hours
Fire flow (l/s)	4.86	
Fire flow (l/s) / hydrant	0.243	

Public Open Spaces

	DHS and CSIR (2019) guideline	
Description	**Value**	**Comments**
Irrigation demand(l/ha/day)	10000	
Area (Ha) of irrigation spaces	13.92	
Irrigation demand (l/day)	139216	Area (Ha) of irrigation spaces x irrigation demand
Duration(hrs)	2	Table 9.19
Irrigation demand (l/day)	11601.33	Assuming that irrigation is done 2 hours each day
Irrigation demand (l/s)	0.13	

The use of the city's guideline (COE, 2006) resulted in an annual average daily demand of 37.57 ℓ/s with a corresponding peak flow of 76.82 ℓ/s. The DHS and CSIR (2019) guidelines resulted in an annual average daily demand of 64.68 ℓ/s and a corresponding peak flow of 48.51 ℓ/s. Since the peak flow is less than AADD, the AAAD values of the other land users were added to determine a peak flow of 92.97 ℓ/s though it is unlikely for the other users to peak at the same time as the residential users. Due to this disparity, the results as determined using the COE (2006) guideline was used in the design of the conveyance infrastructure.

Network Design

Following the determination of the peak design flows, WaterMate was used to design the pressurized water reticulation network. The results are presented in a layout drawing attached in Appendix A2.

6.2.2 Stormwater runoff estimation

The new development is a mixed use development consisting of residential, business, municipal and institutional units. The conventional stormwater drainage system design includes curb inlets, manholes, and pipes capable of handling the local 10-year design storm. The network drains towards the outfall position. The pipes are sized such that there is an increase in capacity at the downstream. Pipes closer to the outfall are larger than pipes upstream.

The use of the Rational Method to determine runoff

The results of the use of the Rational Method in determining the pre- and post-development runoffs are presented in Table 6.2. The detailed calculations are attached in Appendix A3.

Table 6.2 - Rational Method design runoff results

Rational Method Design Runoff Estimation		
Return Period	**Pre-development Peak Discharge**	**Post-development Peak Discharge**
years	m^3/s	m^3/s
2	0.557	7.712
5	0.788	10.021
10	1.154	12.806
20	1.642	16.098
50	2.464	20.962
100	3.722	25.965

Computed results of the conventional stormwater drainage system

PCSWMM uses simple diagnostic tools to confirm the validity of the model results and to optimize the design. The analysis is such that the runoff and routing continuity errors are less than 1%. In the model, the total precipitation should equal the losses due to infiltration, evaporation, surface runoff and final storage on the sub-catchment. The runoff coefficient also indicates the ratio of total runoff to the total precipitation. The input parameters are listed in Table 6.3.

Table 6.3 - PCSWMM input parameters

Model Parameter Description	Units	Value	
		Impervious	Pervious
Flow length	m	2050	
Imperviousness	%	0	100
Slope	%	0.1	
Manning's n		0.01	0.1
Depression Storage	mm	1.27	2.54
Green-Ampt Suction Head	mm	n/a	290.07
Green-Ampt Conductivity	mm/h	n/a	0.51
Green-Ampt Initial Moisture Deficit	fraction	n/a	0.228

OR Tambo International Airport weather station records 5-minute data records which were used in the analysis to determine the trends of the pre-development scenarios. Rain data for the period of 19 October 1994 to 28 August 2016 supplied by SAWS was modelled. There was a total of 8728 days modelled. Figure 6.1 shows rainfall and modeled runoff during this period.

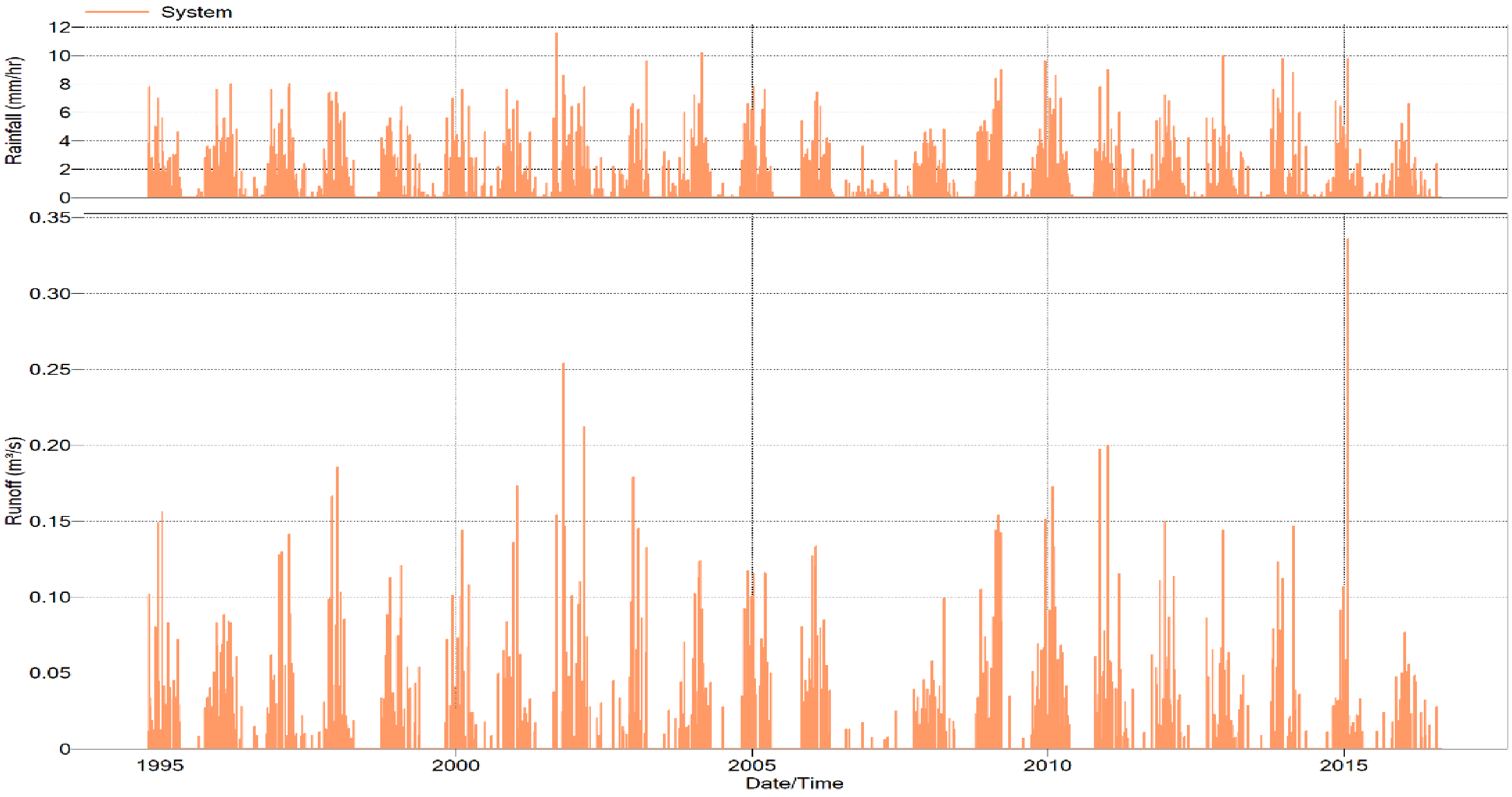


Figure 6.1 - Continuous rainfall vs runoff analysis

From 9/14/1993 9:25:15 PM to 10/1/2017 5:49:45 AM (8782.35 days)

	System
Maximum Rainfall (mm/hr)	11.6
Minimum Rainfall (mm/hr)	0
Mean Rainfall (mm/hr)	0.006654
Duration of Exceedances (h)	191600
Duration of Deficits (h)	189100
Number of Exceedances	1
Number of Deficits	11750
Volume of Exceedances (mm)	1275
Volume of Deficits (mm)	0
Total Rainfall (mm)	1275

Figure 6.2 - Model results of the continuous rainfall analysis.

The highest rainfall intensity (11.6 mm/hr) occurred on the 12th of September 2001 and lasted for 1 hour. The continuous pre-development rainfall - runoff model in Figure 6.1 assumes that the future trend will be the same as the past and this is useful for long-term purposes such as the estimation of design discharges and impact of climate and land use change. Event based rainfall - runoff models are applied to infrastructure designs, where a storm design is required to achieve the design hydrograph, generally catering for maximum and extreme events (Sordo-Ward et al., 2016). In this case study, the WSUD design was undertaken using event-based rainfall modelling to match the conventional design.

Figure 6.3 indicates the flow at the outfall versus the precipitation during this event. It can be noticed that the peak runoff flow falls shortly after the rainfall peak.

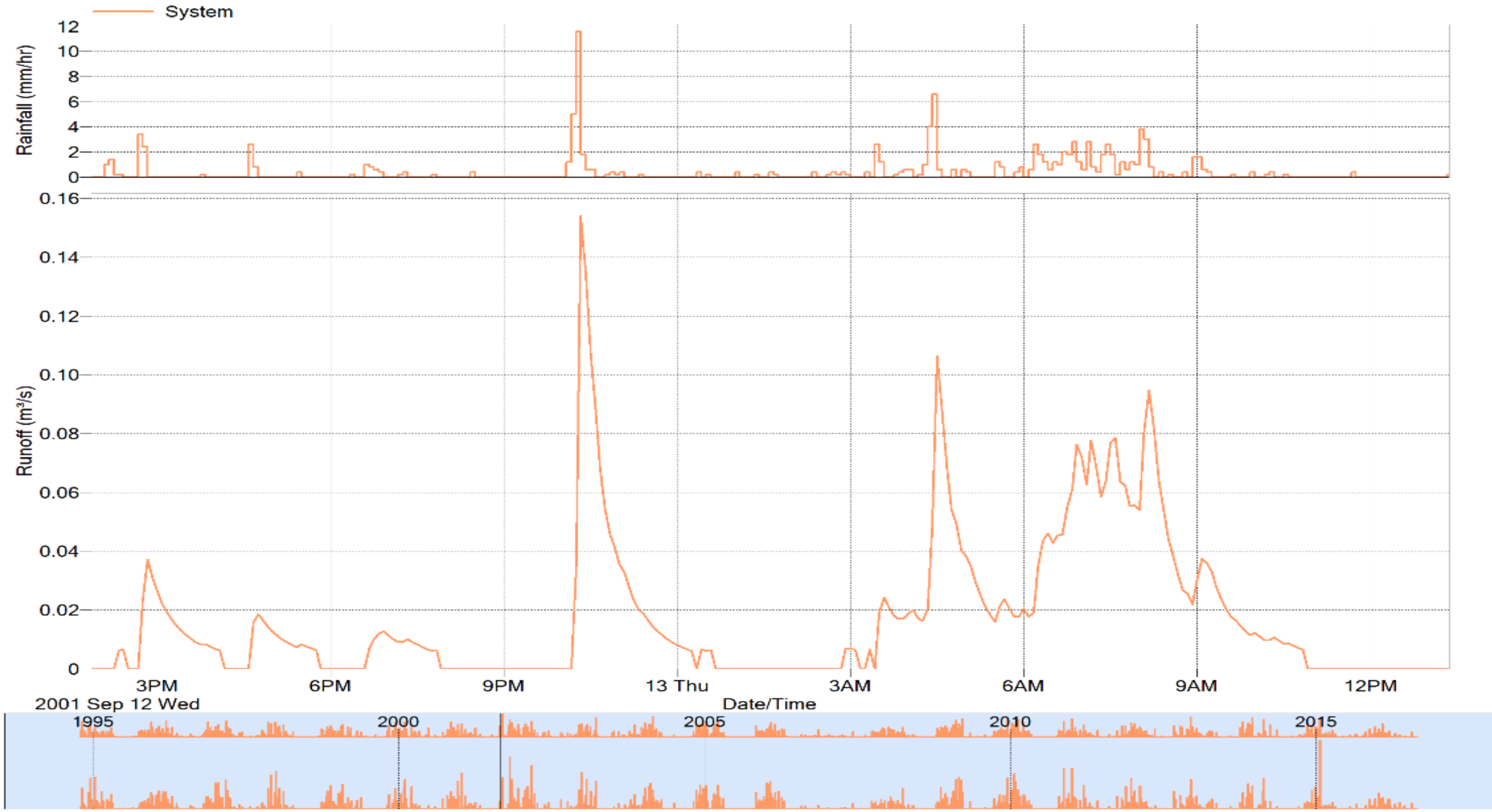


Figure 6.3 - Highest rainfall event versus runoff generated

Due to the size and complexity of the system, and the requirement for SuDS, it is important to understand the performance of the system under natural conditions. Therefore, the need for the continuous simulation above. It allows the designer to check the performance of the system over several events, storms and seasons. It also allows the designer to make an informed judgement on the effects of climate change and antecedent rainfall.

The system was further evaluated using multiple return period design storms. This section aims to illustrate how to evaluate the impact of runoff in response to multiple design storm events. The development was simulated using design storms for various Recurrence Intervals. Input for this simulation (i.e. rainfall depth and Recurrence Interval) was obtained for an SCS Type 3 from the Design Rainfall Estimator of Storm from Smithers and Schulze (2012) and pre- and conventional post- development peak runoff calculated (Summarized in table Table 6.4). It is important that stormwater infrastructure design consider predevelopment hydrology and drainage patterns in order to preserve the natural runoff response characteristics of the site. As such, the system will be evaluated against multiple design storms. The results are attached in Appendix A4

Urban development changes the natural hydrology of an area (as illustrated in Figure 6.4) by altering the volume, frequency, duration, timing and distribution of runoff. Buildings and surfaced roads reduce opportunities for natural infiltration. As a result, these impervious areas generate increased runoff and pollutant loads discharged to receiving water bodies.

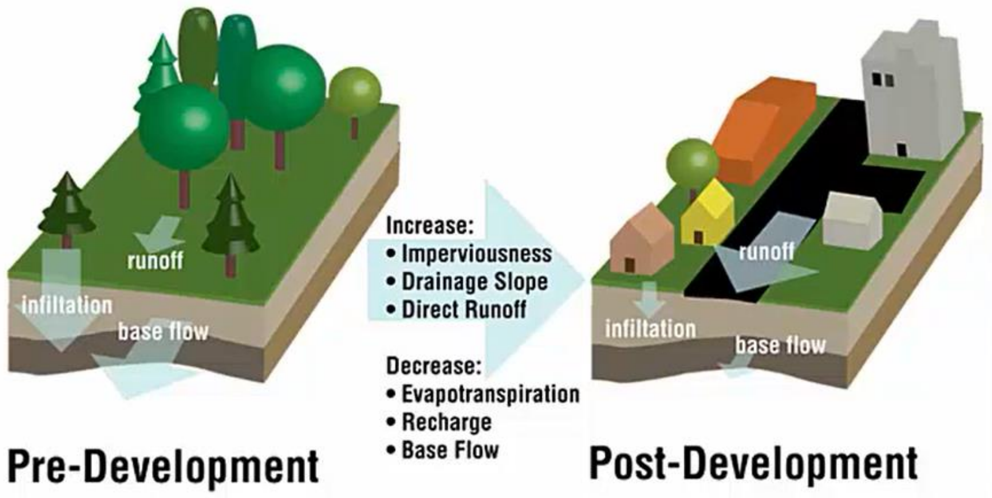


Figure 6.4 - Pre- versus Post- development urban runoff (CHI, 2019)

In this section, 6 different design storm events were used to access runoff quantity under pre- and post- development conditions. Figure 6.5 illustrates the comparisons.

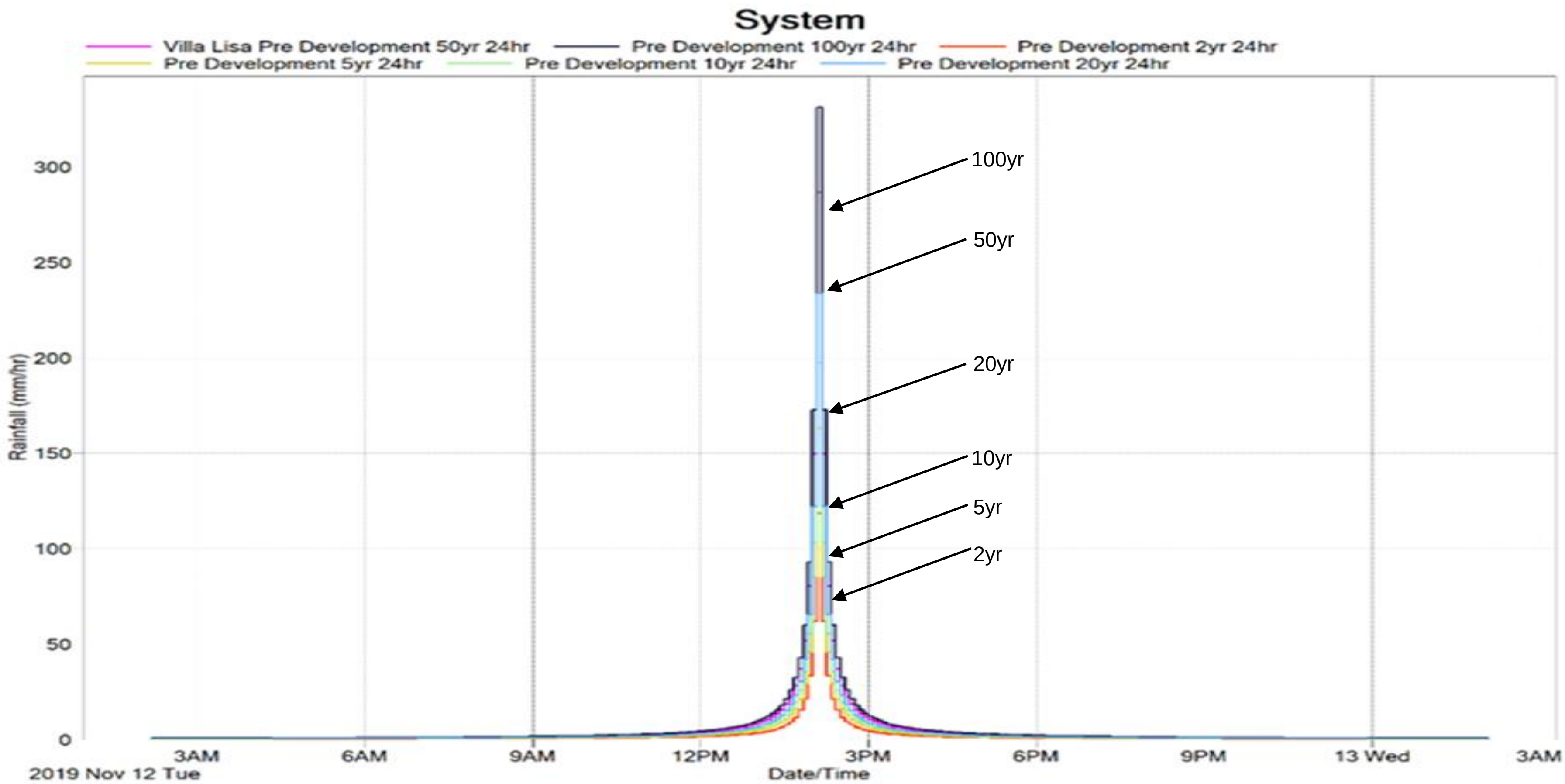


Figure 6.5 - Comparison of all recurrence interval Rainfall

Illustrated Figure 6.6 is the corresponding runoff generated for the different event rainfall in the pre-development scenario. The peak runoff increases as the event rainfall increases.

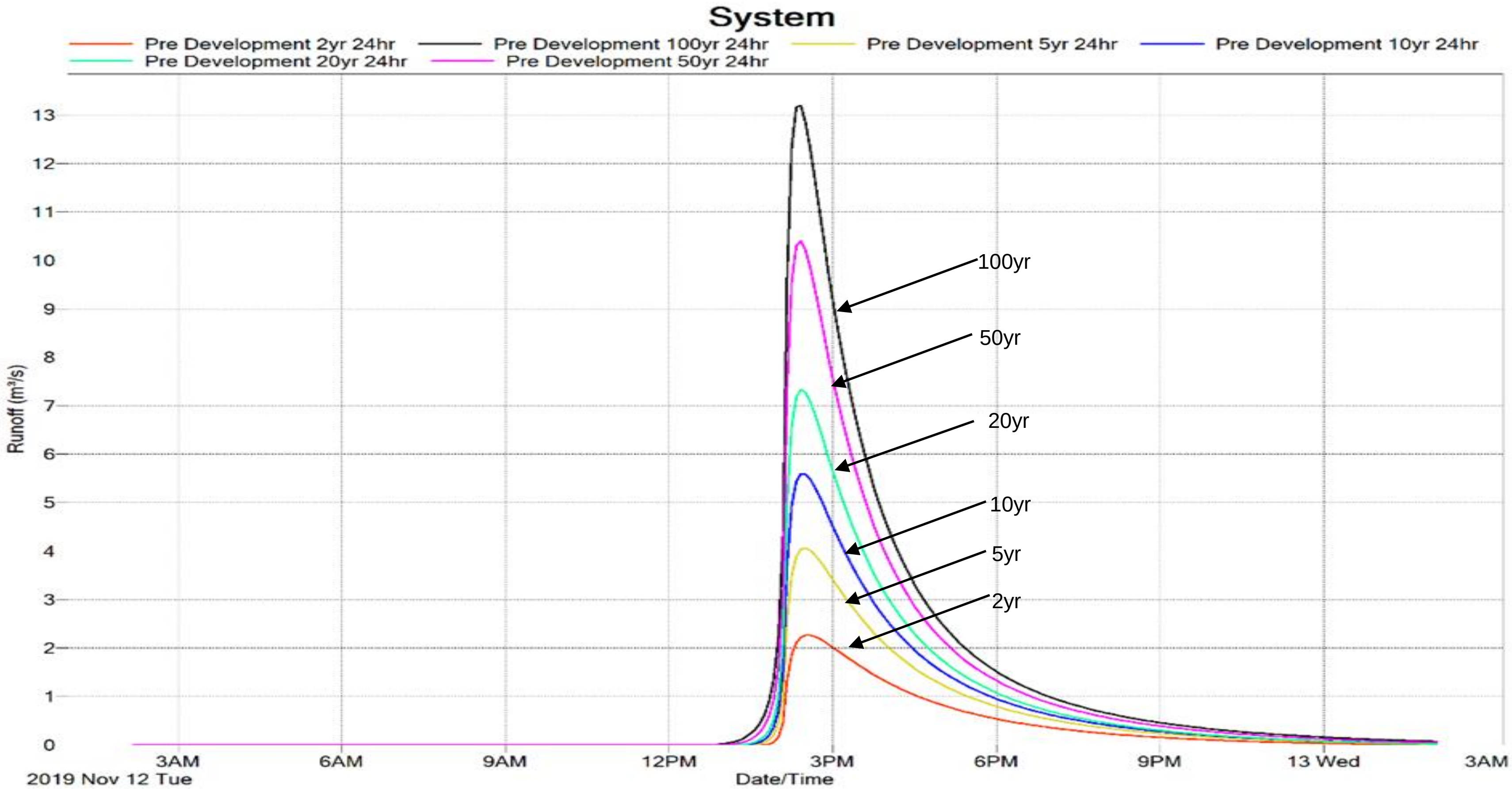


Figure 6.6 - Pre-development runoff generated for the different event rainfall

For example, a 158.8mm rainfall event is referred to as the local 100 years return period event which has an expected recurrence interval of 100 years or a 1% chance of occurring in any given year. PCSWMM design storm scenario creator was used to create a model for each return period event. Scenario results were then plotted together so that the performance of the model during different design storms and development conditions can be compared.

