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IDENTIFYING THE CAUSES OF FLOODING ON A NATIONAL ROUTE IN EKURHULENI, SOUTH AFRICA

by

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A research report submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science.

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DECLARATION

I, Kudzai Muyambo, declare that this research proposal report is my own work and that I have:

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ABSTRACT

Changing rainfall patterns and intensity are often considered as the main causes of flooding events. Consequently, detailed investigations into other likely causes of flooding are often not carried out. The increased frequency of flooding events and their associated impacts, such as disruption and the loss of both life and property, highlight the need for comprehensive evaluation of the causes of urban flooding.

The purpose of the study was to investigate the causes of flooding at the South African National Route-12 (N12), section-19, between the N12/R24 and Gillooly's Interchanges located in Johannesburg. This followed repeated flooding at the study area most notably on the 2nd of March 2017, 21st of January 2016 and 9th November 2016. The study determined the return period of the rainfall which led to these flood events using a previously developed regional approach of design rainfall estimation. The analysis revealed that only the rainfall event of the 9th of November 2016 exceeded the prescribed design return period of 1:80 years for the drainage structure. Further, hydraulic modelling using the HEC-RAS software package found that the current and older configurations of the drainage structure are inadequate for the recommended 80-year flood, but the upgrades improve capacity by 87.4%.

The findings of the study were that the flooding was a result of the inadequate size of the drainage structure. The structure was inadequate for the 2-year recurrence interval flood, far less than the 80-year recurrence interval required for the Class-1 Road. Photographic images showed increased land development (~6%) within the catchment which was estimated to have increased the peak flood by 24%. Lapses in maintenance of the drainage structure were also observed. The intervention to raise the headwall and wingwalls at the inlet to the structure, implemented in 2019, reduces the instances of overtopping of the road but does not mitigate flooding in the surrounding properties and is therefore not holistic. This study provides a methodology to determine the causes of flooding in urban settings and can aid authorities as they seek to understand and address flooding hotspots in urban areas.

DEDICATION

Dedicated to my younger brother, Mukundi.

ACKNOWLEDGEMENTS

I would like to thank Prof. Ndiritu for his valuable support and guidance throughout this research. Your stimulating suggestions and your encouragement moved me out of my comfort zone. This was a never-ending project and finally it is complete!

To my colleague, Mr Maisera, thank you for the several ways you have contributed to my professional growth.

I would like to thank the South African Weather Service for making available the rainfall data for the flooding events analysed in the research.

I want to extend my deepest gratitude to my family for your sincere support and prayers, without you, I would not have made it this far. To my dad and sister, thank you for pushing me beyond my own beliefs, consistently. To my mum, thank you for supporting me throughout my studies. Special thanks to Lala for her patience whilst I worked on this report.

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LIST OF ACRONYMS

DEA	Department of Environmental Affairs
NAP	National Adaptation Plan
SANRAL	The South African National Road Agency
Stats SA	Statistics South Africa
SADC	Southern African Development Community
EMM	Ekurhuleni Metropolitan Municipality
IDP	Integrated Development Plan
TRH	Technical Recommendations for Highways
RLMA	Regional L-Moment Algorithm
T_C	Time of Concentration
MAP	Mean Annual Precipitation
IDF	Intensity-Duration-Frequency also known as the Depth-Duration-Frequency
RLMA	Regional L-Moment Algorithm
Q_D	Peak Flood
SDF	Standard Design Flood
WSUD	Water Sensitive Urban Design
DRAINAGE MANUAL/ SANRAL DRAINAGE MANUAL	The South African Road Agency Limited- Drainage Manual, 5th Edition, Fully Revised.
IPCC	Intergovernmental Panel on Climate Change

HRU	Hydrological Research Unit
WMO	World Meteorological Organisation
EG PF	Energy Gradient Profile
WS PF	Water Surface Profile
CRIT PF	Critical Profile
JRA	Johannesburg Roads Agency

CHAPTER 1

Introduction to the Research

1.1 Overview

This chapter presents the research question, aims and research objectives. The research method is also outlined and an overview of the chapters of the research report is provided.

1.2 Introduction

"Flood-related catastrophes have increased by 134% since 2000, compared with the two previous decades, according to the World Meteorological Organization (WMO)" (UNEP, 2021). Road networks facilitate the movement of goods and people and when these networks are disrupted, there are significant social and economic impacts. To mitigate against these impacts requires managing flood risks. These management efforts should be informed by an understanding of the complex interaction of the human and natural factors which contribute to urban flooding.

Weather patterns and the change in the intensity and frequency of rainfall events alters the catchment response to the event and impacts on the reliability of designed hydraulic systems. Ndiritu (2005), revealed that annual maxima of 1–7-day duration have been increasing in South Africa and highlighted that it may not be justified to ignore these trends without verifying their insignificance.

The alteration of natural streams in urban areas increases the vulnerability to flooding, particularly for downstream settlements. Piketh, et al., (2014), showed that within Ekurhuleni Metropolitan Municipality (EMM) Gauteng, streams are routed underground into stormwater drains, e.g., Nigel, Alberton, and Springs. This makes the drainage systems acutely sensitive to any lapses in maintenance where debris build-up and blockage of these streams could result in severe flooding.

Significant land use in urban areas typically consists of the development of malls, residential and commercial complexes. These alter catchment characteristics through the covering and compaction of the natural ground when constructing roads, buildings and parking areas, resulting in reduced infiltration and increased runoff volumes. Some paving materials, roofs and storm network systems result in increased peak flows by reducing the time to peak. EMM requires that runoff mitigation measures be implemented during the approval processes of such developments (Ekurhuleni Metropolitan Municipality, 2015). However, the focus of these town planning regulations is primarily on attenuating the peak flow and not necessarily runoff volumes. The result is that in the absence of effectively managed catchments, *attenuation systems often have limited impact on overall urban catchment flood responses and can even exacerbate flood impacts* (Piketh, et al., 2014).

Pregnolato, et al., (2017) found that existing approaches fail to capture the interactions between floodwater and the transport system, noting that, "*typically a road is considered either fully operational or fully blocked which is not supported by observations"*. These observations refer to the relationship between the depth of floodwater and the disruption of traffic which the referenced study expressed as vehicle speed. The study also states that the interaction between flooding and the socio-economic impacts of "*nuisance flooding*" i.e., where there are low levels of inundation, particularly in urban areas, may not be fully acknowledged.

The contributory factors to and impacts of flooding discussed above give impetus for establishing a systematic approach for identifying the causes of urban flooding so that they may be effectively addressed. This research investigates the causes of flooding for an appropriately selected case study site. The appropriateness is based on the importance of the drainage structure at the selected site and the vulnerability of the site to flooding based on the reported occurrences of flooding.