	System Pre Development 100yr 24hr	System Pre Development 10yr 24hr	System Pre Development 20yr 24hr	System Pre Development 2yr 24hr	System Pre Development 50yr 24hr	System Pre Development 5yr 24hr
Maximum Runoff (m^3/s)	13.19	5.595	7.335	2.261	10.41	4.061
Minimum Runoff (m^3/s)	0	0	0	0	0	0
Mean Runoff (m^3/s)	1.108	0.5637	0.6853	0.2669	0.9141	0.4379
Duration of Exceedances (h)	23.92	23.92	23.92	23.92	23.92	23.92
Duration of Deficits (h)	10.67	11.25	11.17	12.25	10.92	11.42
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	1	1	1	2	1	1
Volume of Exceedances (m^3)	95420	48530	59000	22980	78690	37700
Volume of Deficits (m^3)	0	0	0	0	0	0
Total Runoff (m^3)	95430	48530	59010	22980	78700	37710

Figure 6.7 - Model results of pre-development runoff generated for the different event rainfall

Illustrated in Figure 6.8 is the runoff generated for the different event rainfalls in the post-development scenario.

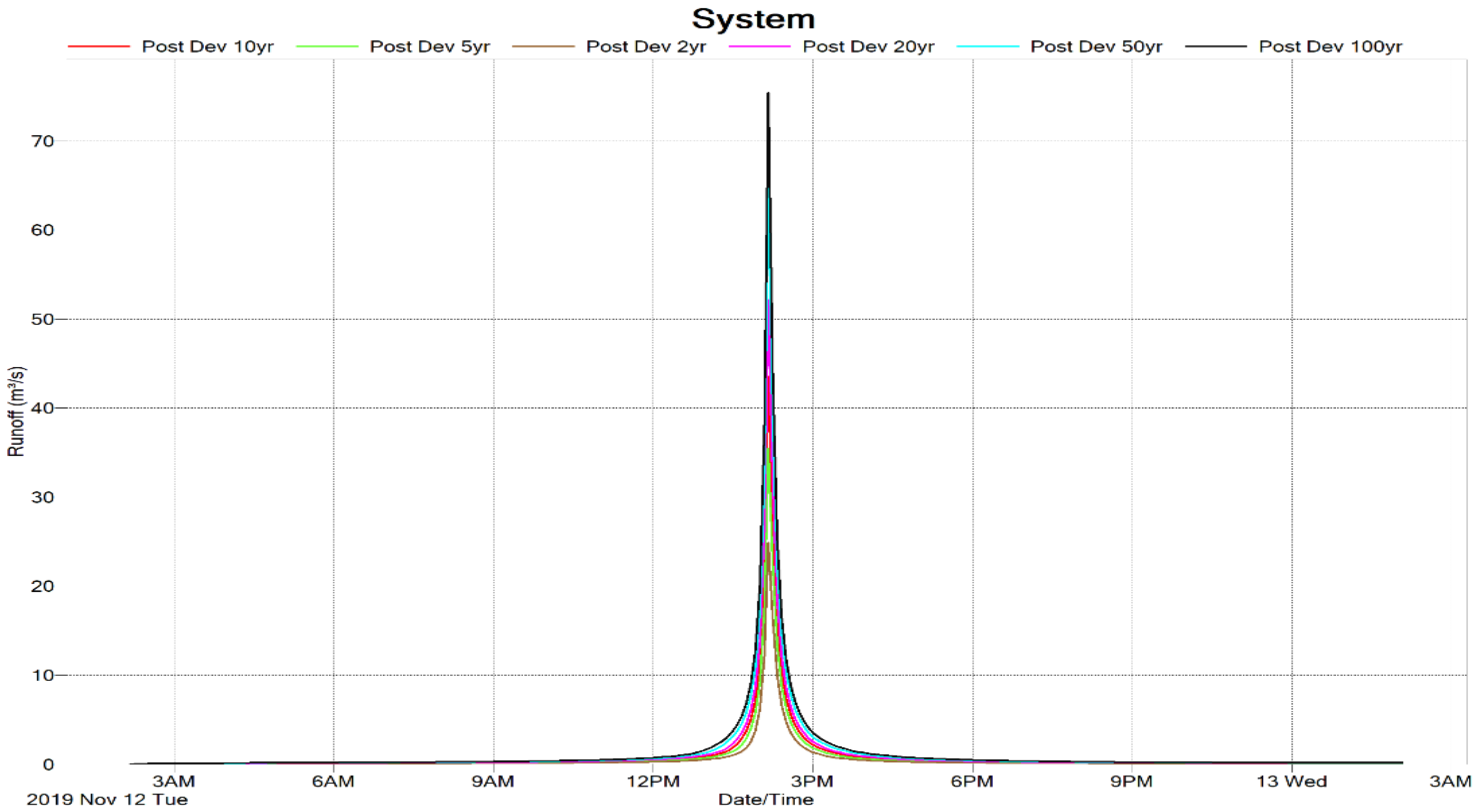


Figure 6.8 – Post-development runoff generated for the different event rainfalls

In the pre-development 100-year scenario, the total runoff value is less than the post development 100-year scenario even though the rainfall value is the same. The shape of the hydrograph is also very different as the post development scenario has a much larger and sharper peak and larger overall duration. This trend occurs in all the design storms from the 2-year to the 100-year event.

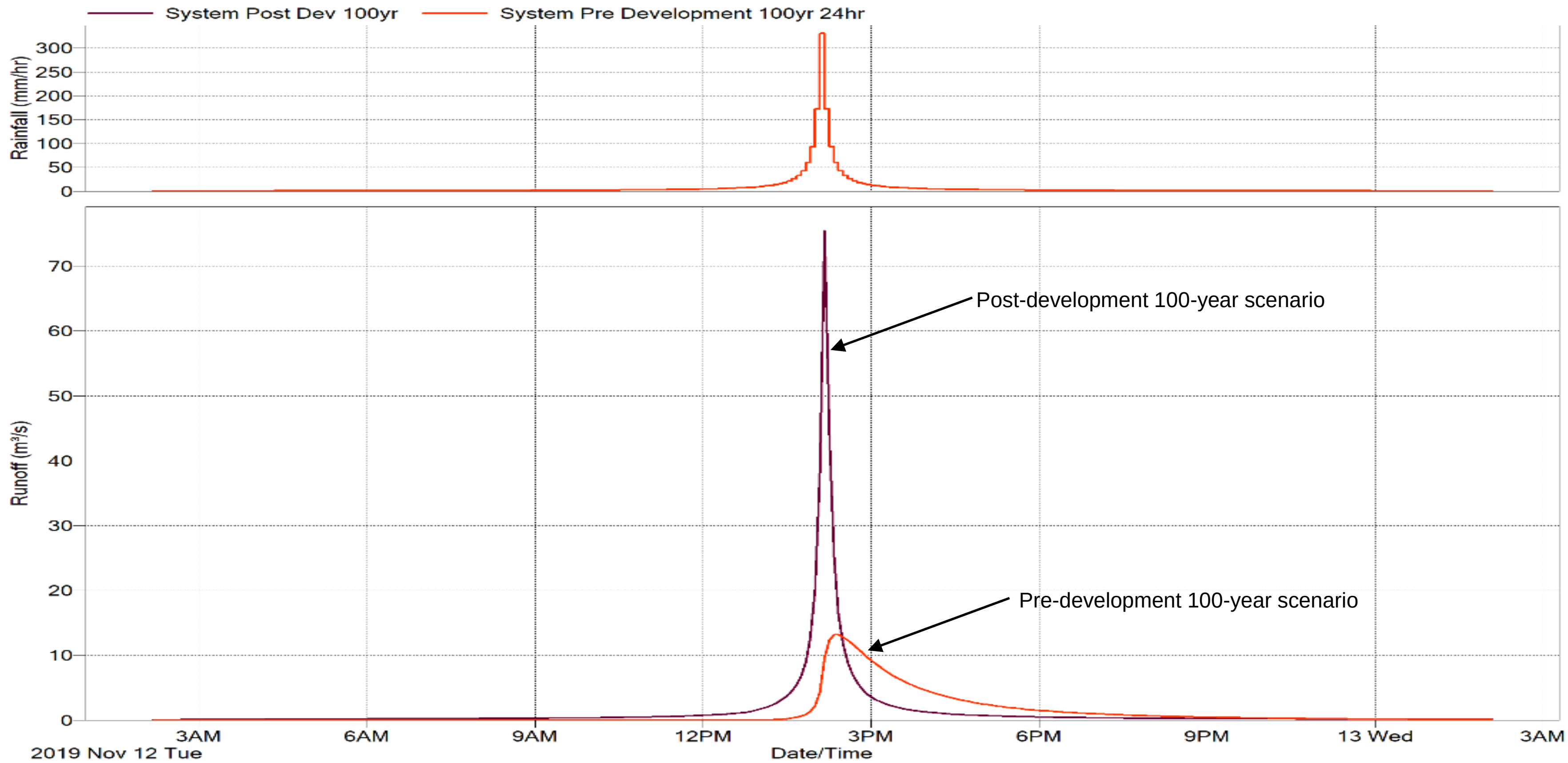


Figure 6.9 – Pre- versus post- development 100-year scenario

The model results are illustrated in Figure 6.10. A maximum runoff of 13.19 m^3/s is observed in the pre-developemnt scenario as compared to 75.41m^3/s in the post-development scenario. This represents an increas of 571.1% in runoff due to the development.

Objective functions for Runoff (m^3/s)

From 11/12/2019 12:58:15 AM to 11/13/2019 3:16:45 AM (26.31 hours)

	System Post Dev 100yr	System Pre Development 100yr 24hr
Maximum Runoff (m^3/s)	75.41	13.19
Minimum Runoff (m^3/s)	0	0
Mean Runoff (m^3/s)	1.484	1.108
Duration of Exceedances (h)	23.92	23.92
Duration of Deficits (h)	0	10.67
Number of Exceedances	1	1
Number of Deficits	1	1
Volume of Exceedances (m^3)	127700	95420
Volume of Deficits (m^3)	0	0
Total Runoff (m^3)	127800	95430

Figure 6.10 - Model results of Pre- versus post- development 100-year scenario

The runoff hydrograph provides a graphical presentation of instantaneous runoff against time. The shape of hydrographs presented is dependent on the climatic factors (such as rainfall type, intensity, duration etc.) and physiographic factors (basin characteristics such as shape, size, roughness, slope, storage characteristic etc.). The change in shape of the recession limb is caused by the changes in pre-development and post-development physiographic factors.

The losses from infiltration and sub-catchment storage are also different between the pre- and post- development scenarios. Since the pre-development scenario has more pervious area, more infiltration loss is observed. A 21.2% decrease in infiltration losses were determined. This is illustrated in Figure 6.11.

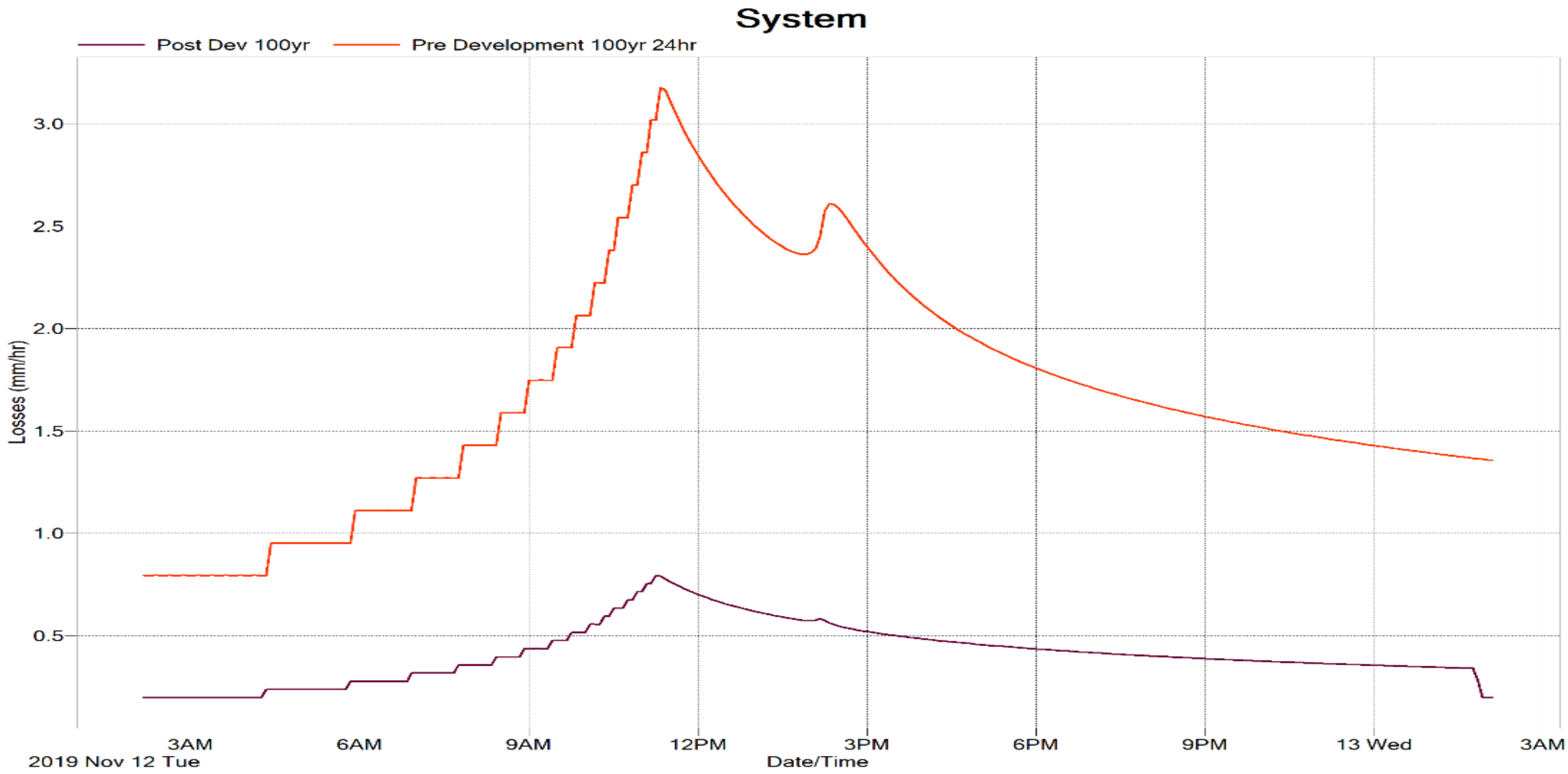


Figure 6.11 - Pre- and post- development infiltration losses.

The results of this section are summarized in Table 6.4.

Table 6.4 - Computed Pre and post development runoff results.

SCS Type 3 Storm				
Recurrence Interval	**Probability of occurrence in one year**	**Rainfall (mm)**	**Pre-development Peak Discharge**	**Post-development Peak Discharge**
years	**%**	**mm**	**m^3/s**	**m^3/s**
2	50	56.8	2.261	24.91
5	20	78.3	4.061	35.49
10	10	94.7	5.595	43.58
20	4	112.1	7.335	52.17
50	2	137.5	10.41	64.8
100	1	158.8	13.19	75.41

The target Recurrence Interval for the development is the 10-year design storm as required by DHS and CSIR (2019) and agreed to by the municipality, taking into consideration the development characteristics.

Computed versus Rational Method runoff estimation

The Rational Method as detailed in the SANRAL Drainage Manual was used to determine runoff volumes. PCSWMM, was also used to determine runoff. The results from the Rational Method is compared in Figure 6.12 to that of the computed results.

In both the pre- and post- development conditions, the computed results are higher than that of the rational method. According to the recently released draft stormwater design manual for the City of Johannesburg, the Rational Method is not recommended for the calculation of the effectiveness of stormwater interventions because of the following reasons (CoJ, 2019):

- It is required that stormwater interventions be capable of controlling runoff from storms with the same recurrence interval, but different durations. The value of the runoff coefficient varies with antecedent moisture conditions the rational method is not able to determine this variation. Software such as SWMM and PCSWMM are able to compute this.
- There is little to support the fundamental assumptions of the rational method that the rainfall is of uniform intensity and duration exactly equal to the time of the concentration of the drainage system.

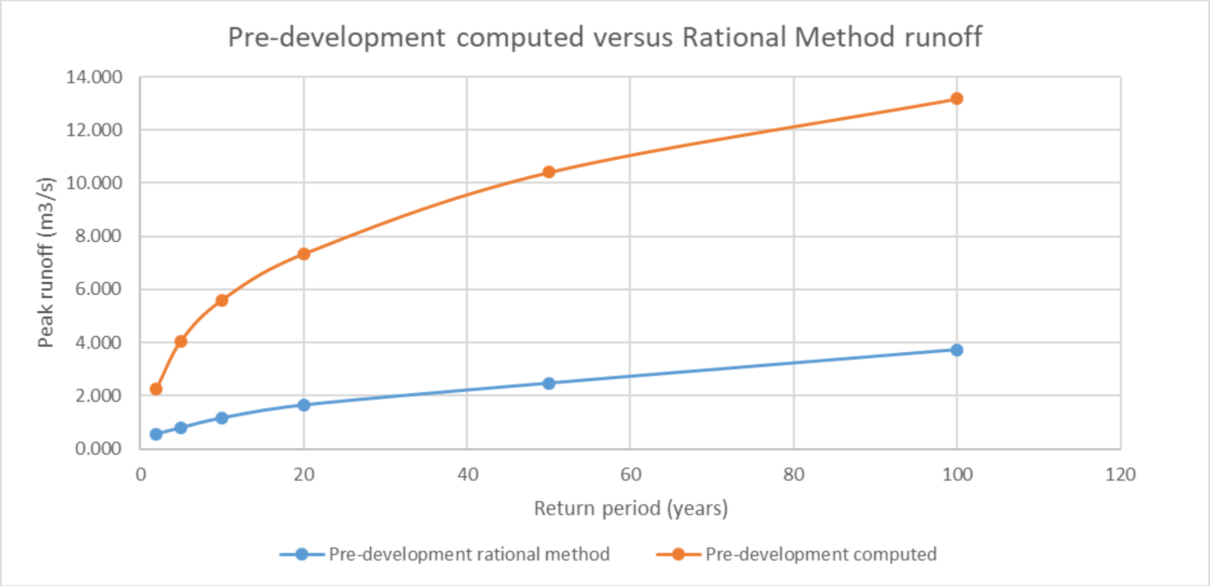


Figure 6.12 - Computed vs Rational Method pre-development results

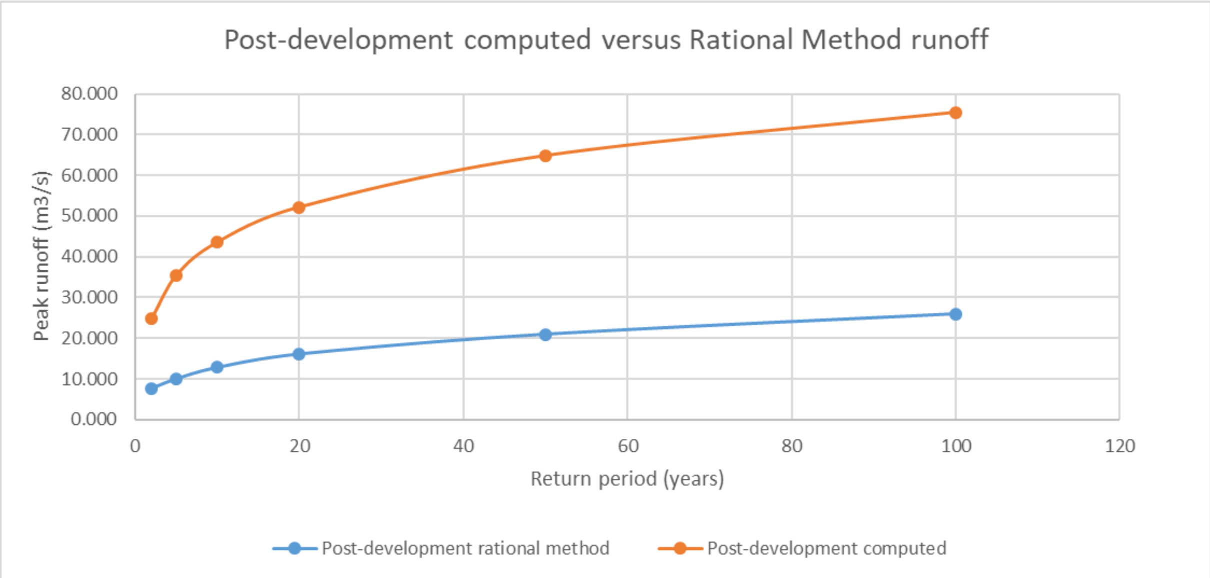


Figure 6.13 - Computed vs Rational Method post-development results

<u>Network Design</u>

The resulting stormwater network is presented in a layout drawing attached in Appendix A6.

6.3 WSUD

6.3.1 Rainwater Harvesting

Non-potable water demand

The use of rainwater in domestic households for non-potable end-uses can contribute to reducing the demand on the municipal water distribution system. Prior to the determination of the RWH tank size, it is vital that the non-potable demand is accurately determined. The non-potable uses identified in this development with their corresponding demand estimation methods are listed in Table 6.5.

Table 6.5 - Non-potable water demand estimation

<table>
<tr><th colspan="3">Non-potable parameters</th></tr>
<tr><th colspan="2">Parameter</th><th>Source</th></tr>
<tr><td rowspan="4">Residential 2 and 4</td><td>Toilet = 3 events/c/d
Volume per event = 9ℓ/event</td><td>There exists one standard toilet in each household. According to Jacobs et al (2017) one flush event is equivalent to 9 ℓ.</td></tr>
<tr><td>Laundry = 0.143 events/c/d
Volume per event = 62ℓ/event</td><td>Rudin (2008) cited by O'Brien (2014) indicates the laundry without a washing machine demand at 62 ℓ per wash. Jacobs et al (2017) indicates washing machine demand as 80 ℓ/event.</td></tr>
<tr><td>Household Cleaning = 2 events/c/d
Volume per event = 6ℓ/event</td><td>Jacobs et al (2017) indicates a typical volume for kitchen sink events of 6ℓ/event which is adapted for this study, as it is the most realistic value.</td></tr>
<tr><td>Gardening = 1 events/c/d
Res 2 Volume per event = 5ℓ/event
Res 4 Volume per event = 1.092ℓ/day/m^2</td><td>Res 2: Jacobs & Haarhoff (2004) cited by O'Brien (2014) indicates an outside tap demand of 5 ℓ/event, which can be adopted for garden use. The gardens for the Res 2's are very small, making Jacobs et al (2017) estimate of 82 ℓ/day too large an estimate as it applies to irrigated areas of $75m^{2}$.
Res 4: The gardens for the Res 4's are on average $640m^2$, making Jacobs et al 2017 estimate of 573.3 ℓ/day/stand for irrigated areas of $525m^2$ a realistic estimate.</td></tr>
<tr><td colspan="2">Non-potable Church Demand
Landscaping = 28%
Cooling & Heating = 11%
Restrooms = 45%</td><td>USEPA (2012c), WaterSense at work: best management practices for commercial and institutional facilities, provides typical end uses for commercial and institutional facilities.</td></tr>
<tr><td colspan="2">Non-potable Crèche Demand
Landscaping = 28%
Cooling & Heating = 11%
Restrooms = 45%</td><td>USEPA (2012c), WaterSense at work: best management practices for commercial and institutional facilities, provides typical end uses for commercial and institutional facilities.</td></tr>
<tr><td colspan="2">Non-potable Primary School Demand Landscaping = 28%
Cooling & Heating = 11%
Restrooms = 45%</td><td>The largest uses of non-potable water in educational facilities are restrooms, landscaping, heating and cooling.</td></tr>
<tr><td colspan="2">Non-potable Institutional Demand Landscaping = 22%
Cooling & Heating = 28%
Restrooms = 37%</td><td>USEPA (2012c), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non-potable water in office buildings are restrooms, heating and cooling, and landscaping.</td></tr>
<tr><td colspan="2">Non-potable Business Demand
Landscaping = 22%
Cooling & Heating = 28%
Restrooms = 37%</td><td>USEPA (2012c), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non-potable water in office buildings are restrooms, heating and cooling, and landscaping.</td></tr>
<tr><td colspan="2">Non-potable Municipal Demand
Landscaping = 22%
Cooling & Heating = 28%
Restrooms = 37%</td><td>USEPA (2012c), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non-potable water in office buildings are restrooms, heating and cooling, and landscaping.</td></tr>
</table>

The non-potable water demand for each land use type is summarized in Table 6.6. A detailed breakdown is provided in Appendix A7.

Table 6.6 - Non-potable water demand.

Description	Average Potable Demand (ℓ/s)	Average Non-potable demand (ℓ/s)
Residential 2	16.435	3.229
Residential 2 (Duplex Units)	0.475	0.093
Residential 4	15.097	4.889
Church	0.014	0.008
Crèche	0.007	0.004
Primary School	0.047	0.029
Institutional	1.624	1.413
Business 2	2.261	1.967
Municipal	1.476	1.284
Public Open Spaces	0.134	0.134
Total	**37.570**	**13.051**
Percentage of Potable Demand		**35%**

RWH tank sizing

The Mpofu (2019) generalised Storage-Yield-Reliability relationships for the sizing of RWHS was then used to determine the appropriate tank size to service the non-potable demand of each land use type at a reliability of 50%. A detailed breakdown is provided in Appendix A8. Table 6.7 represents a summary of the results.

Table 6.7 - Sizing of RWH tanks.

Zoning	Area (Ha)	No. of Units	Kr = Storage required to achieve reliability r (ℓ)	No. of blocks	Standard Size (ℓ)
Residential 2	10.4554	1420	3232	1	1x3500l tank per unit.
Residential 2 (Duplex Units)	0.51	41	3115	1	1x3500l tank per unit.
4029 (5 x 15 Units per block)	0.6272	75	24096	5	2x10000l & 1x5000l tank per block.
4030 (1 x 9 Units per block)	0.664	79	14840	1	1x10000l & 1x5000l tank per block.
4030 (6 x 12 Units per block)			18131	6	2x10000l tank per block.
4031 (1 x 9 Units per block)	0.6864	82	17036	1	3x5000l & 1x2500 tank per block.
4031 (6 x 12 Units per block)			19464	6	2x10000l tank per block.
4032 (1 x 9 Units per block)	0.8194	98	14843	1	1x10000l & 1x5000l tank per block.
4032 (6 x 15 Units per block)			22602	6	2x10000l & 1x5000l tank per block.
4033 (2 x 9 Units per block)	1.0198	122	17041	2	3x5000l & 1x2500 tank per block.

Zoning	Area (Ha)	No. of Units	Kr = Storage required to achieve reliability r (ℓ)	No. of blocks	Standard Size (ℓ)
4033 (7 x 15 Units per block)			22542	7	2x10000l & 1x5000l tank per block.
4034 (11 x 9 Units per block)	0.8357	100	14841	11	1x10000l & 1x5000l tank per block.
4035 (8 x 15 Units per block)	0.9933	119	24095	8	2x10000l & 1x5000l tank per block.
4079 (2 x 9 Units per block)	1.268	152	14841	2	1x10000l & 1x5000l tank per block.
4079 (9 x 15 Units per block)			22603	9	2x10000l & 1x5000l tank per block.
4080 (1 x 9 Units per block)	1.4795	177	19647	1	2x10000l tank per block.
4080 (11 x 15 Units per block)			24095	11	2x10000l & 1x5000l tank per block.
4081(9 x 15 Units per block)	1.1242	135	22602	9	2x10000l & 1x5000l tank per block.
4082 (7 x 15 Units per block)	0.8755	105	24095	7	2x10000l & 1x5000l tank per block.
5524 (1 x 9 Units per block)	1.1551	138	14841	1	1x10000l & 1x5000l tank per block.
5524 (2 x 12 Units per block)			18131	2	2x10000l tank per block.
5524 (7 x 15 Units per block)			22603	7	2x10000l & 1x5000l tank per block.
5523 (1 x 9 Units per block)	1.0697	128	19647	1	2x10000l tank per block.
5523 (1 x 12 Units per block)			21651	1	2x10000l & 1x2500l tank per block.
5523 (7 x 15 Units per block)			24095	7	2x10000l & 1x5000l tank per block.
5522 (3 x 9 Units per block)	0.9332	111	14843	3	1x10000l & 1x5000l tank per block.
5522 (7 x 12 Units per block)			18131	7	2x10000l tank per block.
5521 (4 x 9 Units per block)	0.8171	98	19652	4	2x10000l tank per block.
5521 (4 x 15 Units per block)			24098	4	2x10000l & 1x5000l tank per block.
5520 (1 x 9 Units per block)	1.7358	208	19648	1	2x10000l tank per block.
5520 (13 x 12 Units per block)			21652	13	2x10000l & 1x2500l tank per block.
5520 (3 x 15 Units per block)			24096	3	2x10000l & 1x5000l tank per block.
5519 (1 x 9 Units per block)	2.0632	247	19648	1	2x10000l tank per block.
5519 (16 x 15 Units per block)			24096	16	2x10000l & 1x5000l tank per block.
Church	0.1184	1	13160	1	2x10000l & 3x3500l tank per block.
Crèche	0.386	1	11597	1	2x10000l & 1x2500l tank per block.
Primary School	2.6991	1	82387	1	8x10000l & 1x2500l tank per block.
Institutional	0.9356	1	663583	1	5x20000l

Zoning	Area (Ha)	No. of Units	Kr = Storage required to achieve reliability r (ℓ)	No. of blocks	Standard Size (ℓ)
Business 2	2.4418	2	959185	2	5x20000l
Municipal	3.2977	11	619765	11	5x10000l tank per block.
Total		1574			

6.3.2 Modelling of SuDS in PCSWMM

The modelling of SuDS in PCSWMM allows for the use of a combination of detention, infiltration and evapotranspiration to accurately determine runoff. A flow-duration curve illustrates the frequency of occurrence of various rates of flow. It is the cumulative frequency curve prepared by arranging all discharges in order of magnitude and subdividing them according to the percentages of time during which specific flows are equalled or exceeded. When utilizing flow-duration curves, all chronologic order or sequence is lost (Cheremisinoff, 1997). The cumulative duration for which individual discharge rates are exceeded under both pre- and post-development conditions are compared. The runoff hydrograph provides a graphical presentation of instantaneous runoff against time. The shape of hydrographs presented is dependent on the climatic factors (such as rainfall type, intensity, duration etc.) and physiographic factors (basin characteristics such as shape, size, roughness, slope, storage characteristic etc.). The change in shape of the recession limb is caused by the changes in pre-development and post-development physiographic factors.

Figure 6.14 represents the pre- and post-development flow duration curves before the implementation of any SuDS intervention.

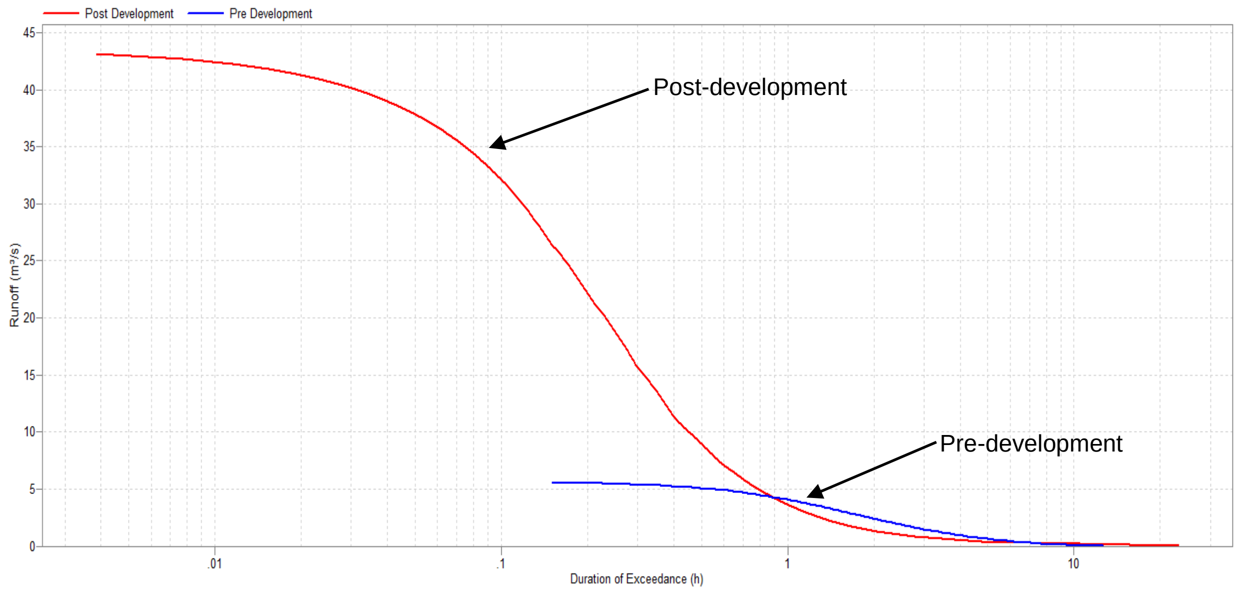


Figure 6.14 - Pre- versus post-development runoff

Three scenarios will be modelled in order to arrive at the best combination of interventions that meet our design specifications.

<u>Scenario one</u>

In the first scenario, RWH tanks are used to capture runoff from the roofs. It can be noted in Figure 6.15 that the implementation of this scenario reduced runoff volumes.

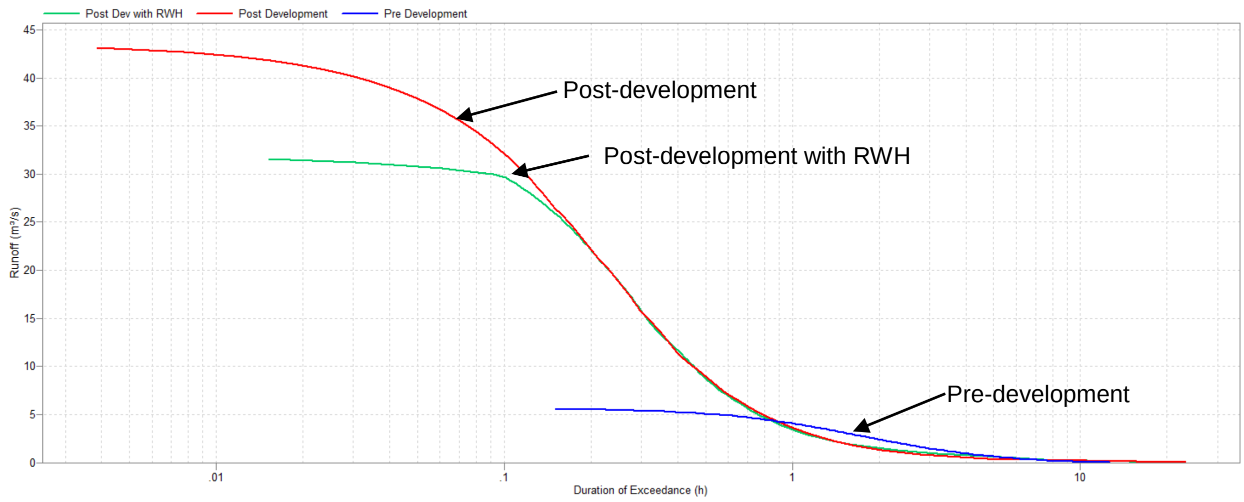


Figure 6.15 - Pre- versus post-development runoff with RWH

The peak runoff reduced from 43.58 m^3/s to 31.88 m^3/s as illustrated in Figure 6.16. This SuDS intervention has reduced post-development runoff by 11.70 m^3/s (26.85%).

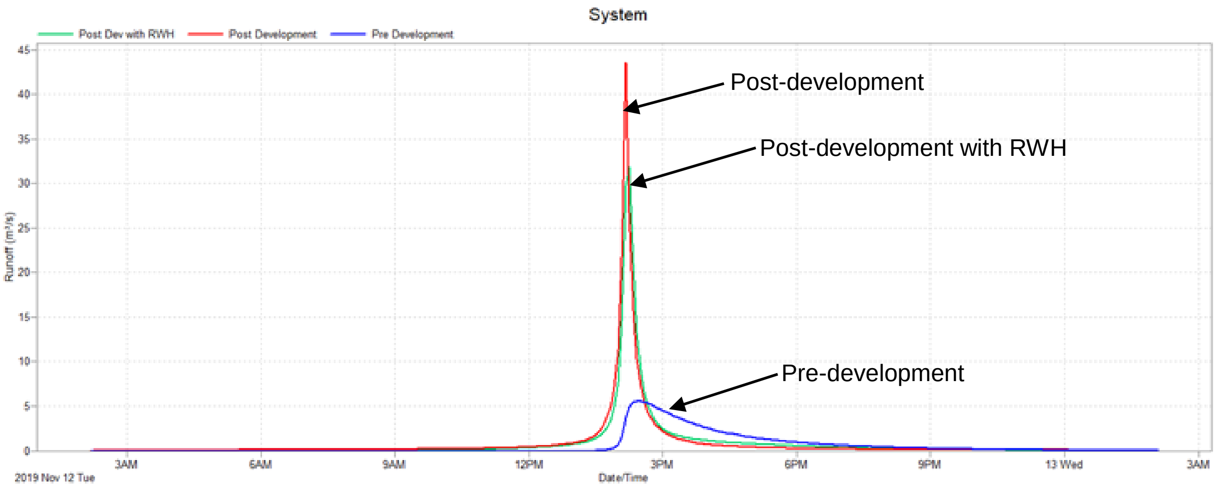


Figure 6.16 - Pre- versus post-development hydrograph with RWH

The model results are presented in Figure 6.17.

Objective functions for Runoff (m³/s)

From 11/12/2019 12:58:15 AM to 11/13/2019 3:16:45 AM (26.31 hours)

	System Post Dev with RWH	System Post Development	System Pre Development
Maximum Runoff (m³/s)	31.88	43.58	5.595
Minimum Runoff (m³/s)	0	0	0
Mean Runoff (m³/s)	0.8427	0.8669	0.5637
Duration of Exceedances (h)	23.92	23.92	23.92
Duration of Deficits (h)	0	0	11.25
Number of Exceedances	1	1	1
Number of Deficits	1	1	1
Volume of Exceedances (m³)	72550	74630	48530
Volume of Deficits (m³)	0	0	0
Total Runoff (m³)	72550	74640	48530

Figure 6.17 - Model results of pre- versus post-development with RWH

<u>Scenario two</u>

In the second scenario, bio-retention cells (or rain gardens) and vegetated swales are placed at strategic positions to intercept and convey runoff to an exit point. The vegetated swales also allow some infiltration of the runoff to take place. This is in addition to the RWH system.

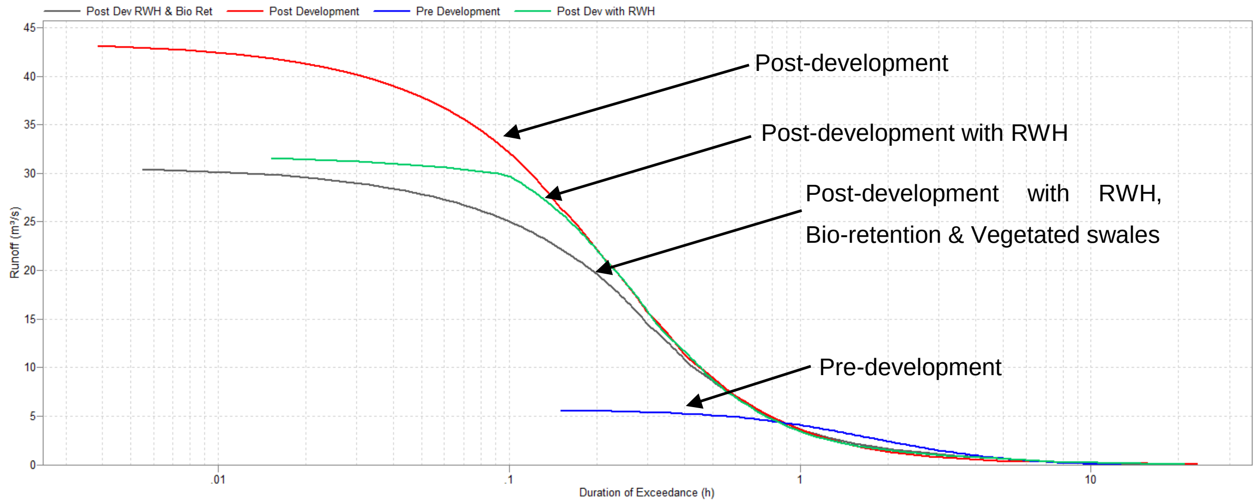


Figure 6.18 - Pre- versus post-development runoff with RWH, bio-retention and vegetated swales

The peak runoff reduced from 31.88 m^3/s to 30.70 m^3/s as illustrated in Figure 6.19. These SuDS interventions reduced post-development runoff by 1.18 m^3/s (3.70%).

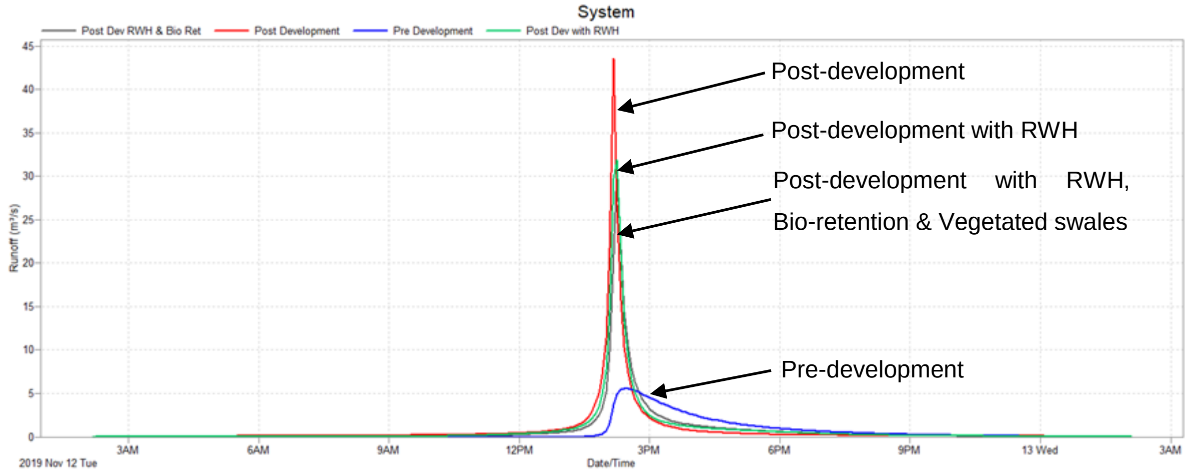


Figure 6.19 - Pre- versus post-development with RWH, bio-retention and vegetated swales

The model results are presented in Figure 6.17.