The outcomes of this study may be valuable to catchment management authorities and urban authorities in understanding and addressing flooding hotspots in urban areas.

1.3 Problem Statement

This study sets out to identify the causes of flooding at section 19 of the N12 between the N12/R24 and Gillooly's Interchanges in the Gauteng province of South Africa (the study area). This is in response to the repeated flooding, most notably on the following dates: the 2nd of March 2017 (eNCA, 2017), the 21st of January 2016 (EWN, 2016) and the 9th November 2016 (EWN, 2016).

The chosen study area, Section 19 of the N12 between the N12/R24 and Gillooly's Interchanges has national significance, and its selection is further justified as follows:

- The road section has been subject to repeated flooding occurrences.
- Due to its management by the South African National Road Agency (SANRAL), the highest road management body in the country, the design philosophy that would have been followed at the study area is representative of the national norms, broadening the significance of the research findings.
- The N12's classification as a national route validates the assumption that the guidelines of the SANRAL Drainage Manual (2006) would have been applied. This provides a basis to review the design of the drainage structures and the recommended design methods of the SANRAL Drainage Manual.
- Modifications consisting of raising the wingwalls and the headwall, upstream of the structure, were implemented in 2019. The influences of these modifications on the adequacy of the drainage structure are assessed in this study to provide context to maintenance and operational issues at the study area.
- The selected site provides an opportunity to investigate how national (SANRAL) and local (EMM) authorities collaborate to manage drainage infrastructure within a shared catchment.

There was inadequate information available to investigate the more significant Gillooly's interchange which is just 1.9km West of the study location. The Gillooly's interchange also experienced flooding on the previously mentioned dates but it is characterised by a drainage network consisting of both open channel flow and piped networks. Information on the pipe networks was unavailable for this study and the cost required to survey the pipe networks for an adequate analysis was beyond the scope of this master's level study.

In the immediate aftermath of a flooding event, discourse around climate change and the recency of the rainfall event often lead to a narrow focus on the precipitation event alone, as the cause of the flooding. DuPlessis & Burger (2015) concluded that a lack of reliable rainfall data in South Africa has contributed to the absence of significant evidence supporting increasing rainfall intensities. Despite the close relation between the climate change phenomenon and changing rainfall trends and intensity, determining the impacts of climate change is beyond the scope of this master's research. This study will contextualise its findings based on current literature to highlight the potential influence of climate change on the reliability of flood drainage structures.

To evaluate the impact of climate change at the study area would require a considerable amount of data and modelling which may not necessarily be representative of the study area in this research. Piketh, et al., (2014) found that, "*sophisticated bias correction methods, that relate the probability density function of the simulated data to that of rainfall observations at the point scale, are needed to obtain simulations that are useful to force hydrological models that are calibrated at the point scale*."

Further, the linkages between climate change and how it impacts the hydrological cycle, in particular, rainfall patterns (intensity and frequency) and catchment characteristics (attenuation and storage), are not wholly understood yet according to the reviewed literature (Mackellar, et al., (2014) and Kruger (2006 cited in; MacKellar, et al., 2014) and FAO (1996)). Most studies investigating climate change impacts focus on the influence of climate change on rainfall patterns and not necessarily the entire hydrological cycle. Rind, et al., (1990) highlighted the inadequacy of general circulation models to simulate the land phase of the hydrological cycle satisfactorily.

1.4 Research Question

What are the causes of repeated flooding at section 19 of the N12 between the N12/R24 and Gillooly's Interchanges?

1.4.1 *Research Aims*

Based on the research question above, the project had the following aims:

Aim 1: To determine the adequacy of the currently installed drainage structure and assess whether the adequacy of the drainage structure is consistent with the experienced flooding events.

Aim 2: To assess, based on the case study, the impact of various states of maintenance and catchment management practices on the reliability of flood drainage structures.

Aim 3: To contextualise the findings of the study within a wider geographic context and in consideration of the impacts of climate change.

1.5 Research Objectives

To answer the research question, the following research objectives have been developed:

Objective 1: Identify from literature, mutually dependent parameters with an influence on the design and hydraulic reliability of drainage structures. Assess the influence of these identified parameters at the study area. *(This Objective is Linked to Research Aim 1 and 2)*

Objective 2: Assess the adequacy of the case study drainage structure by comparing its capacity to the design peak flow for the site based on the guidelines of the SANRAL Drainage manual. *(This Objective is Linked to Research Aim 1)*

Objective 3: For the reported flooding events, estimate the return period of the rainfall events and assess whether flooding would have been expected from these events. Assess the probable contribution of climate change to the severity of the rainfall events and what the impact of including factors of safety of climate change would have on the adequacy of the structure *(This Objective is Linked to Research Aim 1,2 and 3)*

Objective 4: Assess the state of maintenance at the study area and the alterations or changes in land-use within the catchment area. Estimate the corresponding influence of these on the peak flow and the adequacy of the drainage structure. Assess the flood mitigation impact of recent alterations to the drainage structure. *(This Objective is Linked to Research Aim 2 and 3)*

Objective 5: Broaden the significance of the analyses in objectives 1 to 4 by assessing the applicability of these analyses results to other locations with similar characteristics. *(This Objective is Linked to Research Aim 3)*

1.6 Research Method

The approach taken in this study is to initially determine the required drainage capacity(m^3/s) at the study area (determined using the SANRAL drainage manual). This theoretical capacity is compared to the available capacity of the currently installed drainage structure at the study area to assess adequacy. The return periods of the rainfall events corresponding to the flooding are determined and the associated peak floods are calculated for comparison with the previously computed theoretical and available peak floods values. This is carried out to assess the influence of the rainfall events on the flooding experienced at the study area.

The causes of the flooding are investigated further, guided by the outcome of the adequacy assessment. Of particular interest are flooding events which occur despite theoretical adequacy of drainage structures to pass the storm event. These would point to limitations in the drainage design which could be attributed to the selection or limitations of hydrological models and their associated parameters, and the limitations in the understanding of the interaction of the precipitation event with the wider catchment.

It is also essential that when a drainage structure under analysis fails the adequacy test, further investigations are made into the initial design to understand the validity of the design parameters such as catchment size and to eliminate errors in the application of established methods. This systematic approach to investigating flooding events is not always common in practice and betrays the widespread acknowledgement in literature (SANRAL (2006), Kite, et al., (1994) that mutually dependent parameters (topographic factors, climatological variables, and developmental influences) impact run-off.