Objective functions for Runoff (m³/s)

From 11/12/2019 12:58:15 AM to 11/13/2019 3:16:45 AM (26.31 hours)

	System Post Dev RWH & Bio Ret	System Post Dev with RWH	System Post Development	System Pre Development
Maximum Runoff (m^3/s)	30.7	31.88	43.58	5.595
Minimum Runoff (m^3/s)	0	0	0	0
Mean Runoff (m^3/s)	0.7881	0.8427	0.8669	0.5637
Duration of Exceedances (h)	23.92	23.92	23.92	23.92
Duration of Deficits (h)	0	0	0	11.25
Number of Exceedances	1	1	1	1
Number of Deficits	1	1	1	1
Volume of Exceedances (m^3)	67850	72550	74630	48530
Volume of Deficits (m^3)	0	0	0	0
Total Runoff (m^3)	67850	72550	74640	48530

Figure 6.20 - Model results of Pre- versus post-development with RWH, bio-retention and vegetated swales

Appendix A10 provides a layout indicating the placement of the bio-retention cells (1355 m^2) and the vegetated swales (6419 m^2).

<u>Scenario three</u>

In the third scenario, a Detention Pond was added to the treatment train.

The detention pond was sized to provide the storage volume required to meet the pre-development peak runoff conditions after the implementation of RWH, bio-retention and vegetated swales. A major SuDS development requirement is to maintain peak runoff at or below pre-development conditions. One of the SUDs interventions for attenuating the additional runoff due to the development is to restrict discharge with a detention pond control structure. This was utilized in this water sensitive design alternative.

The PCSWMM storage calculator was used to estimate the required storage volume of the detention pond after the implementation of RWH, bio-retention and vegetated swales. This was achieved by first determining the peak runoff of the existing condition (after RWH, bio-retention and vegetated swales). The storage calculator was then used to estimate the required storage volume of the proposed pond to match the target peak runoff value. Once the detention ponds required volume was estimated, a detention pond was represented in the model using a storage unit and an outlet with an assigned rating curve. Figure 6.21 provides an illustration of the required storage volume of the proposed pond. The shaded area represents the difference between the runoff volumes after the implementation RWH, bio-retention and vegetated swales and the pre-development runoff. The storage volume required to obtain maintain the pre-development peak runoff is 33 018.82m^3.

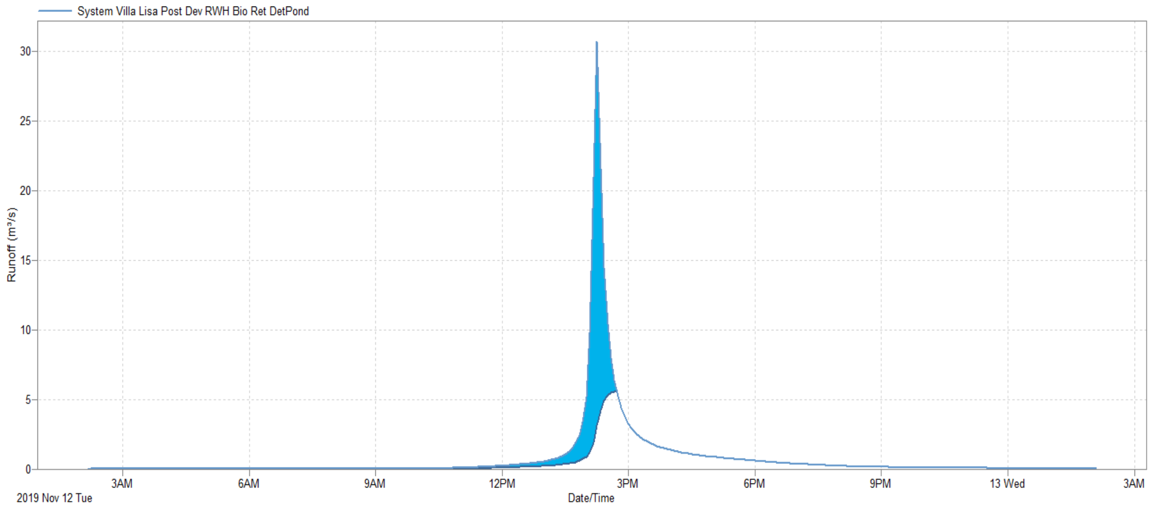


Figure 6.21 - Storage required to meet pre development flows

The third scenario represents the implementation RWH, bio-retention, vegetated swales and a detention pond to meet pre-development conditions. In Figure 6.22, the red hydrograph represents the post-development scenario, the blue hydrograph represents the pre-development and the magenta hydrograph represents the proposed post-development design with SUDs implemented.

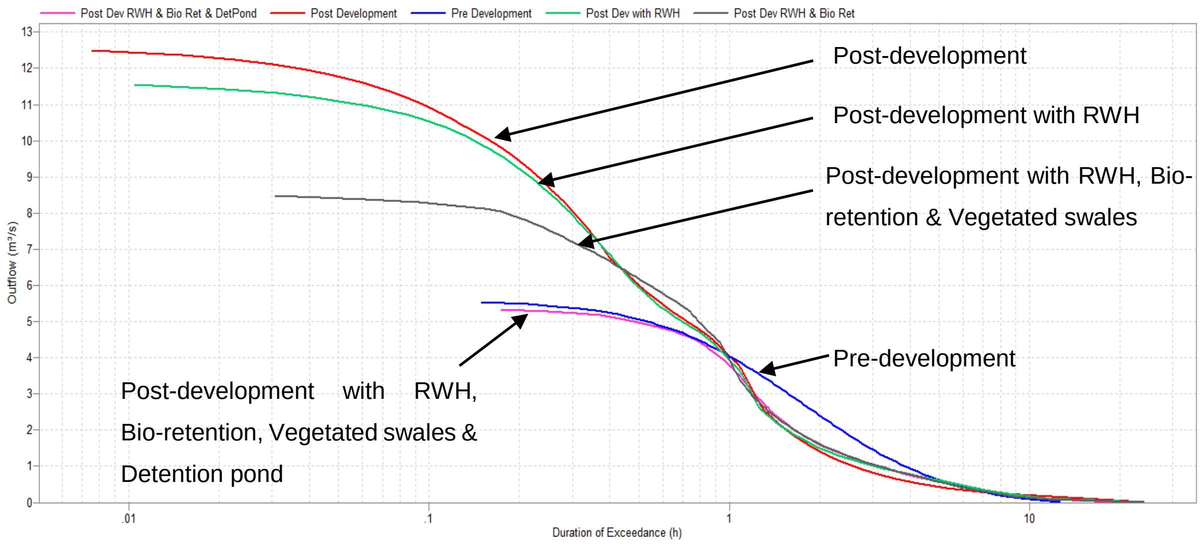


Figure 6.22 - Pre- versus post-development runoff with RWH, bio-retention, vegetated swales & detention pond.

The peak runoff generated reduced from 30.70 m^3/s to 30.03 m^3/s as illustrated in Figure 6.23. These SuDS interventions reduced post-development runoff by 0.67 m^3/s (2.18% drop in runoff generated).

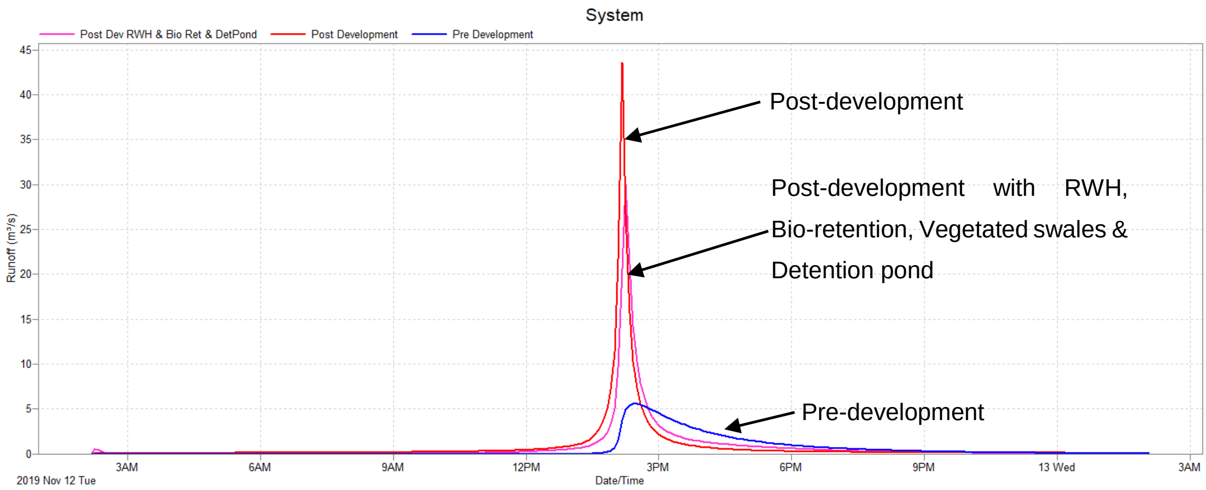


Figure 6.23 - Pre- versus post-development with RWH, bio-retention, vegetated swales and detention pond

The model results are presented in Figure 6.24.

Objective functions for Runoff (m^3/s)

From 11/12/2019 12:58:15 AM to 11/13/2019 3:16:45 AM (26.31 hours)

	System Post Dev RWH & Bio Ret & DetPond	System Post Development	System Pre Development
Maximum Runoff (m^3/s)	30.03	43.58	5.595
Minimum Runoff (m^3/s)	0	0	0
Mean Runoff (m^3/s)	0.7776	0.8669	0.5637
Duration of Exceedances (h)	23.92	23.92	23.92
Duration of Deficits (h)	0	0	11.25
Number of Exceedances	1	1	1
Number of Deficits	1	1	1
Volume of Exceedances (m^3)	66940	74630	48530
Volume of Deficits (m^3)	0	0	0
Total Runoff (m^3)	66950	74640	48530

Figure 6.24 - Model results of Pre- versus post-development with RWH, bio-retention, vegetated swales and detention pond

The detention pond has been sized to obtain and outlet flow of 5.387 m^3/s to meet the pre-development runoff conditions. The sizing of the detention facilities in this scenario is for the purposes of flood control and not runoff reduction. Figure 6.22 thus presents the outflow duration curves indicating an outflow drop to pre-development conditions while the runoff hydrograph in figure 6.23 indicates a miniature drop in runoff levels.

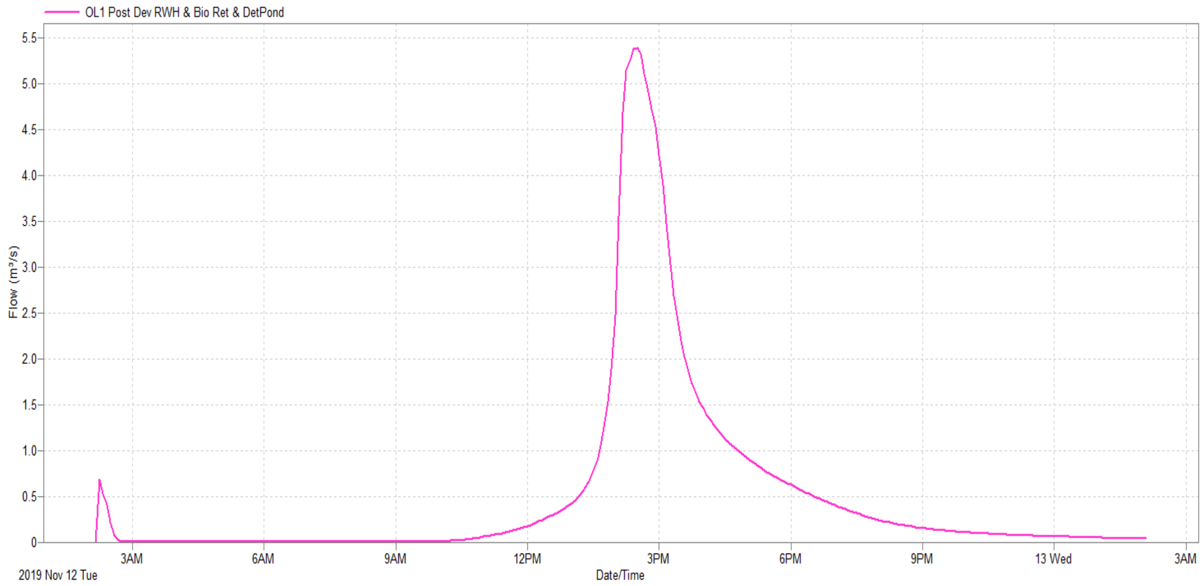


Figure 6.25 -Outlet flow of detention pond

Appendix A10 provides a layout indicating the placement of the detention pond.

6.4 Summary of results

In total, a 31.10% reduction in the peak runoff was achieved (from conventional post-development runoff of 43.58 m3/s to 30.03 m3/s in Scenario 3). Table 6.8 presents a summary of runoff and outflow results.

Table 6.8 - Summary of results

	Pre-development	**Post-development no control**	**Post-development with RWH, bio-retention, vegetated swales and detention pond**
Runoff	5.595 m^3/s	43.580 m^3/s	30.030 m^3/s
Outflow (m^3/s)	5.589 m^3/s	43.580 m^3/s	5.387 m^3/s
Total runoff volume (m^3)	48 530 m^3	74 640 m^3	66 950 m^3

6.5 Project costs and benefits

This section aims to evaluate the potential benefits and/or losses from the implementation of recommended WSUD solutions in relation to conventional infrastructure solutions. The costs associated with the implementation include:

- The initial capital investment which covers the cost of constructing the project
- Operations and maintenance costs incurred while the project is operational

The benefits include:

- Savings in water tariffs
- Environmental benefits
- Social benefits

A Cost Benefit Analysis (CBA) was utilized to determine whether the implementation of WSUD is viable. There are several types of analytical and economic methods that can be used to determine if a project is economically feasible (Taigbenu, 2017). The different types are; Net Present Value (NPV), Internal Rate of Return (IRR), Benefit Cost Ratio (BCR), Return on Investment (ROI), Cost Effectiveness Ratio and the payback period. The BCR method was used in this study.

A BCR is a ratio used to indicate the overall relationship between the project costs and benefits. A BCR greater than 1.0 means that the project is expected to deliver a positive net present value (Hayes, 2019). This method is regularly used in analysing and making decisions on development alternative of projects.

It is outside of the scope of this study to measure the environmental and social monetary benefits, such as improved amenity, reduced pollutants and improved air quality. As such the benefits will focus on the savings in water tariffs.

6.5.1 Projects costs

To establish the least expensive intervention, the time stream of costs throughout the project's lifetime was brought to its present value (PV). The present value is a single sum that represent all cost incurred presently (capital cost), annually and periodically throughout the life of the project. Table 6.9 provides a comparison of all costs associated with each intervention. A detailed bill of quantities is provided for each intervention in appendix A11.

Table 6.9 - Project Cost

Item	Capital Cost	Annual operations and maintenance cost	Present value of operations and maintenance cost	Total
Potable water infrastructure	R16,289,588.00	R286,539.18	R4,053,247.67	R20,342,835.67
Stormwater infrastructure	R64,914,140.92	R277,176.00	R3,920,800.55	R68,834,941.48
Conventional post-development infrastructure cost				**R89,177,777.15**
Potable water infrastructure	R16,289,588.00	R286,539.18	R4,053,247.67	R20,342,835.67
Stormwater infrastructure (pipe network only)	R45,599,838.35	R232,650.50	R3,290,963.90	R48,890,802.24
RWH	R19,907,365.00	R668,726.44	R9,459,487.82	R29,366,852.82
WSUD Scenario 1 – Post-development (with RWH)				**R98,600,490.73**
Bio-retention	R6,423,955.84	R174,438.05	R2,467,518.12	R8,891,473.96
Vegetated swales	R1,758,881.08	R350,024.95	R4,951,287.38	R6,710,168.45
WSUD Scenario 2 – Post-development (with RWH, bio-retention cells and vegetated swales)				**R114,202,133.14**
Detention pond	R6,111,021.58	R231,786.06	R3,278,735.94	R9,389,757.51
WSUD Scenario 3 – Post-development (with RWH, bio-retention cells, vegetated swales and detention ponds)				**R123,591,890.65**

As presented in Table 6.9, the conventional water infrastructure will cost R16,289,588.00 while the annual operations and maintenance cost is R286,539.18. This annuity sum over the project life cycle of 40 years, at a nominal interest rate of 6.5% (JSE, 2019) translates to a present value of R4,053,247.67.

The total cost of the conventional design is R89,177,777.15 and the total cost of the WSUD is R123,591,890.67.

6.5.2 Savings in water tariffs

The COE's 2019/2020 water tariffs policy was used to determine the value of water saved due to the use of RWH systems. As per the policy, COE provides a basic (free) portion of water and waste water services to poor households (registered indigent households) and households with property values not exceeding R750 000. 3 kl of free basic consumption (COE, 2019) is granted to all registered indigent account holders subject to the stipulations of the Council's indigent policy. Table 6.10 indicates the applicable tariffs to be applied.

Table 6.10 - COE 2019/2020 Water tariffs

Tariff type	Tariff Summary	Tariff R/kl 2019/20 (Excluding VAT)
Household Use: Tariff Code WA0017	Number of residential units x (0 – 6 kl/month)	11.74
	Number of residential units x (7 – 15 kl/month)	19.34
	Number of residential units x (16 – 30 kl/month)	23.69
	Number of residential units x (31 – 45 kl/month)	29.47
Institutional Use: Tariff Code WA0009	0-200 kl/month	19.75
Business and Other Uses Municipal: (Tariff Code WA0035)	0 – 5 000 kl/ month	25.37

The tariffs were then used to determine the value of potable water saved by the use of RWH systems to meet the non-potable demand of this development. The results of this exercise are indicated in Table 6.11.

Table 6.11 - Value of potable water savings

Description	Average potable demand (ℓ/s)	Average non-potable demand (ℓ/s)	Potable demand (kℓ/month)	Cost of consumption per month	Non-potable demand (kℓ/month) (potential Water savings)	Potential savings per month
Residential 2	16.435	3.229	43196.400	R 1,023,322.72	8486.538	R 201,046.07
Residential 2 (Duplex Units)	0.475	0.093	1247.220	R 29,546.64	245.034	R 5,804.85
Residential 4	15.097	4.889	39679.848	R 940,015.60	12848.974	R 248,499.16
Church	0.014	0.008	36.017	R 711.34	22.151	R 437.47
Crèche	0.007	0.004	17.796	R 351.47	10.944	R 216.15
Primary School	0.047	0.029	123.160	R 2,432.41	75.743	R 1,495.93
Institutional	1.624	1.413	4269.143	R 108,308.15	3714.154	R 94,228.09
Business 2	2.261	1.967	5942.364	R 150,757.79	5169.857	R 131,159.27
Municipal	1.476	1.284	3880.071	R 98,437.40	3375.662	R 85,640.54
Public Open Spaces	0.134	0.134	352.913	R 8,953.39	352.913	R 8,953.39
Total	**37.570**	**13.051**	**98744.932**	**R 2,362,836.90**	**34302**	**R 777,480.94**
Percentage of Potable Demand		**34.74%**				**33.00%**

The use of RWH can potentially save R 777,480.94 (33% in water bills) each month and R 131,974,530.89 over 40 years. This is equivalent to 34.74% savings in the potable water demand. The 1.74% difference between the savings in litres and the savings in Rands is due to the variable average demand per land use and the different tariffs levied per end user. Table 6.12 presents the value of the water savings.

Table 6.12 - Annuity present value of savings

Benefits			
Item	Monthly savings	Annual Savings	Annual Savings (**Annuity Present Value**)
	2019	2019	Rate: 6.5% Period: 40 years
Savings in water consumption	R 777,480.94	R 9,329,771.32	R 131,974,530.89

6.5.3 Benefit Cost Ratio

The benefit cost analysis utilizes the ratio:

$$\frac{\boldsymbol{B}}{\boldsymbol{C}} > \mathbf{1} \quad \textbf{\textit{or}} \quad \boldsymbol{B} - \boldsymbol{C} > \mathbf{0} \tag{6.1}$$

where B represents the benefits, and C the costs. There is justification for the implementation of the project when equation 6.1 is satisfied.

$$\frac{BENEFITS}{WSUD\ COSTS} = \frac{R\ 131{,}974{,}530.89}{R\ 123{,}591{,}890.67} = 1.068$$

With a benefit cost ratio greater than unity (1), the WSUD is considered better than the conventional design alternative.

7. CONCLUSIONS AND RECOMMENDATIONS

The key variables evaluated in the design of this case study area is potable water demand and stormwater quantity (volume and peak flow). Calculation methods and parameters used were described and referenced to literature and acceptable best management practice. Following the design of both systems, the cost of implementation was determined by measuring the quantities of the different elements of the solutions and applying the applicable market related cost.

7.1 Summary findings

This study aimed to determine if the benefits of applying WSUD methods to a greenfields township development in South Africa are likely to outweigh the benefits of conventional design methods. WSUD and conventional practices were applied to the case study area, results were compared and used to confirm or contradict the practicality of WSUD design in greenfield development.

The study contributed to the objectives in the following manner:

<u>Objective 1</u>

The first objective was to identify applicable conventional and WSUD solutions for a greenfields township development in Johannesburg. The conventional solutions included the design of a stormwater drainage management system where runoff is conveyed in a system of pits and pipes. The conventional design of a water supply systems was also done, where municipal water is conveyed in a system of pipes to end users. The study determined the following WSUD solutions as the most applicable to the study area: All conventional solutions, RWH, bio-retention, vegetated swales and detention pond.