A presentation of the methodology is provided in Figure 1.1 below. The research method combines complementary aspects of the exploratory and confirmatory research approaches.

1. Identification of Mutually Dependent Parameters from Literature

Runoff Estimation | Catchment Surface Characteristics | Risk/Sizing

Research Aim 1 and 2 can be contextualised by carefully analysing literature to understand the relationships amongst the different parameters and their influence on the sizing and hydraulic reliability of drainage structures.

2. Data Collection and Assessment of Drainage Structure Adequacy

Catchment Hydrological and Hydraulic Data | Climatic Data | Drainage Design Methods

Research Aim 1 can be partially fulfilled by carrying out an engineering design based on the SANRAL Drainage Manual and comparing the design results with the measured size and capacity at the study area to evaluate the adequacy of the currently installed drainage structures.

3. Categorisation of Rainfall Events Responsible for the Recorded Flooding

Rainfall Event Data | Flooding Reports | Parliamentary Reports Outlining Interventions by SANRAL

Research Aim 1 can be partially fulfilled by evaluating the return periods of the experienced rainfall events against the available capacity determined in 1 above. The categorisation by return period, of the rainfall event data associated with the flooding events allows for the contextualisation of the impact of extreme events at the study area and by extension the impact of climate change as contemplated in research Aim 3.

4. Assessment of Land Use Changes and their Influence on the Hydraulic Reliability of the Drainage Structures

Repeat Photography

Estimate Change in Peak Flow from Land Use Changes

Research Aim 2 can be partially fulfilled by analysing the extent to which the findings on land use changes influence the adequacy of the drainage structure.

5. Assessment of Maintenance on Hydraulic Reliability

- Use hydraulic simulation to assess the impact of the interventions by SANRAL to raise wingwalls and headwalls,
- Model the effect of debris/blockage

Research Aim 2 can be achieved by assessing the impact of maintenance using hydraulic simulation of blockages at the entrance to the drainage structure. The response by SANRAL to raise wingwalls and the headwall is evaluated using hydraulic modelling to assess its effectiveness in addressing the challenge of flooding at the study area. A discussion of maintenance practices identified in the literature review is also carried out to assess adherence and their effectiveness at the study area.

6. Broadening the Significance of the Analyses and Evaluating the Influence of Climate Change

- Highlighting of the limitations in the recommended design flood methods of the SANRAL Drainage Manual,
- Highlighting experiences at the M1 double decker section to emphasize the influence of debris on hydraulic reliability of drainage structures.
- Highlighting the influence of catchment changes and the potential impact of climate change influenced rainfall patterns and intensity on the hydraulic reliability of drainage structures.

Research Aim 3 draws from the key findings of the causes of flooding at the study area linkages which broaden the significance of the findings of the overall research.

Figure 1.1: Presentation of the Methodology Used in this Study

Given that the most recent and most widely reported flooding event at the study area happened over 4 years ago from the authoring of this report, the study will use confirmatory approaches to review prior work done in understanding the causes of the flooding. The confirmatory approaches will help verify the findings of these initial studies which are mostly unreferenced and only available from written responses by the Minister of Transport (National Assembly, 2017).

The exploratory aspects of the research will be relevant to the investigation of maintenance and land use aspects using repeat photography. Ground photography will be used to highlight maintenance issues at the study area and the changes in land-use will be investigated using Google Earth® Historical Imagery Tool (aerial photography). By following the guidelines of the SANRAL Drainage manual, the study also assesses the adherence to these guidelines, at the study area. Historic event rainfall data is obtained from the South African Weather Service (SAWS).

1.7 Structure of the Report

CHAPTER 1: Introduction to the Research

This chapter introduces the research and provides a background upon which the study is set. The research question and aims are presented together with the objectives to be met in response to the research question.

CHAPTER 2: Literature Review

This chapter reviews literature to inform on the different drainage design methods and the current best practice in drainage management. Parameters with an influence on the reliability of flood drainage structures are identified and discussed. The chapter outlines the remedial work done at the drainage structures at the study area and evaluates SANRAL's maintenance procedures. The influence of climate change on the design of drainage structures is also discussed.

CHAPTER 3: The Study Area

In this chapter, the general description of the study area is presented. The catchment is delineated, and the climatic, hydraulic, and hydrological parameters are specified. The chapter also presents the investigation into the changes in land use management and maintenance practices at the study area.

CHAPTER 4: Historic Flooding at the Study Area and the Determination of Adequacy of the Existing Drainage Structures

This chapter begins by theoretically determining the required capacity of the drainage structure using the SANRAL Drainage Manual Guidelines. A comparison is made between the theoretical capacity and the available capacity to assess the adequacy of the installed drainage structures. Hydraulic modelling in HEC-RAS is carried out to evaluate the effect of the maintenance changes done by SANRAL in response to the flooding at the study area and to establish the peak flood capacity of the drainage structure both before and after the modifications by SANRAL.

Further, reports of flooding at the study area are verified using the event data. The historic flooding events are then characterised according to their return period and evaluated for consistency with reported climate change trends in literature. The chapter also determines whether flooding would have been expected from these events based on a comparison with the theoretical and existing capacity at the drainage structure.

CHAPTER 5: The Influence of Maintenance and Climate Change on the Hydraulic Reliability of Drainage Structures

In this chapter, hydraulic modelling in HEC-RAS is carried out to evaluate the impact of debris and blockages at the study area. Various considerations derived from literature regarding parameters such as risk, land-use and climate change are also evaluated for their influence on the design flood.

CHAPTER 6: Conclusions and Recommendations

This chapter concludes the research. The key findings are presented together with the limitations of the research. The conclusion also informs whether the research aims were achieved and makes recommendations for future research.

Appendices: These provide additional material to support the discussions in the study.

CHAPTER 2

Literature Review

2.1 Overview

This chapter provides a basis for the response to the research questions by establishing a background to the urban road flooding problem. The chapter aims to outline the principal concepts underpinning urban road flooding. This will enable the study to build upon these concepts and establish how they integrate to cause flooding events such as those experienced at the study area on three separate occasions. The chapter also reviews peak/design flood estimation methods with the aim of selecting appropriate methods for use in the study.

2.2 Urban Road Flooding: Background

The inconvenience caused by flooded roads is of immediate concern to the road user as it presents a huge risk to injury and loss of property. The broader impacts of flooding include the interruption to businesses and the economy because of restricted mobility as well as property and personal loss.

Flooding also harms the road infrastructure as it causes structural damage to the road prism. This is a concern for the design life of the road in the long-term whose reconstruction would require considerable recapitalization. This loss of value is significant given that the price of constructing a kilometre of road was R3.5 million per kilometre for a lightly trafficked paved rural road, while constructing and maintaining heavy freeway structures was estimated to cost tens of millions of Rands per kilometre according to a 2011 national treasury report (National Treasury, 2011).