<u>Objective 2</u>

The second objective was to determine pre- and post- development run-offs using both methods. The pre- and post- development run-offs determined are presented in Table 6.8. The runoff for the pre-development condition is 5.589m^3/s, 43.580m^3/s for the post-development no control scenario and 30.030m^3/s for the post-development scenario with RWH, bio-retention, vegetated swales and detention pond.

Objective 3

The third objective was to calculate and compare the cost of conventional and WSUD stormwater and potable water infrastructure. As presented in Table 6.9, the total cost of the conventional design is R89,177,777.15 and the total cost of the WSUD is R123,591,890.67.

Objective 4

The final objective was to assess the potential savings and/or losses (if applicable) from the implementation of recommended WSUD solutions in relation to conventional infrastructure solutions. The cost benefit ratio was used to determine if the WSUD is economical as compared to the conventional design. A BCR of 1.07 (conventional versus WSUD Scenario 3 post-development) was calculated. This benefit cost ratio indicates that the WSUD Scenario 3 alternative is financially better than the conventional alternative. WSUD Scenario 1 and WSUD Scenario 2 also result in higher BCRs of 1.34 and 1.16 respectively.

7.2 Conclusions

The major negative impacts of rapid urban development, such as increased stormwater flows, and increased sediment and nutrient loads associated with these flows, needs to be carefully managed to ensure ecosystem sustainability (Water By Design, 2010). The WSUD approach encourages water management authorities to find sustainable solutions that address all other affected sectors and the impact of the total water cycle in design solutions.

The key goal of this study was to determine if the benefits of applying WSUD methods to a greenfield urban development in South Africa is likely to outweigh the benefits of conventional design methods. Prior to the detail design of chosen WSUD an overview to the many WSUD interventions were provided. The study focused on the design of water systems using two approaches and its effectiveness in reducing runoff and potable water demand. The implementation of WSUD within new developments allows for the reduction in stormwater run-off generation and a decrease in potable water demand. Although the total cost of implementing WSUD was significantly higher than the conventional design, the cost benefit analysis indicates that WSUD is economically more desirable than the conventional design. This conclusion is arrived at even without considering the environmental and social benefits (e.g. improved amenity, reduced pollutants and improved air quality) from the implementation of WSUD infrastructure.

In a developing country such as RSA which is committed to addressing the effects of rapid urbanisation despite being a water-stressed country, it is imperative that national and local authorities as well as developers implement alternative approaches to conventional water management. This will assist in ensuring all citizens have access to the basic water service in a manner that is resource-efficient and affordable whilst minimising the negative environmental impacts.

7.3 Recommendations

Greenfield developments present a unique opportunity to be designed in a water sensitive manner from the onset. This is particularly true in the case of public / government settlements commissioned by the Department of Human Settlements, where the municipality / local council can use approval processes as a means of introspection and to ensure that the concept of water sensitivity is incorporated and adapted. This will make it easier to manage and approve private developments.

7.4 Future Research Suggestions

The possibilities for further research has been identified by this study and presented in this section. These questions stem from gaps identified during the research period. Future research projects identified include the following:

- The intensive study of WSUD applications on a regional/catchment level. The study should include phases such as: the development of a pre-intervention total water balance; the development of a post-intervention total water balance.
- An investigation into the viability of water reuse options such as RWH, greywater harvesting, and water efficient devices on a regional level in the RSA context.

- A comprehensive study into the means of determining environmental benefits, socio-economic benefits, property value benefits including amenity value post the implementation of WSUD in the RSA context.
- The effects of water quality in the conventional and WSUD context.

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9. APPENDICES

Appendix A1 - CoE Water services Demands

WATER SERVICES DIVISION TABLE OF CONSUMPTION: CALC BULK CONTRIBUTIONS FOR DEVELOPMENTS AVERAGE ANNUAL DAILY DEMAND (AADD)			
NO	ZONING	FINAL SUBMISSION Proposed Final Recommendations	
		Water	Waste Water
1	**Residential development:-**		
1.1	Low cost housing - erf up to 500m² *(kl per erf)*	0.7	0.6
1.2	Conventional small sized erf up to 500m² (Res 1) *(kl per erf)*	0.8	0.5
1.3	Medium sized erf 501m² - 1000m² (Res 1) *(kl per erf)*	0.8	0.5
1.4	Large sized erf 1001m² - 1500m² (Res 1) *(kl per erf)*	1.0	0.6
1.5	Extra large erf - 1 501m² and larger (Res 1) *(kl per erf)*	1.5	1.0
1.6	Cluster housing up to 20 units per hectare (Res 2) *(kl per unit)*	1.0	0.65
1.7	Cluster housing 21 up to 40 units per hectare (Res 3) *(kl per unit)*	0.8	0.53
1.8	Cluster housing 41 up to 60 units per hectare (Res 4) *(kl per unit)*	0.6	0.4
1.9	Cluster housing 61 up to 80 units per hectare (Res 4) *(kl per unit)*	0.5	0.34
1.10	Cluster housing 81 up to 100 units per ha (Res 5) *(kl per unit)*	0.4	0.28
1.11	High-rise flats (± 50m² per unit) + FSR residential *(kl/unit/50m²)*	0.6	0.4
1.12	Boarding houses, hostels, hotels, retirement centres & villages, orphanages *(kl per 100m²)*	0.9	0.6
	ALTERNATIVE CATEGORY: Hotels *(kl per 100m²)*	0.2	0.2
1.13	Guesthouses - allocation per room regardless of room size *(kl per room)*	0.4	0.4
1.14	Agricultural holdings & farm land (connection for domestic use only) *(kl per domestic unit)*	2.4	1.56
1.15	Agricultural holdings (house + servants quarters + garden) - to be used only for subdivisions to create multiple holdings *(kl per holding)*	4.0	1.4
1.16	Gate house for security villages *(kl per unit)*	0.2	0.2
2	**Business development:-**		
2.1	General business with an FSR (dry) *(kl per 100m²)*	0.8	0.6
2.2	General business with an FSR (wet) *(kl per 100m²)*	1.2	0.8
2.3	Business 4 offices *(kl per 100m² floor space)*	0.6	0.4
2.4	Gym, health spa *(kl per 100m2 floor space)*	0.6	0.4
2.5	Commercial *(kl per 100m² floor space)*	1.0	0.7
2.6	Restaurant , bakery *(kl per 100m² floor space)*	1.0	0.8
2.7	Butchery *(kl per 100m² floor space)*	0.2	0.2
2.8	Warehousing (including up to 20% offices) *(kl per100m²)*	0.6	0.4
2.9	Garage or filling station *(kl per 100m²)*	1.2	1.0
2.10	Car wash facility (no recycling) *(kl per washbay)*	6.0	6.0
2.11	Car wash facility (with recycling plant) (kl per washbay)	3.6	3.6
2.12	Motor city / Retail park as a single zoning (car sales + limited offices) *(kl per 100m²)*	0.6	0.4
2.13	Vehicle parking garage/grounds *(kl per bay)*	0.0	0.0
2.14	Nursery (sales area) *(kl per 100m²)*	0.4	0.4
2.15	Nursery (planting and production area) *(kl per hectare)*	15.0	0.2
3	**Industrial development:-**		
3.1	Industrial (dry) *(kl per 100m²)*	0.5	0.4
3.2	Industrial (wet) *(kl per 100m²)*	min of 1.2 kl	min of 1.0 kl
4	**Institutional uses:-**		
4.1	Club buildings *(kl per 100m²)*	0.3	0.3
4.2	Club grounds *(kl per hectare)*	3.0	0.0
4.3	Stadium building *(per 1 000 people)*	1.5	1.5
4.4	Stadium grounds *(kl per hectare)*	3.0	0.0
4.5	Municipal park buildings *(kl per 100m²)*	0.4	0.4
4.6	Municipal park grounds *(kl per hectare)*	3.5	0.0
4.7	Hospital buildings without laundry *(kl per 100m²)*	1.54	1.0
4.8	Hospital buildings with laundry *(kl per 100m²)*	4.2	3.15

Appendix A1 - CoE Water services Demands

NO	ZONING		Proposed Final Recommendations	
			Water	Waste Water
4.9	Hospital grounds	(kl per hectare)	10.0	0.0
4.10	Church buildings	(kl per 100m ²)	0.1	0.1
4.11	Church grounds	(kl per hectare)	10.0	1.0
4.12	School, creche, educational buildings	(kl per pupil)	1.5	0.5
4.13	School, creche, educational grounds	(kl per hectare)	1.5	0.5
4.14	Municipal, governmental developments	(kl per 100m ²)	1.0	0.8
5	**Miscellaneous uses:-**			
5.1	Mixed use	(kl per hectare)	20.0	13.0
5.2	Private open space	(kl per hectare)	10.0	0.0
5.3	Special	(Development specific)	Development specific	Development specific
5.4	Home enterprise (dry, re office, IT)	(kl per 100m2)	0.4	0.4
5.5	Home enterprise (wet, re tavern, hairdresser)	(kl per 100m2)	0.8	0.8

A2 - Conventional water network layout plan

A2 - Conventional water network layout plan

A2 - Conventional water network layout plan

Water Network Design Parameters		
Parameter	**Element**	**Guidelines**
Flow Velocity	Residential Areas	Max 2 m/s (Excluding Fire Flow)
Losses	Secondary	10%
Fire Flow	Business Area / Cluster Housing (SANS 10090:2003)	
	Flow at Hydrant	25l/s
	Total Flow	100l/s
	Pressure	Min 15m (At Abstraction point)
Fire Flow	Single Residential erven(SANS 10090:2003)	
	Flow at Hydrant	25l/s
	Total Flow	50l/s
	Pressure	Min 15 m (At Abstraction point)
Fire Hydrants	Spacing	Business Area: 120m
		Residential Area: 240m
Water Supply Zones	Number of Stands per zone	Max :2000
Maintenance	Isolation zone size	600m of pipeline
	Max Number of valves to isolate a zone	4 valves
Piping	Size	Min: 75mm Dia
	Material	HDPE as per SANS 1601
Pipe Location	All Areas	As per CoE Typical Road Cross Sections
Valves	Spacing	Maximum 500m, not more than 4 valves to isolate a section (usually 3)
Cover to Pipes	Sidewalks	Min 0.8m
	Road Crossings	Min 1.0m
	Across Erven	Min 0,75m
		Max 1.0m
	Other services Present	Min 0,8m
		Max 1,5m

PRESSURE REPORT						
Id	From	To	Max Pressure	Chainage	Min Pressure	Chainage
11	11	10	8.528	0	8.511	6.779
14	13	11	8.573	0	8.519	36.862
15	13	14	8.573	0	8.462	20.915
18	15	16	8.625	0	8.566	9.891
93	117	116	8.972	0	8.956	7.5
94	117	118	8.972	0	8.935	17.5
99	115	123	9.347	133.867	9.232	38.086
100	123	117	9.327	0	8.962	91.804
103	125	124	9.258	0	9.201	25.801
105	125	127	9.282	0	9.212	38.102
106	127	126	9.221	0	9.168	24.865
112	131	130	8.933	0	8.862	13.693
113	131	132	8.933	0	8.795	34.502
114	129	133	9.138	0	9.06	55.71
115	133	131	9.074	0	8.924	45.557
118	135	134	8.7	0	8.653	13.697
119	135	136	8.7	0	8.609	26.517
123	138	135	8.841	0	8.691	38.878
130	140	141	9.13	0	9.081	47.019
133	142	139	8.793	0	8.733	43.341
138	144	141	9.102	0	9.081	47.019
141	145	143	9.013	0	8.936	37.695
148	147	149	9.075	0	9.054	45.991
149	149	145	9.081	0	8.986	49.646
150	147	150	9.132	43.717	9.066	0
151	150	148	9.136	10.284	9.123	0
154	151	149	9.15	0	9.054	49.784
162	154	152	9.04	0	8.96	47.645
165	155	142	8.786	22.022	8.663	0
168	142	157	8.858	50.358	8.777	0
172	158	156	7.911	47.146	7.769	0
174	152	160	8.969	0	8.888	47.331
177	157	161	8.876	16.467	8.849	0
181	163	158	7.778	47.168	7.706	0
187	143	166	9.081	10.052	8.936	0
194	169	168	8.6	0	8.544	9.552
195	169	170	8.6	0	8.473	22.555
207	174	178	8.986	47.713	8.937	0
218	183	182	8.212	0	8.147	47.713
220	184	183	8.248	0	8.203	47.243
222	185	167	8.344	51.48	8.335	0
224	186	185	8.418	0	8.335	47.243
226	187	186	8.478	0	8.409	47.713
229	188	189	8.763	47.643	8.735	0
231	189	190	8.787	47.713	8.754	0
233	190	191	8.825	47.243	8.778	0
237	192	193	8.996	47.243	8.953	0
239	193	194	9.041	47.713	8.987	0
240	194	140	9.13	65.319	9.032	0

PRESSURE REPORT						
Id	From	To	Max Pressure	Chainage	Min Pressure	Chainage
245	196	197	7.933	47.713	7.886	0
246	197	173	7.979	49.121	7.924	0
249	198	199	8.152	46.139	8.056	0
251	199	200	8.267	47.156	8.142	0
253	200	201	8.432	47.803	8.258	0
254	201	155	8.673	53.452	8.423	0
258	203	202	8.803	0	8.774	41.737
260	204	203	8.834	0	8.794	47.713
262	205	204	8.882	0	8.824	47.243
265	175	207	9.001	0	8.974	47.058
267	141	208	9.09	0	9.017	60.743
268	208	175	9.026	0	8.992	47.899
275	177	212	7.977	0	7.878	48.67
276	212	211	7.891	0	7.827	47.434
282	215	214	8.271	0	8.104	47.243
285	216	217	8.884	71.12	8.855	0
288	218	143	8.945	64.596	8.894	0
291	219	220	9.022	47.243	8.994	0
293	220	221	9.056	47.713	9.013	0
294	221	144	9.102	56.166	9.047	0
300	224	223	7.615	49.124	7.601	0
302	225	224	7.61	45.781	7.599	0
306	227	215	8.481	0	8.262	47.652
311	160	230	8.915	58.661	8.888	0
312	230	174	8.946	47.243	8.906	0
313	178	231	8.995	10.628	8.977	0
317	232	233	9.047	0	8.994	47.498
319	233	234	9.003	0	8.969	47.447
322	235	154	9.05	0	9.031	57.413
324	236	235	9.074	0	9.041	46.914
325	153	237	9.184	0	9.102	82.713
326	237	236	9.111	0	9.064	47.968
332	240	239	9.113	12.327	9.096	47.61
333	154	241	9.117	55.697	9.031	0
334	241	240	9.117	0	9.102	56.444
338	243	242	9.007	0	8.985	46.881
340	244	243	9.047	0	8.998	47.911
341	146	245	9.142	28.946	9.099	0
343	245	246	9.142	0	9.111	49.273
345	246	247	9.12	0	9.089	48.401
347	247	248	9.098	0	9.072	47.306
348	248	147	9.081	0	9.066	21.454
351	137	255	9.26	49.749	9.169	0
352	255	128	9.26	0	9.25	5.86
355	257	164	8.83	0	8.815	44.322
356	257	258	8.83	0	8.793	19.645
361	259	228	7.828	0	7.778	30.552
362	259	251	7.849	23.323	7.819	0
363	228	261	7.787	0	7.726	39.313

PRESSURE REPORT						
Id	From	To	Max Pressure	Chainage	Min Pressure	Chainage
364	261	229	7.74	63.244	7.711	12.661
365	261	252	7.757	23.014	7.726	0
368	167	263	8.658	30.645	8.335	0
369	263	169	8.658	0	8.587	25.049
373	166	264	9.081	0	8.99	47.641
374	264	165	9.017	0	8.933	47.694
376	265	138	8.935	0	8.827	100.358
377	165	265	8.959	0	8.902	28.203

PIPE REPORT							
Id	From	To	Max Flow	Max Velocity	Time	Volume	Avg. Flow
11	11	10	3.75	1.186	5:30	324	3.75
14	13	11	3.75	1.186	18:15	324	3.75
15	13	14	4.917	1.555	7:45	424.829	4.917
18	15	16	4.222	1.335	19:00	364.781	4.222
93	117	116	2.917	0.923	15:15	252.029	2.917
94	117	118	1.282	0.405	22:30	110.765	1.282
99	115	123	91.53	1.283	0:15	7908.203	91.53
100	123	117	4.448	1.407	2:45	384.307	4.448
103	125	124	0.845	0.267	15:15	73.008	0.845
105	125	127	86.237	1.209	0:15	7450.888	86.237
106	127	126	0.268	0.085	7:30	23.155	0.268
112	131	130	3.833	1.212	15:45	331.171	3.833
113	131	132	3.556	1.125	22:15	307.238	3.556
114	129	133	81.084	1.136	0:15	7005.661	81.084
115	133	131	7.389	1.081	19:30	638.41	7.389
118	135	134	3.083	0.975	15:45	266.371	3.083
119	135	136	2.722	0.861	6:45	235.181	2.722
123	138	135	5.805	0.849	11:45	501.552	5.805
130	140	141	3.005	0.95	6:00	259.629	3.005
133	139	142	23.604	1.042	0:15	2039.361	23.604
138	141	144	2.284	0.722	0:15	197.365	2.284
141	143	145	8.529	0.377	6:00	736.898	8.529
148	149	147	0.402	0.127	6:00	34.766	0.402
149	145	149	6.925	0.306	6:00	598.355	6.925
150	147	150	0.073	0.023	6:00	6.34	0.073
151	150	148	0.116	0.037	15:00	10.022	0.116
154	149	151	5.204	0.23	0:15	449.615	5.204
162	152	154	5.086	0.351	23:45	439.428	5.086
165	142	155	4.74	1.499	0:15	409.493	4.739
168	142	157	18.135	0.801	0:15	1566.866	18.135
172	156	158	4.367	1.381	0:15	377.328	4.367
174	160	152	7.475	0.33	23:45	645.818	7.475
177	157	161	11.492	0.507	0:15	992.899	11.492
181	158	163	3.69	1.167	0:15	318.803	3.69
187	166	143	8.761	2.771	0:15	756.985	8.761
194	169	168	5.778	1.267	23:00	499.219	5.778
195	169	170	6.861	1.504	4:00	592.79	6.861
207	174	178	0.163	0.052	6:00	14.071	0.163
218	183	182	7.038	0.486	0:15	608.059	7.038
220	184	183	7.779	0.537	0:15	672.081	7.779
222	185	167	1.365	0.432	1:45	117.964	1.365
224	186	185	2.106	0.666	1:45	181.986	2.106
226	187	186	2.847	0.9	1:45	246.008	2.847
229	188	189	0.159	0.05	6:00	13.774	0.159
231	190	189	0.582	0.184	0:15	50.249	0.582
233	191	190	1.323	0.418	0:15	114.271	1.323
237	193	192	0.344	0.109	0:15	29.732	0.344
239	194	193	1.085	0.343	0:15	93.755	1.085
240	140	194	1.826	0.578	0:15	157.777	1.826

PIPE REPORT							
Id	From	To	Max Flow	Max Velocity	Time	Volume	Avg. Flow
245	197	196	0.294	0.093	0:15	25.358	0.293
246	173	197	1.035	0.327	0:15	89.381	1.034
249	199	198	2.517	0.796	0:15	217.426	2.516
251	200	199	3.258	1.03	0:15	281.448	3.257
253	201	200	3.999	1.265	0:15	345.47	3.998
254	155	201	4.74	1.499	0:15	409.493	4.739
258	203	202	0.012	0.004	0:15	1.02	0.012
260	204	203	0.753	0.238	0:15	65.043	0.753
262	205	204	1.494	0.472	0:15	129.065	1.494
265	207	175	0.761	0.241	23:45	65.781	0.761
267	141	208	0.721	0.228	6:00	62.264	0.721
268	175	208	0.02	0.006	23:45	1.758	0.02
275	177	212	2.287	0.723	0:15	197.565	2.287
276	212	211	1.546	0.489	0:15	133.542	1.546
282	215	214	3.769	1.192	0:15	325.61	3.769
285	216	217	0.879	0.278	6:00	75.903	0.879
288	143	218	0.232	0.074	0:15	20.087	0.232
291	219	220	1.34	0.424	6:00	115.791	1.34
293	220	221	0.599	0.189	6:00	51.769	0.599
294	144	221	0.142	0.045	23:45	12.253	0.142
300	223	224	1.852	0.586	19:30	160.013	1.852
302	224	225	1.111	0.351	20:30	95.99	1.111
306	227	215	4.51	1.426	0:15	389.632	4.51
311	160	230	1.645	0.52	6:00	142.116	1.645
312	230	174	0.904	0.286	6:00	78.094	0.904
313	178	231	0.163	0.052	6:00	14.071	0.163
317	233	232	0.166	0.052	23:45	14.322	0.166
319	234	233	0.907	0.287	23:45	78.344	0.907
322	154	235	1.601	0.506	23:45	138.302	1.601
324	235	236	0.86	0.272	23:45	74.28	0.86
325	153	237	0.622	0.197	6:00	53.765	0.622
326	236	237	0.119	0.038	23:45	10.257	0.119
332	240	239	2.004	0.634	6:00	173.168	2.004
333	154	241	2.744	0.401	6:00	237.104	2.744
334	241	240	2.374	0.751	6:00	205.136	2.374
338	242	243	0.122	0.038	23:45	10.498	0.122
340	244	243	0.619	0.196	0:15	53.524	0.619
341	146	245	0.782	0.247	1:45	67.564	0.782
343	245	246	0.782	0.247	1:45	67.564	0.782
345	246	247	0.782	0.247	1:45	67.564	0.782
347	247	248	0.412	0.13	1:45	35.596	0.412
348	147	248	0.329	0.104	6:00	28.426	0.329
351	137	255	0.054	0.017	13:45	4.666	0.054
352	255	128	0.047	0.015	14:45	4.061	0.047
355	257	164	1.583	0.501	0:15	136.795	1.583
356	257	258	0.048	0.015	8:45	4.147	0.048
361	259	228	0.774	0.245	23:45	66.887	0.774
362	259	251	0.302	0.096	15:30	26.093	0.302
363	228	261	0.699	0.221	23:45	60.407	0.699