Figure 2.1 below shows the weighted priorities of the 112 wards comprising the Ekurhuleni Metropolitan Council on the y-axis and the categories of the priorities on the x-axis. The roads and stormwater needs dominate all other needs taking 37% of the budget share.

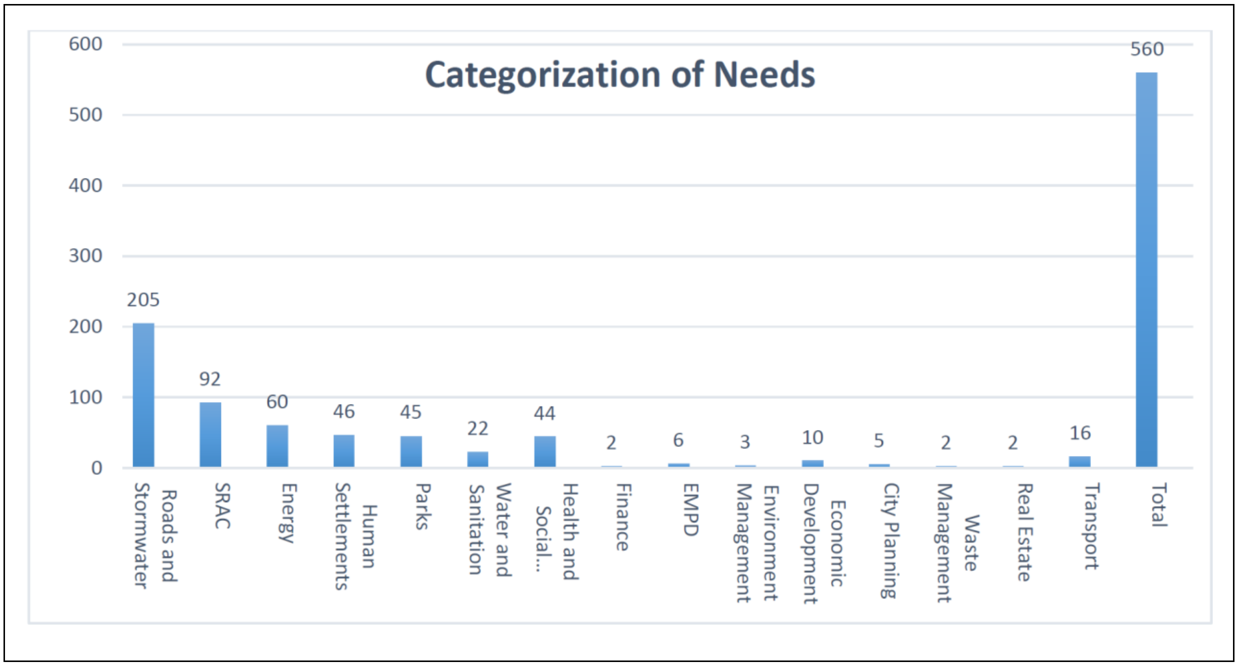


Figure 2.1: Ekurhuleni categorization of ward needs per department 2018/2019 (priorities x-axis, weighting y-axis) (Ekurhuleni Metropolitan Municipality, 2018)

The current budgetary requirements for the maintenance of Ekurhuleni roads, a metropolitan city within Gauteng where the study area is, is an estimated R11.25 billion (Waters MP, 2017).

Floods are amongst the most destructive of sudden-onset hazards and their occurrence is set to increase according to climate change projections which suggest increased volatility in the hydrological cycle (Fauchereau, 2003). The consequences of urban road flooding include the undermining or loss of function or stability of hydraulic structures such as the undercutting of bridges and blockages of kerb inlets. This leads to high maintenance costs for municipalities and increased biological risks due to potential contact with contaminated water.

Figure 2.2 and Figure 2.3 are adopted from Piketh et al., (2014) and they profile flash flooding in Ekurhuleni Metropolitan Municipality in which section 19 of the N12 between the R24/N12 and the Gillooly's interchanges (the study area) is situated. In the study, the topography and impacts of urbanisation are highlighted as the causes of flooding. A considerable extent of the map in Figure 2.2 is prone to flooding which suggests that the influence of drainage practices upstream has significant influence on the study area which is located downstream.

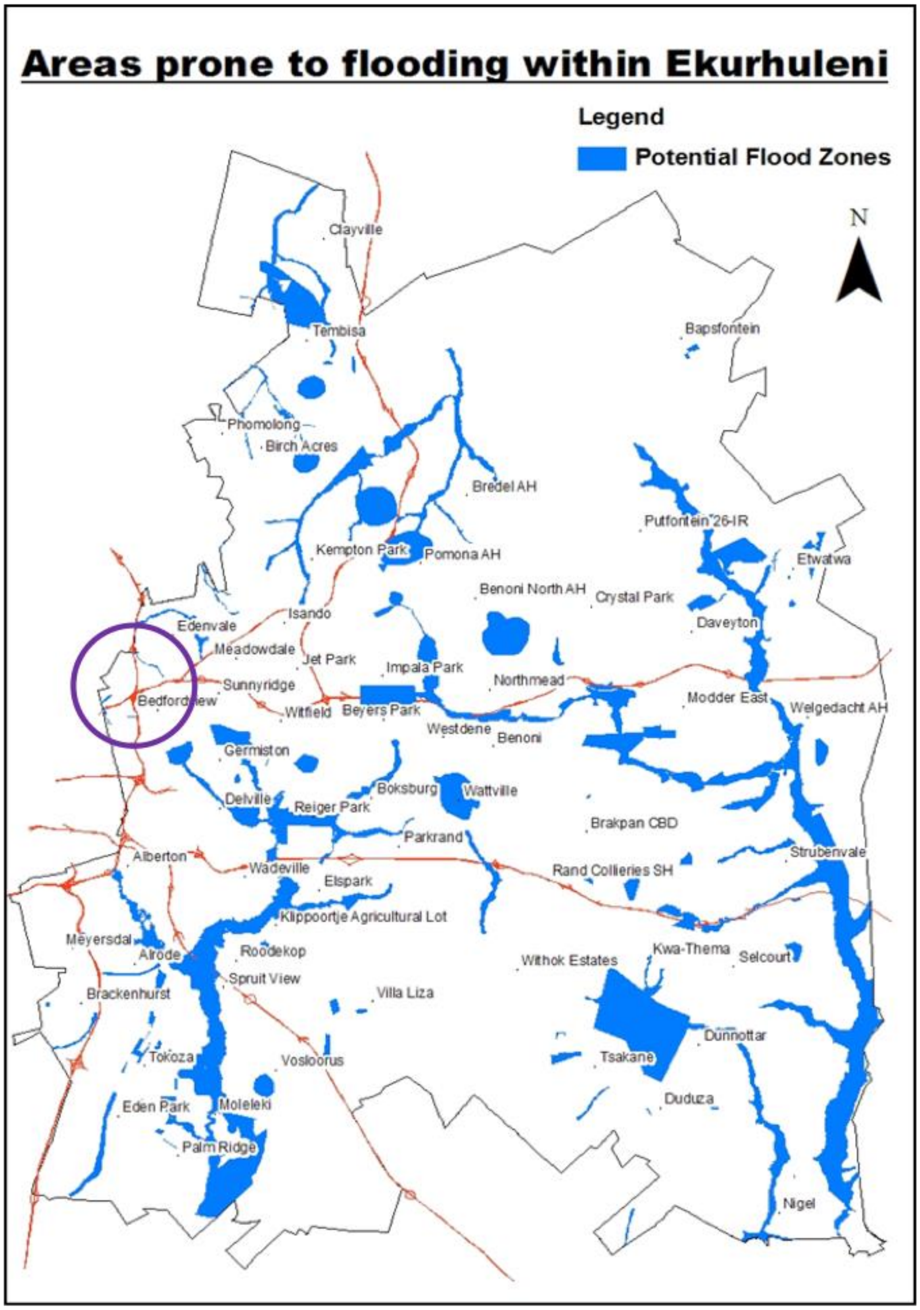


Figure 2.2: Potential Flood Zones for the Ekurhuleni. (Purple Circle Is the Vicinity of the study area- Medium to High Vulnerability) adopted from Figure 4 in (Piketh, et al., 2014)

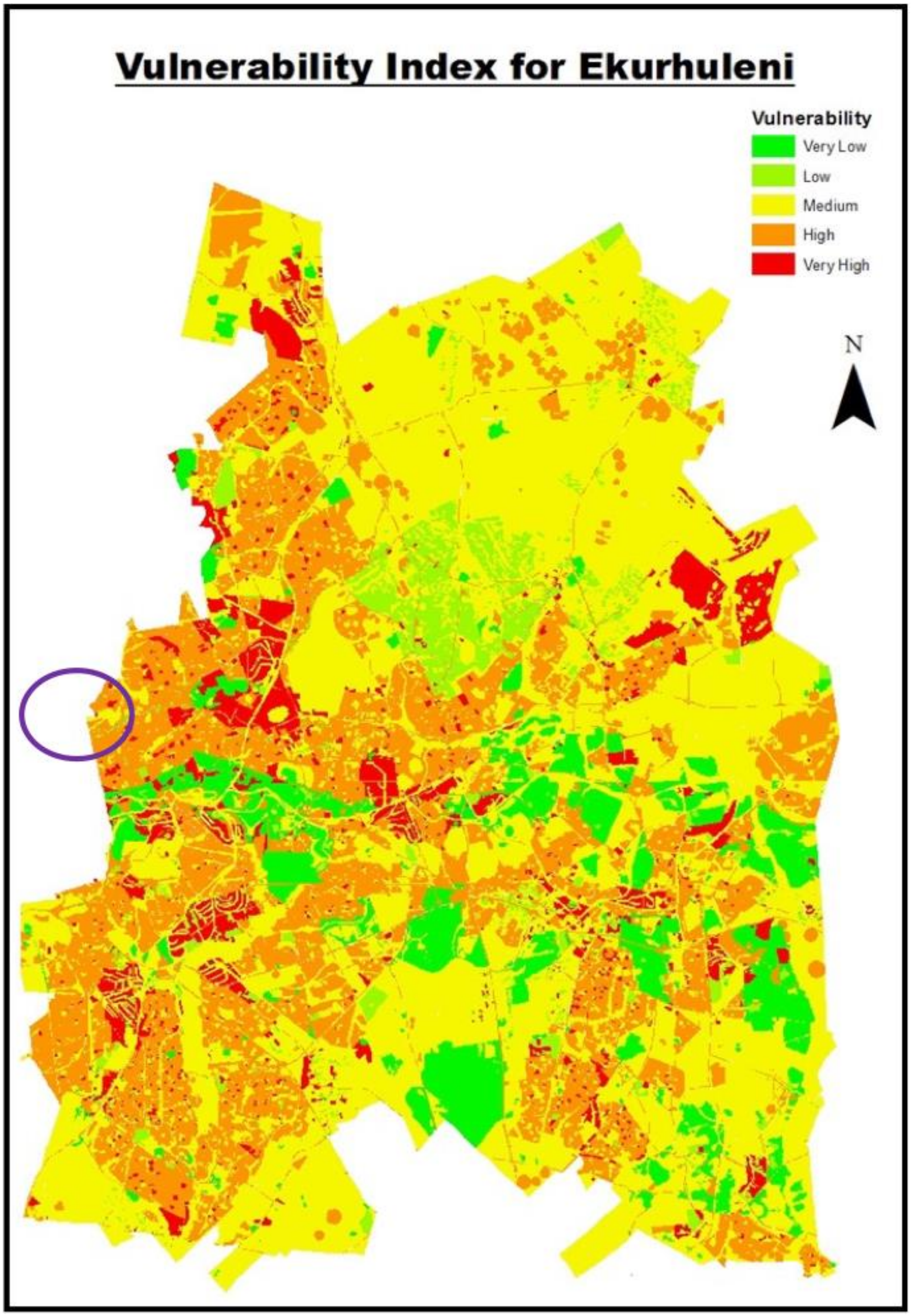


Figure 2.3: Vulnerability Analysis for the EMM Area (Purple Circle Is the Vicinity of the study area- Medium to High Vulnerability) adopted from Figure 5 in (Piketh, et al., 2014)

In his 2018 State of the City Address, the Johannesburg Mayor (a metropolitan city neighbouring Ekurhuleni and sharing several catchments) cited a R56 Billion backlog in storm water drainage management, blaming it for, "*the increasing structural decline of our roads* (Mashaba, 2018)." This highlights the costs of the structural damage that poor drainage management causes to roads.