PIPE REPORT							
Id	From	To	Max Flow	Max Velocity	Time	Volume	Avg. Flow
364	229	261	0.279	0.088	0:15	24.092	0.279
365	261	252	0.978	0.309	15:00	84.499	0.978
368	263	167	7.895	2.497	0:15	682.163	7.895
369	263	169	12.639	0.873	12:00	1092.01	12.639
373	264	166	11.584	0.512	0:15	1000.821	11.584
374	165	264	16.062	0.709	0:15	1387.734	16.062
376	265	138	54.432	0.97	0:15	4702.912	54.432
377	265	165	19.263	0.851	0:15	1664.34	19.263

$$Q = \frac{CIA}{3.6} \qquad I = \frac{P_T}{T_C}$$

$$T_C = 0.604\left(\frac{rL}{S^{0.5}}\right)\text{^}0.467$$

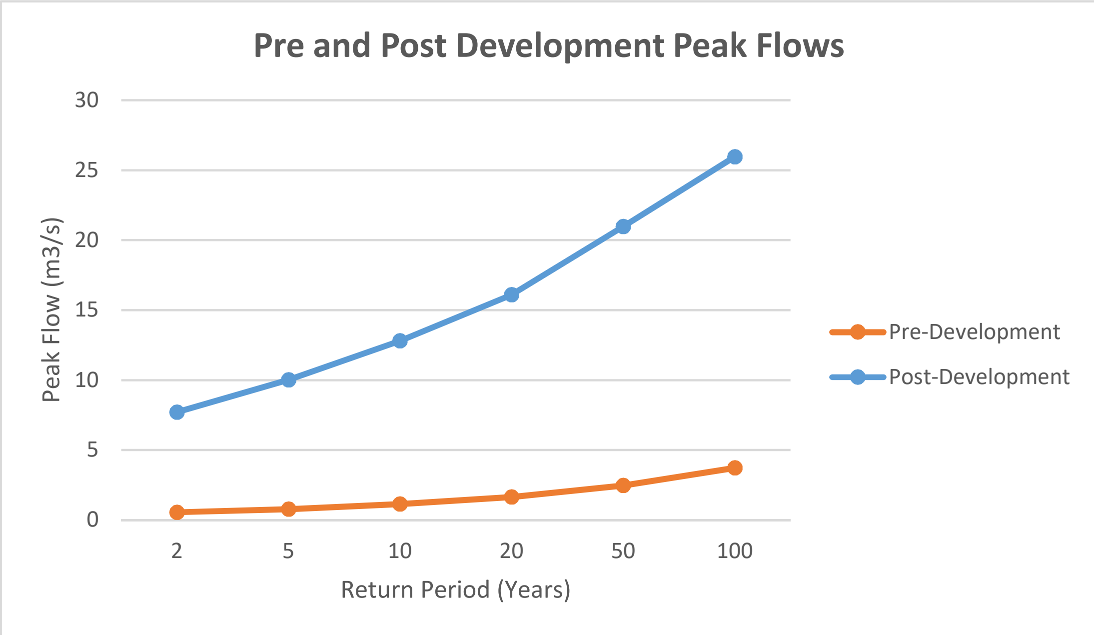
RATIONAL METHOD								
Description of Catchment		**Villa Lisa Ext 4 - Pre-Development**						
River detail		N/A						
Physical Characteristics								
Size of Catchment (A)	0.74648	Km²		Rainfall region		Inland		
Longest WaterCourse (L)	0.898	Km		Return Period		1:2, 1:5, 1:10, 1:20, 1:50 & 1:100		
Height Difference (H)	9.000	m		**Area Distribution factors**				
Average Slope (S)	0.01002	m/m		Rural α	Urban β	Lakes ϒ		
Roughness coefficient (r)	0.8			40.0%	60.0%	0%		
MAP (mm)	750	mm						
Table 3.7								
Rural (C1)					**Urban (C2)**			
Classification	%	**Factor**	**Cs**		**Discription**	%	**factor**	C_2
Surface Slope								
Vleins and pans (<3%)	100%	0.03	0.030		**Lawns**			
Flat areas (3%-10%)					Sandy, flat< 2%	100%	0.1	0.1
Hilly (10%-30%)					Sandy, steep>7%			
Steep Areas (>30%)					Heavy soil, flat<2%			
Total	100%		**0.030**		Heavy soil, steep>7%			
Permeability	%	**Factor**	**Cp**		**Residensial Areas**			
Very permeable					Houses			
Permeable	80%	0.08	0.064		Flats			
Semi-permeable	20%	0.16	0.032		**Industry**			
Impermeable					Light industry			
Total	100%		**0.096**		Heavy industry			
Vegetation	%	**Factor**	**Cv**		**Business**			
Thick bush and plantation					City centre			
Light bush and farm-lands					Suburban			
Grass lands	100%	0.21	0.21		Streets			
No vegetation					Maximum flood			
Total	100%		**0.21**		Total	100%	Total	0.1
		Total C1	**0.336**				**Total C_2**	**0.100**
Time of convcentration (Tc)								
Overland flow		Defined water course			**Notes:** If Tc < 0.25Hrs, then use 0.25Hrs			
	hours	1.51612291	hours					
	Return Period							
Run-off coefficient	**2**	**5**	**10**	**20**	**50**	**100**	**max**	
Run-off coefficient C1	0.336	0.336	0.336	0.336	0.336	0.336		
Adj to run-off coefficient C1								
Adj factor for C1 (Ft)	0.5	0.55	0.6	0.67	0.83	1		
C1T	0.168	0.185	0.202	0.225	0.279	0.336		
CT	0.127	0.134	0.141	0.150	0.172	0.194		
Rainfall								
Return Period (T)	**2**	**5**	**10**	**20**	**50**	**100**	**max**	
Point precipitation PT mm)	32	43	60	80	105	140		
Point intensity PiT (mm/h)	21.1065	28.362	39.575	52.766	69.2555988	92.340798		
ARF %	100%	100%	100%	100%	100%	100%		
Average intensity It mm/h	21.1065	28.362	39.575	52.766	69.2555988	92.340798		
Peak flow Q m^3/s	**0.55669**	**0.7876**	**1.1541**	**1.6417**	**2.463571585**	**3.7222402**		

$$Q = \frac{CIA}{3.6} \qquad I = \frac{P_T}{T_C}$$

$$T_C = 0.604(\frac{rL}{S^{0.5}})\text{^}0.467$$

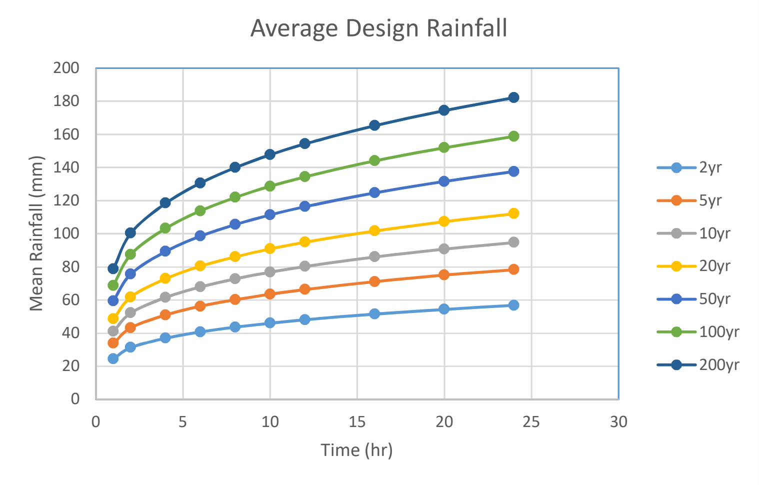
RATIONAL METHOD								
Description of Catchment			**Villa Lisa Ext 4 - Post-Development**					
River detail			N/A					
Physical Characteristics								
Size of Catchment (A)	0.74648	Km²		Rainfall region		Inland		
Longest WaterCourse (L)	0.898	Km		Return Period		1:2, 1:5, 1:10, 1:20, 1:50 & 1:100		
Heigh Difference (H)	9.000	m		**Area Distribution factors**				
Average Slope (S)	0.01002	m/m		Rural α	Urban β	Lakes Ƴ		
Roughness coefficient (r)	0.02			10.0%	90.0%	0%		
MAP (mm)	750	mm						
Table 3.7								
Rural (C1)					**Urban (C2)**			
Classification	%	**Factor**	**Cs**		**Discription**	%	**factor**	**C_2**
Surface Slope								
Vleins and pans (<3%)	100%	0.03	0.030		**Lawns**			
Flat areas (3%-10%)					Sandy, flat< 2%	18.65%	0.10	0.01865
Hilly (10%-30%)					Sandy, steep>7%			
Steep Areas (>30%)					Heavy soil, flat<2%			
Total	100%		**0.030**		Heavy soil, steep>7%			
Permeability	%	**Factor**	**Cp**		**Residensial Areas**			
Very permeable					Houses	14.69%	0.50	0.07345
Permeable					Flats	24.34%	0.70	0.17038
Semi-permeable	18.65%	0.16	0.0298		**Industry**			
Impermeable	81.35%	0.26	0.2115		Light industry			
Total	100%		**0.241**		Heavy industry			
Vegetation	%	**Factor**	**Cv**		**Business**			
Thick bush and plantation					City centre			
Light bush and farm-lands					Suburban	13.23%	0.70	0.09261
Grass lands	100%	0.21	0.21		Streets	29.09%	0.95	0.276355
No vegetation					Maximum flood			
Total	100%		**0.21**		Total	100%	Total	0.631445
		Total C1	**0.481**				**Total C_2**	**0.631**
Time of convcentration (Tc)								
Overland flow		Defined water course			**Notes:** If Tc < 0.25Hrs, then use 0.25Hrs			
	hours	0.27075245		hours				
	Return Period							
Run-off coefficient	**2**	**5**	**10**	**20**	**50**	**100**	**max**	
Run-off coefficient C1	0.481	0.481	0.481	0.481	0.481	0.481		
Adj to run-off coefficient C1								
Adj factor for C1 (Ft)	0.5	0.55	0.6	0.67	0.83	1		
C1T	0.241	0.265	0.289	0.323	0.400	0.481		
CT	0.592	0.595	0.597	0.601	0.608	0.616		
Rainfall								
Return Period (T)	**2**	**5**	**10**	**20**	**50**	**100**	**max**	
Point precipitation PT mm)	17	22	28	35	45	55		
Point intensity PiT (mm/h)	62.788	81.255	103.42	129.27	166.2034825	203.13759		
ARF %	100%	100%	100%	100%	100%	100%		
Average intensity It mm/h	62.788	81.255	103.42	129.27	166.2034825	203.13759		
Peak flow Q m³/s	**7.71228**	**10.021**	**12.806**	**16.098**	**20.96228251**	**25.965246**		

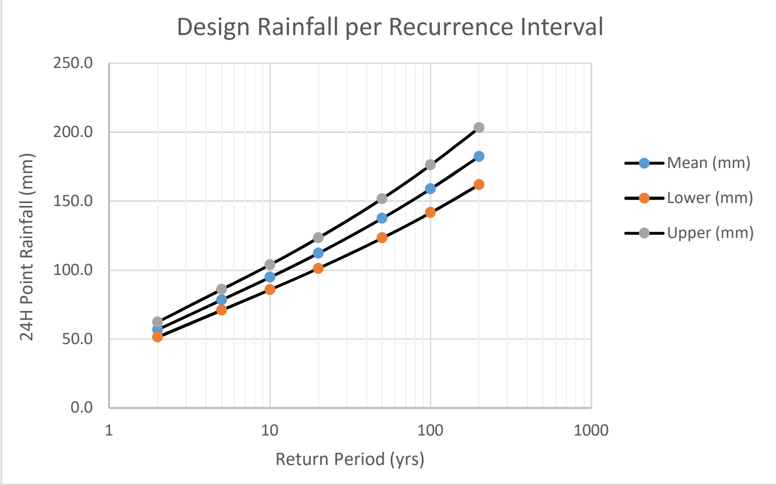
Peak flow Q m³/s		
Return Period	**Pre-Development**	**Post-Development**
2	0.556694834	7.71227716
5	0.787578765	10.02114443
10	1.154091413	12.8057935
20	1.641724577	16.0975588
50	2.463571585	20.96228251
100	3.722240224	25.96524642



Smithers & Schulze Database for OR Tambo International Airport weather station
Database Output Processed

Duration	Event Rain Depth for Recurrence Interval (mm)																				
	2 yr			5 yr			10 yr			20 yr			50 yr			100 yr			200 yr		
hr	Mean	Lower	Upper	Mean	Lower	Upper	Mean	Lower	Upper	Mean	Lower	Upper	Mean	Lower	Upper	Mean	Lower	Upper	Mean	Lower	Upper
1	24.5	19.8	29.2	33.9	27.4	40.4	41	33.1	48.8	48.5	39.1	58	59.5	47.7	71.4	68.7	54.8	82.9	78.8	62.6	95.5
2	31.3	25.7	36.9	43.2	35.5	50.9	52.2	42.9	61.6	61.8	50.6	73.1	75.7	61.7	89.9	87.5	70.9	104.4	100.4	81	120.4
4	36.9	31.2	42.7	51	43	58.9	61.6	52	71.2	72.9	61.4	84.6	89.4	74.8	104.1	103.3	86	120.8	118.5	98.2	139.3
6	40.7	34.9	46.5	56.2	48.2	64.2	67.9	58.2	77.6	80.4	68.7	92.1	98.6	83.7	113.3	113.8	96.3	131.6	130.6	110	151.7
8	43.6	37.8	49.4	60.2	52.2	68.2	72.8	63.1	82.4	86.1	74.4	97.8	105.6	90.7	120.4	122	104.3	139.8	140	119.1	161.2
10	46	40.2	51.7	63.5	55.5	71.4	76.8	67.1	86.4	90.9	79.2	102.5	111.4	96.6	126.2	128.7	111	146.5	147.7	126.8	169
12	48.1	42.3	53.8	66.3	58.4	74.2	80.2	70.6	89.7	94.9	83.3	106.6	116.4	101.6	131.1	134.4	116.8	152.3	154.3	133.4	175.6
16	51.5	45.8	57.1	71.1	63.3	78.9	86	76.5	95.3	101.7	90.3	113.2	124.7	110.1	139.3	144	126.5	161.8	165.3	144.5	186.5
20	54.3	48.8	59.8	75	67.4	82.7	90.7	81.4	99.9	107.3	96.1	118.6	131.6	117.1	146	152	134.6	169.5	174.4	153.8	195.5
24	**56.8**	51.3	62.2	**78.3**	70.9	85.9	**94.7**	85.7	103.8	112.1	101.1	123.3	137.5	123.2	151.7	158.8	141.6	176.2	182.2	161.8	203.1



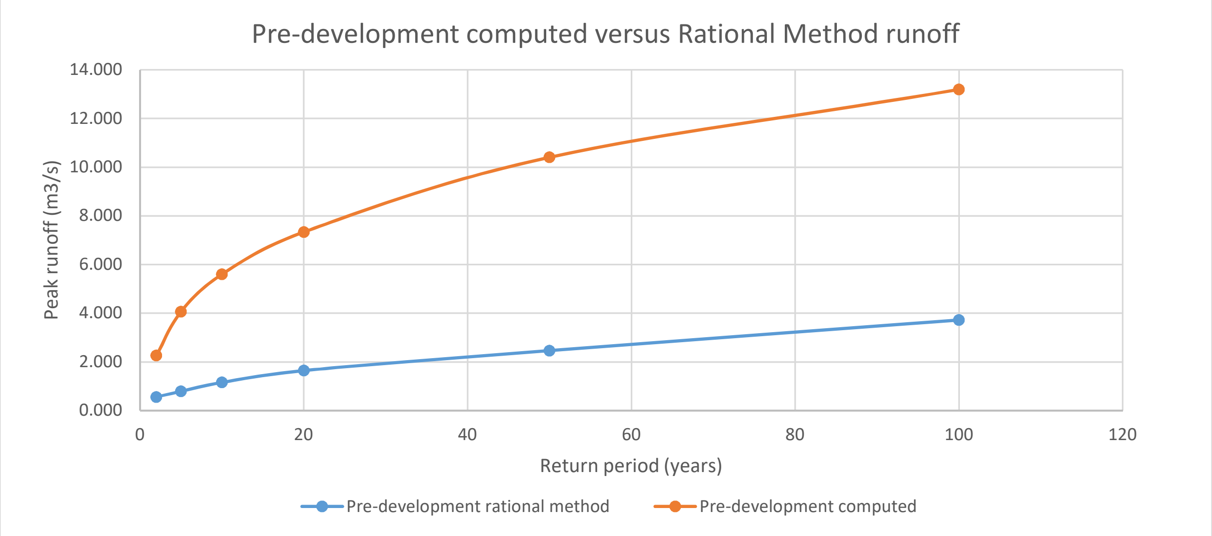


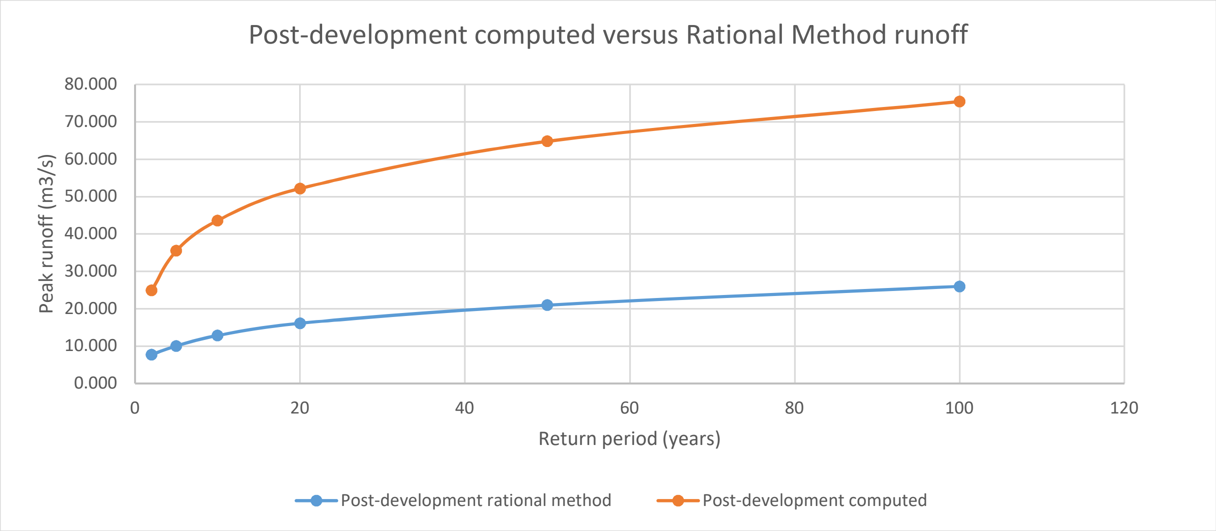
24h Point Rainfall (mm)			
T (yrs)	Mean (mm)	Lower (mm)	Upper (mm)
2	56.8	51.3	62.3
5	78.3	70.9	85.9
10	94.7	85.7	103.8
20	112.1	101.1	123.3
50	137.5	123.2	151.7
100	158.8	141.6	176.2
200	182.2	161.8	203.1

Continuous model			
Description of Catchment	Villa Lisa Ext 4 - Pre-Development		
Drainage into	Exiting Road and Stormwater drain		
Hydrological model parameters			
Model Parameter Description	Units	Value	
		Impervious	Pervious
Flow length	m	2050	
Imperviousness	%	0	100
Slope	%	0.1	
Manning's n		0.01	0.1
Depression Storage	mm	1.27	2.54
Green-Ampt Suction Head	mm	n/a	290.07
Green-Ampt Conductivity	mm/h	n/a	0.51
Green-Ampt Initial Moisture Deficit	fraction	n/a	0.228

Computed Design Storm Rainfall & Runoff				
SCS Type 3 Storm				
Recurrence Interval	**Probability of occurrence in one year**	**Rainfall (mm)**	**Pre-development Peak**	**Post-development Peak**
years	%	mm	m^3/s	m^3/s
2	50	56.8	2.261	24.91
5	20	78.3	4.061	35.49
10	10	94.7	5.595	43.58
20	4	112.1	7.335	52.17
50	2	137.5	10.41	64.8
100	1	158.8	13.19	75.41

Rational Method Design Runoff Estimation		
Return Period	**Pre-development Peak**	**Post-development Peak**
years	m^3/s	m^3/s
2	0.557	7.712
5	0.788	10.021
10	1.154	12.806
20	1.642	16.098
50	2.464	20.962
100	3.722	25.965





Appendix A6 - Conventional stormwater layout plan

Appendix A6 - Conventional stormwater layout plan

Appendix A6 - Conventional stormwater layout plan

Appendix A6 - Stormwater Design Guidelines

Design Criteria	Value
General	
Design Flood Recurrence interval (Minor Floods)	1 : 10
Design Flood Recurrence interval (Major Floods)	1 : 50
Pipe System	
Pipe Material	Concrete (SABS 677)
Pipe Joint	Interlocking
Manning's roughness coefficient	0.013
Minimum pipe size (mm)	450
Minimum pipe class for pipe sizes less than 525mm (Dia <525mm)	100D
Minimum pipe class for pipe sizes more than 525mm (Dia >525mm)	To be designed
Flow Depth ($^{d}/D$)	1.0
Maximum pipe velocity (m/s)	3.0
Minimum pipe slope (for 450mm diameter)	1 : 150
Manhole Material	Concrete (SABS 677)
Manhole chamber minimum height (mm)	1 000
Manhole chamber maximum height (mm)	1 500
Manhole chamber internal diameter (mm)	1 200
Manhole shaft internal diameter (mm)	675
Maximum manhole spacing (m)	100
Minimum kerb inlet size (m)	2,0
Canal System	
Canal Material	Concrete/ grassing
Manning's roughness coefficient	0.015
Maximum side slope of trapezoidal canal	1 : 1
Minimum freeboard (mm)	150
Maximum velocity (m/s)	5.0
Minimum gradient of channel	1 : 150
Roadways	
Maximum velocity in roadway (m/s)	3.0
Maximum gradient for class 4 roads (local distributor)	1:10 for a max length of 100m
Maximum gradient for class 5a roads (residential collector)	1:8 for a max length of 70m
Maximum gradient for class 5b - 5d roads (residential access roads)	1:5 for a max length of 50m
Maximum gradient for steep roads joining a crossroad	6% for a distance of at least 20m

Design Criteria	Value
Maximum cross gradient of sidewalks, excluding the erf access	1:3
Minimum gradient for all road classes	1 : 150
Maximum Encroachment on Roads during minor storms	
Residential and lower-order roads (Class:5a,5b,5c,5d,5e,5f)	No Kerb overtopping, flow to spread to crown
Residential access collector (Class 4)	No Kerb overtopping, flow spread must leave one lane free of water.
Local distribution (Class 3)	No Kerb overtopping, flow spread must leave one lane free of water in each direction.
Higher-order-roads (Class 1 , 2)	No Encroachment is allowed on any traffic lane
Encroachment during Major Storms	No Encroachment on adjacent properties

Tank Water End Use Demand Data for Residential 2			
Parameter	**Household Size (PPH)**	**Value (ℓ/day)**	**Source**
Toilet = 3 events/c/d Volume per event = 9ℓ/event	4	108	There exist one standard toilet in each household. According to Jacobs et al 2017 one flush event is equivalent to 9 ℓ.
Laundry = 0.143 events/c/d Volume per event = 62ℓ/event	4	35.464	Rudin (2008) cited by O'Brien (2014) indicates the laundry without a washing machine demand at 62 ℓ per wash. Jacobs et al 2017 indicates washing machine demand as 80 ℓ/event.
Cleaning = 2 events/c/d Volume per event = 6ℓ/event	4	48	Jacobs et al 2017 indicates a typical volume for kitchen sink events of 6ℓ/event which is adapted for this study, as it is the most realistic value.
Gardening = 1 events/c/d Volume per event = 5ℓ/event	N/A	5	Jacobs & Haarhoff (2004a) cited by O'Brien (2014) indicates an outside tap demand of 5 ℓ/event, which can be adopted for garden use. The gardens for the Res 2's are very small, making Jacobs et al 2017 estimate of 82 ℓ/day too large an estimate as it applies to irrigated areas of 75m2
Tank Water End Use Demand (ℓ/day)		196.464	

Tank Water End Use Demand Data for Residential 4			
Parameter	**Household Size (PPH)**	**Value (ℓ/day)**	**Source**
Toilet = 3 events/c/d Volume per event = 9ℓ/event	4	108	There exist one standard toilet in each household. According to Jacobs et al 2017 one flush event is equivalent to 9 ℓ.
Laundry = 0.143 events/c/d Volume per event = 62ℓ/event	4	35.464	Rudin (2008) cited by O'Brien (2014) indicates the laundry without a washing machine demand at 62 ℓ per wash. Jacobs et al 2017 indicates washing machine demand as 80 ℓ/event.
Cleaning = 2 events/c/d Volume per event = 6ℓ/event	4	48	Jacobs et al 2017 indicates a typical volume for kitchen sink events of 6ℓ/event which is adapted for this study, as it is the most realistic value.
Gardening = 1 events/c/d Volume per event = 1.092ℓ/day/m^2	N/A	1.092	The gardens for the Res 4's are on average 640m2, making Jacobs et al 2017 estimate of 573.3 ℓ/day/stand for irrigated areas of 525m2 a realistic estimate.