Roads are often described as arteries through which the economy pulses (Berg, 2015). A road/pavement may be evaluated from two perspectives, a functional one and a structural one. The former is often of concern to the road-user as it deals with issues of comfort, service, and convenience (TRH6, 1985). The ambit of the road agencies will however extend to the understanding that a pavement is a load-bearing structure requiring timeous maintenance to remain serviceable (TRH6, 1985). Drainage inefficiencies impact on both these perspectives squarely. This challenge is compounded by, "*the failure of existing approaches to assess the disruptive impact of flooding on road transport to capture the dynamics and complex interactions between floodwater and the transport system"* (Pregnolato, et al., 2017)

2.3 Overview of Drainage Design Methods in South Africa

Pegram & Parak, (2004) cited in; Smithers, (2012) state that the design methods in South Africa are based on empirical, deterministic, and probabilistic approaches. Statistical analysis of observed peak discharges is not often possible due to a lack of streamflow data at the sites of interest, giving rainfall-based methods more prominence as a design flood estimation technique in South Africa. (Smithers, 2012).

The lack of streamflow data is common to a South African context and as such rainfall-based methods will be given greater emphasis in the analysis carried out in this study. An overview of the design flood estimation methods is presented in Figure 2.4 below.

Cordery and Pilgrim (2000) state that, *"the demands for improved estimates of floods have not been met with any increased understanding of the fundamental hydrological processes."* Van Vuuren, et al., (2013) proposed an alternative methodology to rainfall extremes for design flood estimation in which they challenge the current assumptions of climate stationarity present in design rainfall estimations.

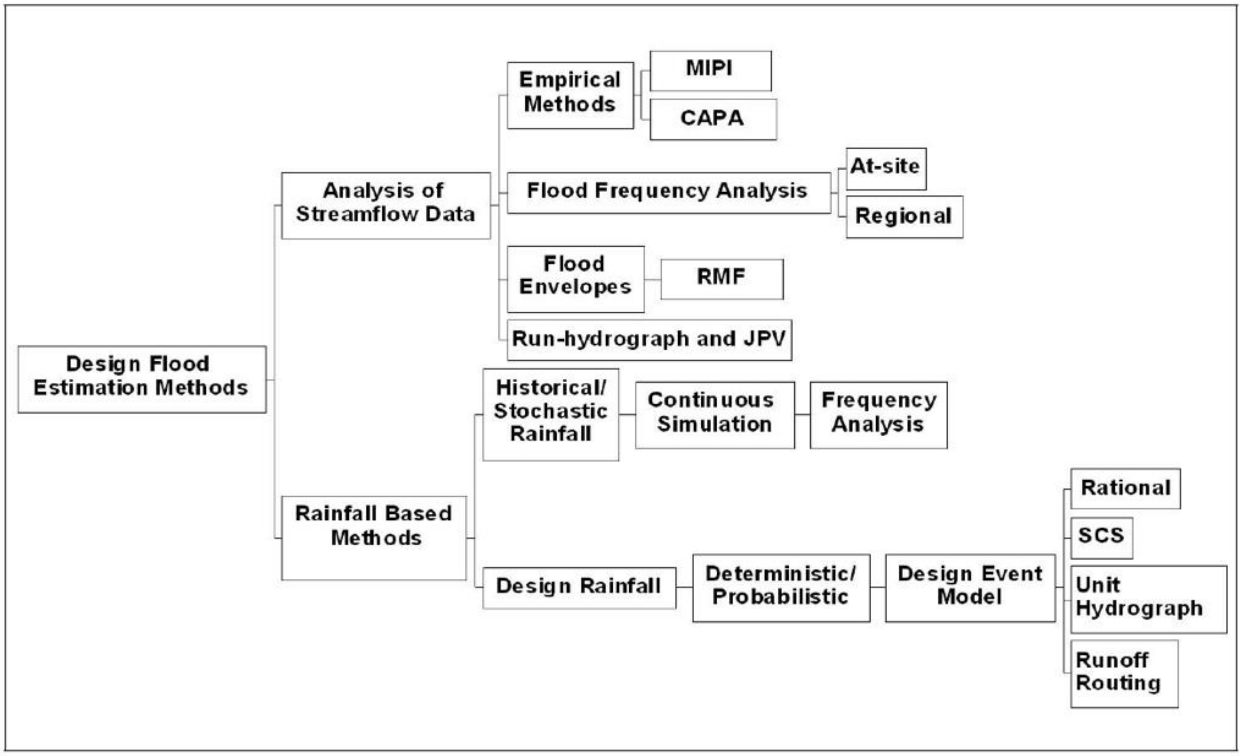


Figure 2.4: Approaches to Design Flood Estimation in South Africa according to (Smithers & Schulze, 2002).

2.3.1 *Design Methods in the SANRAL Drainage Manual*

This section reviews the drainage manual which is prescribed for use as the design guideline for the drainage system under review. The SANRAL drainage manual focuses on the deterministic methods, which is consistent with the survey by Campbell, et al., (1986) as quoted by Smithers (2012); where the outcome was that the Rational method was the most used method in South Africa.

The drainage manual (SANRAL, 2013) recognizes the following 3 factors as influencing the runoff amount generated from precipitation within the catchment:

- Climatological Variables (Climate and rainfall duration, and frequency)
- Topographical Factors (Catchment size, shape, slope, infiltration, soil type and geology, seasonal effects of vegetation and stream patterns)

- Developmental Influences (land-use, storage, reservoirs)

The manual proposes the Standard Design Method (SDF) as the required method for use and one which should be compared to the other methods when designing, as a good practice measure to check for large discrepancies in the peak discharge. The differences in peak flows derived from different methods must be consistent with the underlying assumptions of each of the methods as per the drainage manual. The manual's recommended methods for calculating flood peaks are listed below in Table 2.1

Table 2.1: Summary of Flood Estimation Methods as Presented by SANRAL (The South African National Roads Agency Ltd, 2006)

Method	Description	Input Data
Statistical methods	Applicable to large areas (no limit on catchment area), Flood return period depends on the available records.	Measured data, historical flood peak records,
Rational method	Deterministic, applicable to areas less than 15km^2, 2-100-yr return period	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity.
Alternative Rational method	Deterministic, no limit on catchment area, 2-200-yr. return period	
Unit Hydrograph method	Deterministic, no limit on catchment area, 15-5000km^2 catchment area, 2-200-yr return period	Catchment area, watercourse length, length to catchment centre, mean annual rainfall, veld type and synthetic regional unit hydrographs.
Standard Design Flood (SDF) method	Deterministic, no limit on catchment area, 2-200-yr return period	Catchment area, slope, and SDF basin number
Empirical method	No limit on catchment area, 10-100-yr return period	Catchment area, watercourse length, length to catchment centre, mean annual rainfall

The manual prescribes the Peak Discharge (maximum flow rate during flood) as capturing all three factors influencing the run-off as listed above and as such is the primary output of hydrological flood calculations. The Peak Discharge will inform subsequent considerations such as the sizing of the drainage structures and the allocation of the appropriate risk factor. The risk factor in the drainage manual is associated with the assigned road class and the likelihood of property damage and loss of life amongst other factors see section 2.6 below. The peak discharge allows for the flood levels and velocities to be determined using hydraulic calculations. The recommended methods from the SANRAL drainage manual are discussed in more detail below.

2.3.1.1 *Review of the Rational Method*

Van Vuuren, et al., (2013) identified the following weaknesses associated with the Rational and Modified Rational methods.

- The level of judgement required to determine the most realistic run-off coefficient is largely subjective.
- The variability of the coefficients between different hydrological regimes in the same catchment is not accommodated.
- The estimation of catchment response time is subjected to regional differences in the time of concentration and cannot be based only on measured catchment characteristics.
- The assumption of uniform precipitation intensity and the exclusion of temporary storage limit the use in urban and small rural catchments.
- The use of a probabilistic as opposed to a deterministic approach to determine the run-off coefficients is thus recommended (Alexander, 2001)

The SANRAL Drainage Manual (2006) correctly states that the simplified (triangular) hydrograph for the rational method is highly idealised. Kovács, (2012) cited in; Van Vuuren, et al., (2013) showed how this simplification does not reflect the complex nature of catchment's response to a rainfall event. The report also showed that when the runoff coefficient becomes 1, the runoff volume exceeds the rainfall volume, violating the mass balance.

2.3.1.2 *Standard Design Flood (SDF Method)*

Gericke & Du Plessis, (2012) showed that the original SDF method overestimated the magnitude and frequency of flood peaks in all basins under consideration. They also confirmed that the probabilistic approach of the original SDF method did not have the ability to overcome deficiencies present in other flood estimation methods in use in South Africa. However, Van Bladeren (2005) cited in; Van Vuuren, et al., (2013), revealed that the method was not always conservative unlike in the previous study, noting that the original SDF method did not use all the historical data of the annual maximum flood peaks.

Van Vuuren, et al., (2013) noted the following regarding the SDF; "*During the assessment the data sets used for catchment characteristics in the original development may have errors. A review of some of the site characteristics used indicated that errors of up to 100% were present in some of the characteristics. In one instance the river length used in the original study was 16 km while in a more recent study the river length is estimated to be 32 km.*"

Gericke & Du Plessis, (2012) recommended a review of the basins and the assumption of hydrological homogeneity. Van Vuuren, et al., (2013), also noted the importance of increasing the drainage basins and the recalibration of the method with greater focus on the catchment characteristics.

2.4 Drainage Design Parameters

Having discussed the flood estimation methods above, this section reviews two of the key parameters of these methods i.e., design rainfall and catchment response time.

2.4.1 *Design Rainfall*

Design rainfall is a critical input in the estimation of the peak design flood and the subsequent planning and design of engineering projects. "*Design rainfall is thus the rainfall depth and duration, or intensity, associated with a given probability of exceedance, which in turn is inversely related to the commonly used term, return period*" (Smithers & Schulze, 2002). The frequency analysis of rainfall data is dependent on the availability of adequate records. Frequency analysis refers to the use of historic records to estimate the probability of occurrence of future events.

The assumption of stationarity in design rainfall has been criticised widely in literature (Van Vuuren, et al., (2013) and Milly, et al., (2008)). Stationarity implies that natural systems fluctuate within an unchanging envelope of variability (Milly, et al., 2008), which is not necessarily the case in the context of climate change influenced rainfall patterns.

2.4.1.1 *Rainfall Trends*

The City of Ekurhuleni, in its Climate Change Response Strategy (Ekurhuleni Metropolitan Municipality, 2015) estimates an increase in the Mean Annual Precipitation (MAP) and extreme rainfall as shown in Table 2.2 below. The model used and the underlying assumptions in arriving at the variability in climatic parameters as predicted by the municipality are unclear. However, this indicates an acknowledgement of climate change effects by the municipality. There was no evidence found of these predictions being implemented or accounted for in current and future planning of drainage infrastructure within the municipality.

The results of EMM's model are consistent with Piketh, et al., (2013) who have shown changes of mean Annual Precipitation (MAP) of between +5% and -19% by 2040 and a 4% decrease thereafter up to the year 2100. Notably, the results indicated an overall decrease in rain days per year up to the year 2100 at an average of 9% for Ekurhuleni Metropolitan Municipality.

Table 2.2: Estimated Ekurhuleni Metropolitan Municipality Climate Change Factors (Ekurhuleni Metropolitan Municipality, 2015)

Climate Variable		**Current Conditions**	**2040 Predictions**	**Increase**
Rainfall	Annual Average Temperature	Max 25 ^{o}C (Summer) Min 17 ^{o}C (winter)	>+3 ^{o}C	✓

Climate Variable		Current Conditions	2040 Predictions	Increase
	Annual Average Rainfall	713 mm	841 mm	✓
	Summer	107 mm	116 mm	✓
	Autumn	53 mm	64mm	✓
	Winter	6mm	7mm	✓
	Spring	72 mm	94 mm	✓
	Heavy Rainfall Intensity		+6.4% increase	✓
Humidity (for every 1°C rise in temperature the humidity will increase by 7%	Annual Average Humidity	30-50%	14% increase	✓
Additional	Annual Average Extreme Number of Heatwaves	35 °C	14% Increase +2.1 °C increase in temperature	✓

Changing trends in climate result in the increased frequency in extreme weather events such as cyclones, droughts, and floods (DuPlessis & Burger, 2015). Schulze (2011 cited in; DuPlessis & Burger, 2015) observed that climate change trends were more significant when seasonal behaviour is observed as compared to annual behaviour.

Mackellar (2014), using the Mann-Kendall indicator-of-trend test, showed a trend where the central interior (region 3 in Figure 2.5 and Figure 2.6 below) was receiving increased rainfall with, on average, reduced rain days at varying significance. This trend suggests that the increased mean annual rainfall is occurring within shorter periods leading to increased intensity of rainfall events. More intense rainfall events have a larger impact on drainage especially in altered catchments which have a higher runoff coefficient.

Mackellar (2014) was also able to establish correlation with other scholars including Groisman, et al (2005 cited in; MacKellar, et al., 2014), who showed a significant increase in the annual frequency of very heavy rainfall events over eastern South Africa from 1906 to 1997. Mason, et al., (1999 cited in; MacKellar, et al., 2014) demonstrated increases in the intensity of high rainfall events in the 1961 to 1990 period relative to 1931 to 1960 over much of South Africa.