Res 4 Units	Units per block	Landscapeable area/unit	Landscapeable area/block	Gardening Demand(ℓ/day/block)	Tank Water End Use Demand (ℓ/day) per block
4029 (5 x 15 Units per block)	15	2.509	37.632	41.094	2913.054
4030 (1 x 9 Units per block)	9	2.459	22.133	24.170	1747.346
4030 (6 x 12 Units per block)	12	2.459	29.511	32.226	2329.794
4031 (1 x 9 Units per block)	9	2.542	22.880	24.985	1748.161
4031 (6 x 12 Units per block)	12	2.542	30.507	33.313	2330.881
4032 (1 x 9 Units per block)	9	2.483	22.347	24.403	1747.579
4032 (6 x 15 Units per block)	15	2.483	37.245	40.672	2912.632
4033 (2 x 9 Units per block)	9	2.487	22.386	24.445	1747.621
4033 (7 x 15 Units per block)	15	2.487	37.310	40.742	2912.702
4034 (11 x 9 Units per block)	9	2.532	22.792	24.889	1748.065
4035 (8 x 15 Units per block)	15	2.483	37.249	40.676	2912.636
4079 (2 x 9 Units per block)	9	2.486	22.376	24.435	1747.611
4079 (9 x 15 Units per block)	15	2.486	37.294	40.725	2912.685
4080 (1 x 9 Units per block)	9	2.551	22.958	25.070	1748.246
4080 (11 x 15 Units per block)	15	2.551	38.263	41.783	2913.743
4081(9 x 15 Units per block)	15	2.498	37.473	40.921	2912.881
4082 (7 x 15 Units per block)	15	2.501	37.521	40.973	2912.933
5524 (1 x 9 Units per block)	9	2.511	22.600	24.679	1747.855
5524 (2 x 12 Units per block)	12	2.511	30.133	32.905	2330.473
5524 (7 x 15 Units per block)	15	2.511	37.666	41.132	2913.092
5523 (1 x 9 Units per block)	9	2.547	22.922	25.031	1748.207
5523 (1 x 12 Units per block)	12	2.547	30.563	33.375	2330.943
5523 (7 x 15 Units per block)	15	2.547	38.204	41.718	2913.678
5522 (3 x 9 Units per block)	9	2.522	22.699	24.788	1747.964
5522 (7 x 12 Units per block)	12	2.522	30.266	33.050	2330.618
5521 (4 x 9 Units per block)	9	2.553	22.981	25.095	1748.271
5521 (4 x 15 Units per block)	15	2.553	38.302	41.825	2913.785
5520 (1 x 9 Units per block)	9	2.480	22.317	24.371	1747.547
5520 (13 x 12 Units per block)	12	2.480	29.757	32.494	2330.062
5520 (3 x 15 Units per block)	15	2.480	37.196	40.618	2912.578
5519 (1 x 9 Units per block)	9	2.486	22.372	24.430	1747.606
5519 (16 x 15 Units per block)	15	2.486	37.287	40.717	2912.677

Tank Water End Use Demand Data for Educational Facilities			
Parameter	**Non-potable end uses as a % of Demand**	**Value (ℓ/day)**	**Source**
Church Water Demand = 1184 ℓ/day	Landscaping = 28% Cooling & Heating = 11% Restrooms = 45%	728.16	Environmental Protection Agency (EPA) 2012. WaterSense at work: best management practices for commercial and institutional facilities, provides typical end uses for commercial and institutional facilities.
Creche Water Demand = 585 ℓ/day	Landscaping = 28% Cooling & Heating = 11% Restrooms = 45%	359.775	Environmental Protection Agency (EPA) 2012. WaterSense at work: best management practices for commercial and institutional facilities, provides typical end uses for commercial and institutional facilities.
Primary School Water Demand = 4048.65 ℓ/day	Landscaping = 28% Cooling & Heating = 11% Restrooms = 45%	2489.91975	The largest uses of non potable water in educational facilities are restrooms, landscaping, heating and cooling.

Tank Water End Use Demand Data for Office Buildings			
Parameter	**Non-potable end uses as a % of Demand**	**Value (ℓ/day)**	**Source**
Institutional Water Demand = 140340 ℓ/day	Landscaping = 22% Cooling & Heating = 28% Restrooms = 37%	122095.8	EPA (2012), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non potable water in office buildings are restrooms, heating and cooling, and landscaping.
Business Water Demand = 195344 ℓ/day	Landscaping = 22% Cooling & Heating = 28% Restrooms = 37%	169949.28	EPA (2012), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non potable water in office buildings are restrooms, heating and cooling, and landscaping.
Municipal Water Demand = 127550 ℓ/day	Landscaping = 22% Cooling & Heating = 28% Restrooms = 37%	110968.5	EPA (2012), WaterSense at work provides typical end uses for commercial and institutional facilities. The three largest uses of non potable water in office buildings are restrooms, heating and cooling, and landscaping.

Generalised Storage-Yield-Reliability relationships for the sizing of rainwater harvesting systems in South Africa

$S_{p-r} = \frac{Nr}{365.25} = \alpha(R_{SD})^{\beta}$		$R_{SD} = \frac{nxAxr_t}{D_t}$		$R_{TD-r} = \gamma e^{\varphi(S_{p-r})}$		$R_{TD-r} = \frac{K_r}{D_t x 365.25}$
Where:		Where:		Where:		Where:
$\alpha = 1.362(1-r)^{0.207}$		n = efficiency of RW collection = 0.8		$\gamma = 0.009e^{1.005(1-r)}$		K_r = Storage required to achieve reliability r
$\beta = 1.769e^{-0.445(1-r)}$		A = Roof Area contributing to runoff (m^2)		$\varphi = 8.207e^{-0.996(1-r)}$		D_t = Average annual demand
$0.5 \leq r \leq 0.95$		r_t = Average rainfall over period (mm) or MAP		$0.5 \leq r \leq 0.95$		
S_{p-r} = Yield ratio		MAP = 750 mm/year		R_{TD-r} = Storage-demand ratio		
N_r = Expected number of days of rainwater supply per year		D_t = Average non-portable demand		γ and φ = Coefficients of regression		
R_{SD} = Supply to demand ratio						
α and β = Coefficients of regression		FF= $ff_d x\, n_r$				
r = Reliability		FF = First Flush				
		ff_d = Desired first flush depth (0.2mm, 1.5mm or 3mm)				
		n_r = average number of rain days expected in a year (93 Days)				

Zoning	Area (Ha)	No. of Units	AADD (l/day/unit)	Non-potable AADD (l/day/unit)	Non-potable AADD (m^3/year)	Roof Area contributing to runoff (m^2)	$R_{SD} = \frac{nxAx(r_t - FF)}{D_t}$	$S_{p-r} = \alpha(R_{SD})^{\beta}$	$N_r = S_{p-r} x 365.25$	$R_{TD-r} = \gamma e^{\varphi(S_{p-r})}$	$K_r = R_{TD-r} x (D_t x 365.25)$	Kr = Storage required to achieve reliability r (*l*)
Residential 2	10.4554	1420	800	196.464	71.709	37.700	0.3076	0.2222	81.1746	0.0451	3.2320	3231.9652
Residential 2 (Duplex Units)	0.51	41	800	196.464	71.709	36.810	0.3004	0.2149	78.4742	0.0434	3.1150	3114.9568
Residential 4	**18.167**	**2175**	**600**									
4029 (5 x 15 Units per block)	0.6272	75	9000	2913.054	1063.265	282.155	0.1553	0.0844	30.8283	0.0227	24.0962	24096.1666
4030 (1 x 9 Units per block)	0.664	79	5400	1747.346	637.781	176.672	0.1621	0.0897	32.7612	0.0233	14.8402	14840.2469
4030 (6 x 12 Units per block)			7200	1747.346	637.781	229.414	0.2105	0.1298	47.4263	0.0284	18.1307	18130.6972
4031 (1 x 9 Units per block)	0.6864	82	5400	2329.794	850.375	176.672	0.1216	0.0597	21.7988	0.0200	17.0359	17035.9418
4031 (6 x 12 Units per block)			7200	2329.794	850.375	229.414	0.1579	0.0864	31.5568	0.0229	19.4642	19464.2159
4032 (1 x 9 Units per block)	0.8194	98	5400	1748.161	638.079	176.672	0.1620	0.0896	32.7395	0.0233	14.8428	14842.7856
4032 (6 x 15 Units per block)			9000	1748.161	638.079	282.155	0.2587	0.1739	63.5327	0.0354	22.6015	22601.5166
4033 (2 x 9 Units per block)	1.0198	122	5400	2330.881	850.772	176.672	0.1215	0.0596	21.7844	0.0200	17.0405	17040.5409
4033 (7 x 15 Units per block)			9000	2330.881	850.772	282.155	0.1941	0.1157	42.2737	0.0265	22.5423	22542.2565
4034 (11 x 9 Units per block)	0.8357	100	5400	1747.579	637.866	176.672	0.1621	0.0897	32.7550	0.0233	14.8410	14840.9741
4035 (8 x 15 Units per block)	0.9933	119	9000	2912.632	1063.111	282.155	0.1553	0.0844	30.8347	0.0227	24.0948	24094.7567
4079 (2 x 9 Units per block)	1.268	152	5400	1747.621	637.882	176.672	0.1621	0.0897	32.7539	0.0233	14.8411	14841.1052
4079 (9 x 15 Units per block)			9000	1747.621	637.882	282.155	0.2588	0.1740	63.5604	0.0354	22.6031	22603.1135
4080 (1 x 9 Units per block)	1.4795	177	5400	2912.702	1063.136	176.672	0.0972	0.0435	15.8891	0.0185	19.6470	19647.0225
4080 (11 x 15 Units per block)			9000	2912.702	1063.136	282.155	0.1553	0.0844	30.8336	0.0227	24.0950	24094.9912
4081(9 x 15 Units per block)	1.1242	135	9000	1748.065	638.044	282.155	0.2588	0.1740	63.5376	0.0354	22.6018	22601.8012
4082 (7 x 15 Units per block)	0.8755	105	9000	2912.636	1063.112	282.155	0.1553	0.0844	30.8346	0.0227	24.0948	24094.7687
5524 (1 x 9 Units per block)	1.1551	138	5400	1747.611	637.878	176.672	0.1621	0.0897	32.7541	0.0233	14.8411	14841.0733
5524 (2 x 12 Units per block)			7200	1747.611	637.878	229.414	0.2104	0.1298	47.4161	0.0284	18.1309	18130.9258
5524 (7 x 15 Units per block)			9000	1747.611	637.878	282.155	0.2588	0.1740	63.5610	0.0354	22.6031	22603.1439
5523 (1 x 9 Units per block)	1.0697	128	5400	2912.685	1063.130	176.672	0.0972	0.0435	15.8893	0.0185	19.6469	19646.9428
5523 (1 x 12 Units per block)			7200	2912.685	1063.130	229.414	0.1263	0.0630	23.0019	0.0204	21.6510	21650.9670
5523 (7 x 15 Units per block)			9000	2912.685	1063.130	282.155	0.1553	0.0844	30.8339	0.0227	24.0949	24094.9342
5522 (3 x 9 Units per block)	0.9332	111	5400	1748.246	638.110	176.672	0.1620	0.0896	32.7373	0.0233	14.8431	14843.0502
5522 (7 x 12 Units per block)			7200	1748.246	638.110	229.414	0.2104	0.1298	47.3917	0.0284	18.1315	18131.4743
5521 (4 x 9 Units per block)	0.8171	98	5400	2913.743	1063.516	176.672	0.0972	0.0435	15.8811	0.0185	19.6519	19651.8865
5521 (4 x 15 Units per block)			9000	2913.743	1063.516	282.155	0.1552	0.0844	30.8180	0.0227	24.0985	24098.4685
5520 (1 x 9 Units per block)	1.7358	208	5400	2912.881	1063.202	176.672	0.0972	0.0435	15.8877	0.0185	19.6479	19647.8572
5520 (13 x 12 Units per block)			7200	2912.881	1063.202	229.414	0.1263	0.0630	22.9997	0.0204	21.6518	21651.7746
5520 (3 x 15 Units per block)			9000	2912.881	1063.202	282.155	0.1553	0.0844	30.8309	0.0227	24.0956	24095.5878
5519 (1 x 9 Units per block)	2.0632	247	5400	2912.933	1063.221	176.672	0.0972	0.0435	15.8873	0.0185	19.6481	19648.1026
5519 (16 x 15 Units per block)			9000	2912.933	1063.221	282.155	0.1553	0.0844	30.8301	0.0227	24.0958	24095.7632
Church	0.1184	1	1184	728.16	265.778	148	0.3258	0.2411	88.0622	0.0495	13.1601	13160.0767
Creche	0.386	1	585	359.775	131.318	96.500	0.4300	0.3571	130.4297	0.0883	11.5965	11596.5200
Primary School	2.6991	1	4048.65	2489.920	908.821	674.775	0.4344	0.3623	132.3478	0.0907	82.3868	82386.8299
Institutional	0.9356	1	140340	122095.800	44564.967	163.730	0.0021	0.0002	0.0719	0.0149	663.5835	663583.4780
Business 2	2.4418	2	195344	169949.280	62031.487	3052.250	0.0288	0.0078	2.8352	0.0155	959.1846	959184.6452
Municipal	3.2977	11	127550	110968.500	40503.503	1594.375	0.0230	0.0057	2.0670	0.0153	619.7648	619764.8089
Total		1574										

	Residential 2					
$0.5 \leq r \leq 0.95$	$R_{SD} = \frac{nxAxr_t}{D_t}$	$S_{p-r} = \alpha(R_{SD})^{\beta}$	$N_r = S_{p-r} x 365.25$	$R_{TD-r} = \gamma e^{\varphi(S_{p-r})}$	$K_r = R_{TD-r} x (D_t x 365.25)$	Kr = Storage required to achieve reliability r (*l*)
0.5	0.3076	0.2222	81.1746	0.0451	3.2320	3231.9652
0.667	0.3076	0.1796	65.6063	0.0362	2.5981	2598.1157
0.8	0.3076	0.1448	52.9049	0.0291	2.0899	2089.9401
0.9	0.3076	0.1151	42.0222	0.0234	1.6775	1677.4866
0.95	0.3076	0.0953	34.8076	0.0199	1.4283	1428.2590

	Residential 2 (Duplex Units)					
$0.5 \leq r \leq 0.95$	$R_{SD} = \frac{nxAxr_t}{D_t}$	$S_{p-r} = \alpha(R_{SD})^{\beta}$	$N_r = S_{p-r} x 365.25$	$R_{TD-r} = \gamma e^{\varphi(S_{p-r})}$	$K_r = R_{TD-r} x (D_t x 365.25)$	Kr = Storage required to achieve reliability r (*l*)
0.5	0.3004	0.2149	78.4742	0.0434	3.1150	3114.9568
0.667	0.3004	0.1732	63.2586	0.0349	2.5016	2501.5840
0.8	0.3004	0.1394	50.8985	0.0281	2.0141	2014.1437
0.9	0.3004	0.1105	40.3574	0.0226	1.6216	1621.6365
0.95	0.3004	0.0914	33.3982	0.0193	1.3859	1385.8686

Generalised Storage-Yield-Reliability relationships for the sizing of rainwater harvesting systems in South Africa					
Zoning	**Area (Ha)**	**No. of Units**	**Kr = Storage required to achieve reliability r (l)**	**No. of blocks/units**	**Standard Size**
Residential 2	10.4554	1420	3232	1	1x3500l tank per unit.
Residential 2 (Duplex Units)	0.51	41	3115	1	1x3500l tank per unit.
4029 (5 x 15 Units per block)	0.6272	75	24096	5	2x10000l & 1x5000l tank per block.
4030 (1 x 9 Units per block)	0.664	79	14840	1	1x10000l & 1x5000l tank per block.
4030 (6 x 12 Units per block)			18131	6	2x10000l tank per block.
4031 (1 x 9 Units per block)	0.6864	82	17036	1	3x5000l & 1x2500 tank per block.
4031 (6 x 12 Units per block)			19464	6	2x10000l tank per block.
4032 (1 x 9 Units per block)	0.8194	98	14843	1	1x10000l & 1x5000l tank per block.
4032 (6 x 15 Units per block)			22602	6	2x10000l & 1x5000l tank per block.
4033 (2 x 9 Units per block)	1.0198	122	17041	2	3x5000l & 1x2500 tank per block.
4033 (7 x 15 Units per block)			22542	7	2x10000l & 1x5000l tank per block.
4034 (11 x 9 Units per block)	0.8357	100	14841	11	1x10000l & 1x5000l tank per block.
4035 (8 x 15 Units per block)	0.9933	119	24095	8	2x10000l & 1x5000l tank per block.
4079 (2 x 9 Units per block)	1.268	152	14841	2	1x10000l & 1x5000l tank per block.
4079 (9 x 15 Units per block)			22603	9	2x10000l & 1x5000l tank per block.
4080 (1 x 9 Units per block)	1.4795	177	19647	1	2x10000l tank per block.
4080 (11 x 15 Units per block)			24095	11	2x10000l & 1x5000l tank per block.
4081(9 x 15 Units per block)	1.1242	135	22602	9	2x10000l & 1x5000l tank per block.
4082 (7 x 15 Units per block)	0.8755	105	24095	7	2x10000l & 1x5000l tank per block.
5524 (1 x 9 Units per block)	1.1551	138	14841	1	1x10000l & 1x5000l tank per block.
5524 (2 x 12 Units per block)			18131	2	2x10000l tank per block.
5524 (7 x 15 Units per block)			22603	7	2x10000l & 1x5000l tank per block.
5523 (1 x 9 Units per block)	1.0697	128	19647	1	2x10000l tank per block.
5523 (1 x 12 Units per block)			21651	1	2x10000l & 1x2500l tank per block.
5523 (7 x 15 Units per block)			24095	7	2x10000l & 1x5000l tank per block.
5522 (3 x 9 Units per block)	0.9332	111	14843	3	1x10000l & 1x5000l tank per block.
5522 (7 x 12 Units per block)			18131	7	2x10000l tank per block.
5521 (4 x 9 Units per block)	0.8171	98	19652	4	2x10000l tank per block.
5521 (4 x 15 Units per block)			24098	4	2x10000l & 1x5000l tank per block.
5520 (1 x 9 Units per block)	1.7358	208	19648	1	2x10000l tank per block.
5520 (13 x 12 Units per block)			21652	13	2x10000l & 1x2500l tank per block.
5520 (3 x 15 Units per block)			24096	3	2x10000l & 1x5000l tank per block.
5519 (1 x 9 Units per block)	2.0632	247	19648	1	2x10000l tank per block.
5519 (16 x 15 Units per block)			24096	16	2x10000l & 1x5000l tank per block.
Church	0.1184	1	13160	1	2x10000l & 3x3500l tank per block.
Creche	0.386	1	11597	1	2x10000l & 1x2500l tank per block.
Primary School	2.6991	1	82387	1	8x10000l & 1x2500l tank per block.
Institutional	0.9356	1	663583	1	5x20000l
Business 2	2.4418	2	959185	2	5x20000l
Municipal	3.2977	11	619765	11	5x10000l tank per block.
Total		1574			

Appendix A10 - WSUD layout plan

Appendix A10 - WSUD layout plan

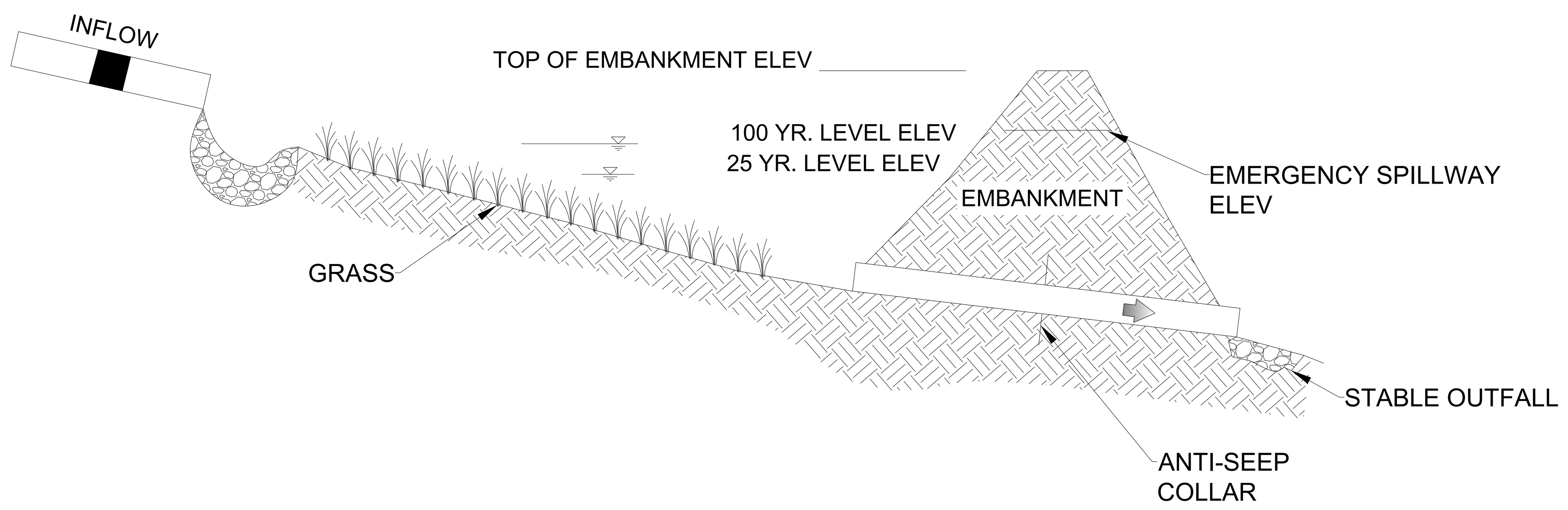
Appendix A10 - WSUD layout plan

Appendix A10

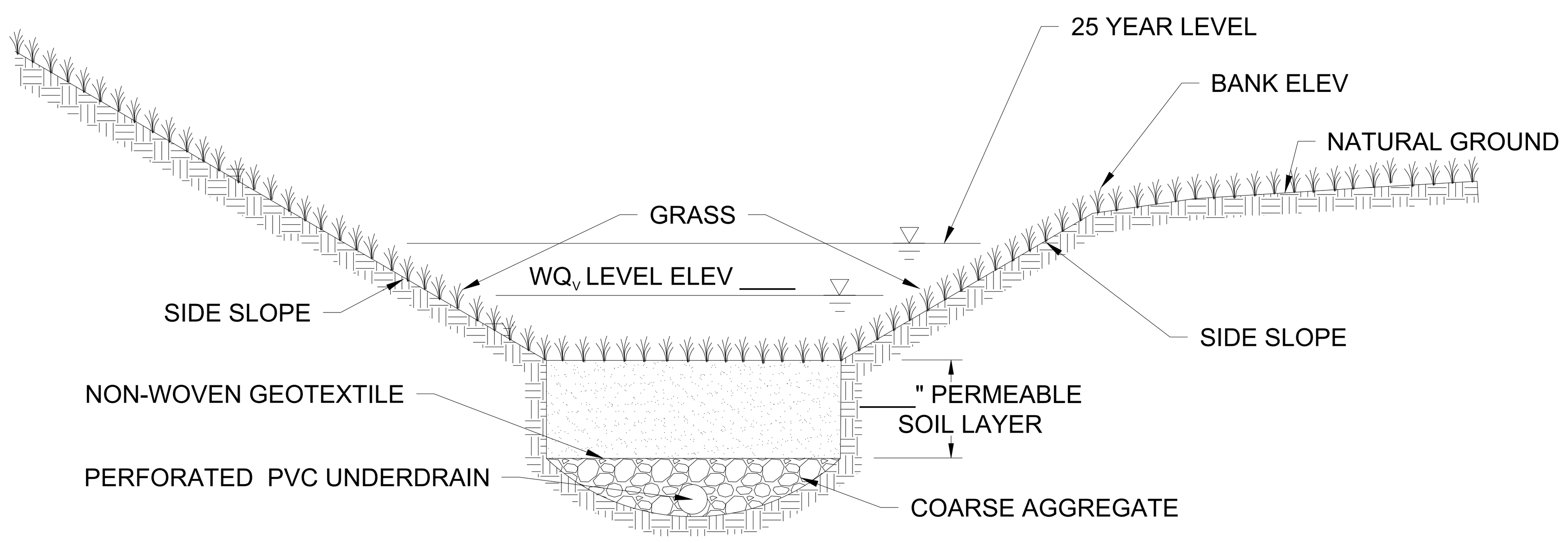
Typical details cross section

NOTES

1. PROVIDE A MINIMUM OF 1.0' OF FREEBOARD BETWEEN EMERGENCY SPILLWAY AND TOP OF EMBANKMENT.
2. PROVIDE A MINIMUM OF 0.1' BETWEEN 100 YR. LEVEL AND EMERGENCY SPILLWAY.



DETENTION POND PROFILE



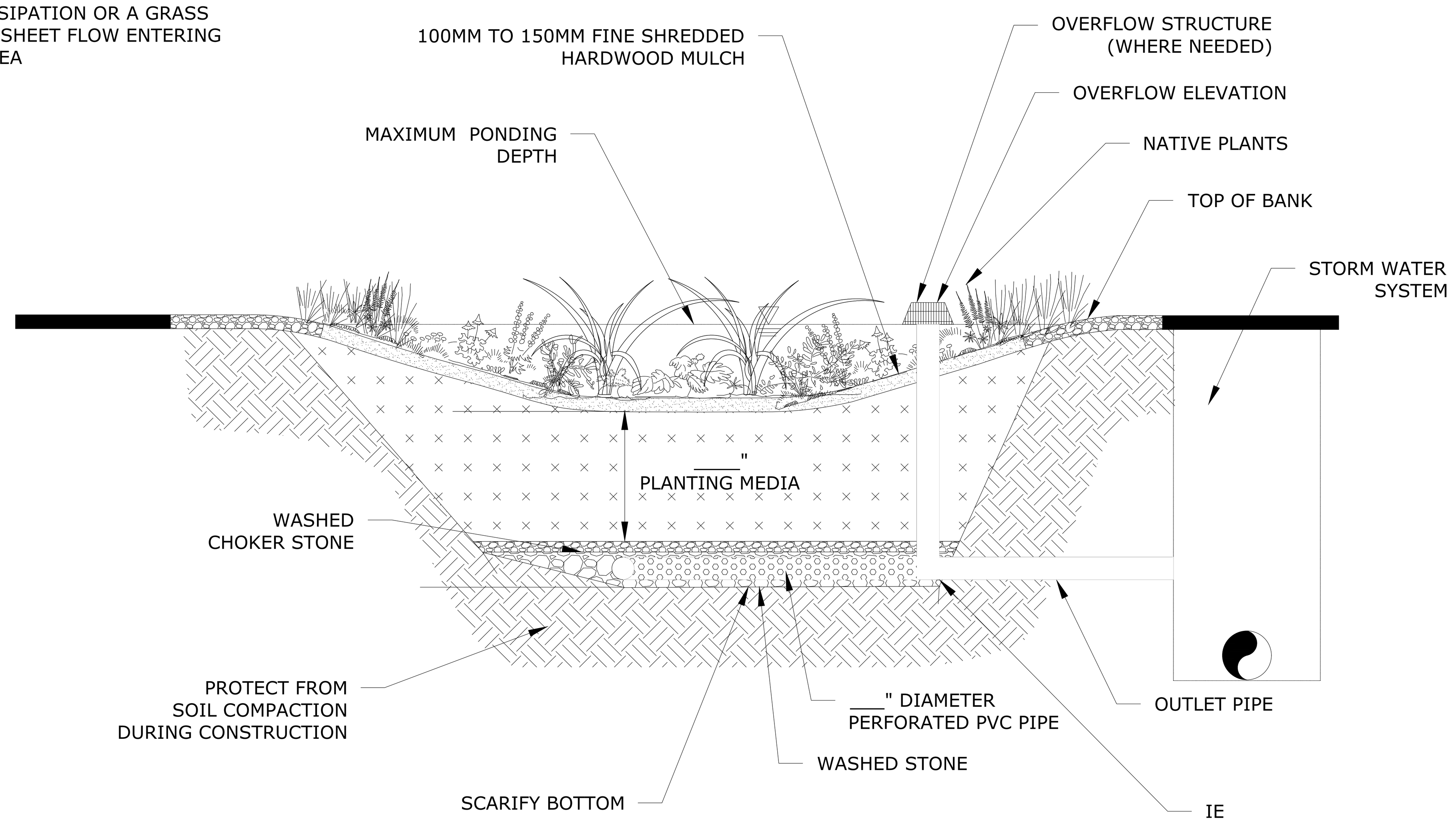
VEGETATED SWALE
TYPICAL SECTION
N.T.S.

NOTES

1. LONGITUDINAL SLOPE MUST BE <4%.
2. MIN. 6" FREEBOARD ABOVE 25-YR STORM WATER SURFACE ELEVATION
3. MAXIMUM SIDE SLOPE IS 2:1.

NOTES:

1. MAXIMUM 3:1 SIDE SLOPES
2. HAVE ENERGY DISSIPATION OR A GRASS FILTER STRIP FOR SHEET FLOW ENTERING BIORETENTION AREA



BIORETENTION WITH UNDERDRAIN

Project Costs				
Item	Capital Cost	Annual Maintenance	Maintenance Cost (**Annuity Present Value**)	Grand total
	2019	2019	Rate: 6.5% Design life: 40 years	
Conventional system				
Conventional water	R 16,289,588.00	R 286,539.18	R 4,053,247.67	R 20,342,835.67
Conventional stormwater	R 64,914,140.92	R 277,176.00	R 3,920,800.55	R 68,834,941.48
			Grand total	R 89,177,777.15
WSUD				
RWH	R 19,907,365.00	R 668,726.44	R 9,459,487.82	R 29,366,852.82
Bio-retention	R 6,423,955.84	R 174,438.05	R 2,467,518.12	R 8,891,473.96
Vegetated swales	R 1,758,881.08	R 350,024.95	R 4,951,287.38	R 6,710,168.45
Detention pond	R 6,111,021.58	R 231,786.06	R 3,278,735.94	R 9,389,757.51
Conventional water	R 16,289,588.00	R 286,539.18	R 4,053,247.67	R 20,342,835.67
WSUD stormwater pipe network	R 45,599,838.35	R 232,650.50	R 3,290,963.90	R 48,890,802.24
			Grand total	R 123,591,890.67

$$PV = AF_T \text{ x } AN$$

$$AF_T = \frac{(1+r)^T - 1}{r(1+r)^T}$$

Where:

PV = Present Value

AF_T = Annuity Factor

AN = Annuity

r = Nominal interest rate

T = Time period

Benefits			
Item	Monthly savings	Annual Savings	Annual Savings (**Annuity Present Value**)
	2019	2019	Rate: 6.5% Period: 40 years
Savings in water consumption	R 777,480.94	R 9,329,771.32	R 131,974,530.89

RAINWATER HARVESTING				
Description	Unit	Quantity	Rate	Amount
Supply and install 2500 litre rain water tank (including pump)	No.	22	R 4,640.00	R 102,080.00
Supply and install 3500 litre rain water tank (including pump)	No.	1461	R 6,174.10	R 9,020,360.10
Supply and install 5000 litre rain water tank (including pump)	No.	124	R 7,242.75	R 898,101.00
Supply and install 10000 litre rain water tank (including pump)	No.	382	R 18,888.16	R 7,215,278.65
Supply and install 20000 litre rain water tank (including pump)	No.	15	R 44,968.85	R 674,532.75
Supply and install First Flush device	No.	1574	R 1,268.75	R 1,997,012.50
Capital Cost				**R 19,907,365.00**
Annual Maintenance				
Inspections	No.	1574	R 66.00	R 103,884.00
First flush – cleaning requires inspection	No.	1574	R 145.67	R 229,284.58
Gutter and Roof cleaning	m^2	2629	R 122.84	R 322,946.36
Sediment removal	m^3	110	R 114.65	R 12,611.50
Total annual maintenance				**R 668,726.44**

BIO-RETENTION				
Description	Unit	Quantity	Rate	Amount
Clear and remove topsoil	m^2	20326.8	R 12.00	R 243,921.96
Cut to spoil	m^3	10163.4	R 135.00	R 1,372,061.03
300x300 stone drain covered in Geofabric (110 mm drainex pipe)	m	2032.7	R 260.00	R 528,497.58
Backfilling with selected material	m^3	3049.0	R 196.00	R 597,608.80
Top soil supplied by contractor, Spread in 100-200 mm thick layers	m^3	4065.4	R 260.00	R 1,056,995.16
Plants supplied & planted	m^2	12196.1	R 75.00	R 914,707.35
Supply and add mulch to shrub areas (20 mm)	m^2	12196.1	R 98.00	R 1,195,217.60
Top soiling of verge areas	m^2	1355.1	R 10.00	R 13,551.22
Grassing	m^2	1355.1	R 82.00	R 111,120.00
Irrigation	m^2	12196.1	R 32.00	R 390,275.14
Capital Cost				**R 6,423,955.84**
Annual Maintenance				
Inspections	No.	32	R3,450.00	R 110,400.00
Litter & vegetation management (based on quarterly visits)	visit.m^2	32	R 5.00	R 160.00
Grass cutting	m^2	1355.1	R 25.00	R 33,878.05
Sediment removal	m^3	250	R 120.00	R 30,000.00
Total annual maintenance				**R 174,438.05**

VEGETATED SWALE				
Description	Unit	Quantity	Rate	Amount
Clear and grub	m^2	10698.8	R 8.00	R 85,590.32
Strip and remove topsoil	m^3	1069.9	R 45.00	R 48,144.56
Cut to spoil	m^3	5349.4	R 135.00	R 722,168.33
Trimming side drains to profile, compact	m	2139.8	R 45.00	R 96,289.11
Levelling verges	m	2139.8	R 35.00	R 74,891.53
Grassing	m^2	6419.3	R 82.00	R 526,380.47
Irrigation	m^2	6419.3	R 32.00	R 205,416.77
Capital Cost				**R 1,758,881.08**
Annual Maintenance				
Inspections	No.	32.0	R3,450.00	R 110,400.00
Litter & vegetation management (based on quarterly visits)	visit.m^2	32.0	R 5.00	R 160.00
Erosion	m^2	2567.7	R 55.00	R 141,224.03
Grass cutting	m^2	3209.6	R 25.00	R 80,240.93
Sediment removal	m^3	150.0	R 120.00	R 18,000.00
Total annual maintenance				**R 350,024.95**

DETENTION POND				
Description	Unit	Quantity	Rate	Amount
Clear and grub	m^2	9704.9	R 8.00	R 77,639.20
Cut to fill	m^3	7278.675	R 65.00	R 473,113.88
Excavate detention ponds 1-2m deep	m^3	14557.35	R 30.00	R 436,720.50
Surface bed preparation for bedding of gabions	m^2	1940.98	R 120.00	R 232,917.60
Gabions (2.0 x 1.0 x 1.0) PVC coated gabion boxes 2,7mmm diameter galvanised wire, to SANS 1580, including rock infill	m^3	970.49	R 2,200.00	R 2,135,078.00
Geotextile (Filter Fabric - Bidim)	m^2	1940.98	R 35.00	R 67,934.30
Reno mattresses (3.0 x 1.0 x 0.3PVC boxes)	m^3	970.49	R 2,700.00	R 2,620,323.00
Pond inlet/outlet	No.	2	R38,500.00	R 77,000.00
Grassing	m^2	1940.98	R 35.00	R 67,934.30
Capital Cost				**R 6,111,021.58**
Annual Maintenance				
Inspections	No.	32	R 3,450.00	R 110,400.00
Litter & vegetation management (based on quarterly visits)	visit.m^2	32	R 5.00	R 160.00
Erosion	m^2	776.392	R 55.00	R 42,701.56
Grass cutting	m^2	1940.98	R 25.00	R 48,524.50
Sediment removal	m^3	250	R 120.00	R 30,000.00
Total annual maintenance				**R 231,786.06**

WATER RETICULATION NETWORK								
		UNIT COST PER METRE FOR WATER PIPES WITH THE FOLLOWING DAIMETERS						
Description	Unit	110Ø	160Ø	200Ø	250Ø	315Ø	355Ø	400Ø
Length	m	**5257**	**1169**	**402**	**270**	**506**	**330**	**1258**
Site Clearance	R/m	20	20	20	20	20	20	20
Excavate and Backfill	R/m	120	120	120	120	120	120	150
Rock Excavation	R/m	300	300	300	300	300	300	300
Bedding (Class B)	R/m	80	80	80	80	80	80	100
Supply, Lay and Test pipe	R/m	104	220	331	538	1131	2000	2434
Supply, Lay and Test Valve	R/m	300	450	600	750	1085	1580	1950
Sub-Total	**R/m**	**924**	**1190**	**1451**	**1808**	**2736**	**4100**	**4954**
Capital Cost	R	4,857,468	1,391,110	583,302	488,160	1,384,416	1,353,000	6,232,132
TOTAL COST ESTIMATE		**R 16,289,588.00**						

Annual Maintenance				
Description	Unit	Quantity	Rate	Amount
Inspections	No.	2	R 22,274.25	R 44,548.50
Scouring	No.	4	R 7,450.67	R 29,802.68
Pressure testing	No.	2	R 23,750.00	R 47,500.00
Sediment removal	m^3	320	R 514.65	R 164,688.00
Total annual maintenance				**R 286,539.18**

STORMWATER NETWORK										
		UNIT COST PER METRE FOR STORMWATER PIPES WITH THE FOLLOWING DAIMETERS								
Description	Unit	450Ø	525Ø	675Ø	900Ø	1050Ø	1200Ø	1500Ø	1650Ø	1950Ø
Length	m	**5864**	**1591**	**1514**	**130**	**544**	**782**	**585**	**292**	**73**
Site Clearance	R/m	20	20	20	20	20	20	20	20	20
Excavate and Backfill	R/m	200	200	200	250	250	280	310	310	335
Rock Excavation	R/m	300	300	300	300	300	300	300	300	300
Bedding (Class B)	R/m	150	150	150	180	180	220	250	250	275
Supply, Lay and Test pipe	R/m	1850	3200	3680	4232.0	4866.8	5596.8	6436.3	7401.8	7771.9
Supply, Lay and Test Manholes & KI's	R/m	1420	1680	1932	2221.8	2555.1	2938.3	3379.1	3885.9	4468.8
Sub-Total	**R/m**	**3940**	**5550**	**6282**	**7204**	**8172**	**9355**	**10695**	**12168**	**13171**
Capital Cost	R	23,104,160	8,830,050	9,510,948	936,494	4,445,497	7,315,728	6,256,822	3,552,979	961,462
TOTAL COST ESTIMATE		**R 64,914,140.92**								

Annual Maintenance				
Description	Unit	Quantity	Rate	Amount
Inspections	No.	2	R 22,274.25	R 44,548.50
Kerb inlets	No.	2	R 26,250.00	R 52,500.00
Sediment removal	m^3	350	R 514.65	R 180,127.50
Total annual maintenance				**R 277,176.00**

WSUD STORMWATER NETWORK										
		UNIT COST PER METRE FOR STORMWATER PIPES WITH THE FOLLOWING DAIMETERS								
Description	Unit	450Ø	525Ø	675Ø	900Ø	1050Ø	1200Ø	1500Ø	1650Ø	1950Ø
Length	m	**4231.36**	**467.36**	**740.98**	**130**	**544**	**782**	**585**	**144.94**	**73**
Site Clearance	R/m	20	20	20	20	20	20	20	20	20
Excavate and Backfill	R/m	200	200	200	250	250	280	310	310	335
Rock Excavation	R/m	300	300	300	300	300	300	300	300	300
Bedding (Class B)	R/m	150	150	150	180	180	220	250	250	275
Supply, Lay and Test pipe	R/m	1850	3200	3680	4232.0	4866.8	5596.8	6436.3	7401.8	7771.9
Supply, Lay and Test Manholes & KI's	R/m	1420	1680	1932	2221.8	2555.1	2938.3	3379.1	3885.9	4468.8
Sub-Total	**R/m**	**3940**	**5550**	**6282**	**7204**	**8172**	**9355**	**10695**	**12168**	**13171**
CONSTRUCTION COST	R	16,671,558	2,593,848	4,654,836	936,494	4,445,497	7,315,728	6,256,822	1,763,592	961,462
TOTAL COST ESTIMATE		**R 45,599,838.35**								

Annual Maintenance				
Description	Unit	Quantity	Rate	Amount
Inspections	No.	2	R 22,274.25	R 44,548.50
Kerb inlets	No.	2	R 22,000.00	R 44,000.00
Sediment removal	m^3	280	R 514.65	R 144,102.00
Total annual maintenance				**R 232,650.50**