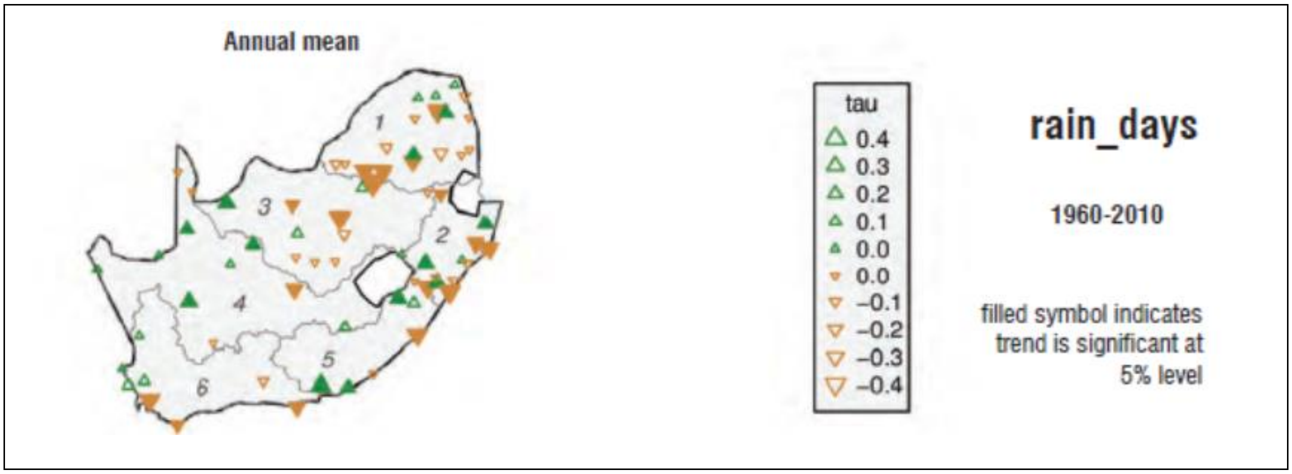


Figure 2.5: Mackellar et. al (2014) 50-year (1960-2010) Annual Mean Rain Days Trend

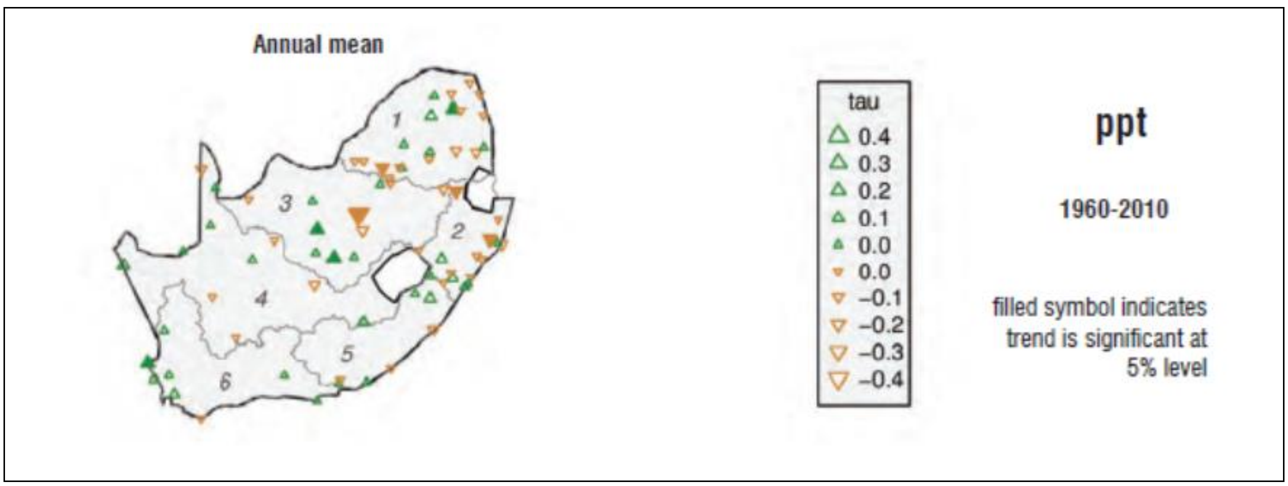


Figure 2.6: Mackellar et. al (2014) Mackellar et. al 50-year (1960-2010) Annual Mean Precipitation Trend

Kruger (2006 cited in; MacKellar, et al., 2014) showed increases in extreme rainfall indices over the Southern Free State and parts of the Eastern Cape from 1910 to 2004 whilst New et.al., (2006 cited in; MacKellar, et al., 2014) showed some evidence for increased rainfall extremes over parts of South Africa for the 1961 to 2000 period.

However, Mackellar, et al., (2014) was unable to simulate the observed trends using some of the prominent climatic models used to predict the increase in more intense rainfall events in the future.

This lack of correlation highlights the difficulty in climate prediction and a lack of consensus on the increased intensity of rainfall events in South Africa. This is further highlighted by Du Plessis and Burger (2015) who found no evidence of trends or indications of changes in rainfall intensities.

The period quoted by both Mackellar, et al., (2014) and Kruger (2006 cited in; MacKellar, et al., 2014) consists of a 100 year time series which might not be sufficient to make confident generalisations on climatic trends. The findings of Mackellar, et al., (2014) and Kruger (2006 cited in; MacKellar, et al., 2014) suggest that there is as yet no conclusive evidence that the country is experiencing more frequent and extreme rainfall because of global climatic change.

2.4.1.1.1 *Climate Change Discourse Related to Flooding Events*

Recent flooding between the 11th and the 12th of April 2022 in Kwazulu-Natal in South Africa has been attributed to climate change as highlighted by Pinto, et al., (2022), who reviewed the flooding event. However, the title of the paper, "*Climate change exacerbated rainfall causing devastating flooding in Eastern South Africa,"* may not be fully justified by the discussions and findings of the paper. This, to an extent, contributes to the discourse which ascribes the causes of flooding to climate change, often neglecting other causes as advanced in this study.

One of the findings of that study states the following regarding rainfall intensities, "*We conclude that greenhouse gas and aerosol emissions are (at least in part) responsible for the observed increases.*" (Pinto, et al., 2022). This may be because support for increasing rainfall intensity could only be found in the climate models and not from the observed data.

Further, the return period of the rainfall event when analysed regionally showed the event had a return period of 20 years although some individual stations recorded return periods of up to 200 years. The study further states that, "*from a meteorological perspective such an event was not unprecedented and additional factors such as the vulnerability and exposure of people, infrastructures and human systems played a role in making this meteorological event so impactful and worth studying.*" This statement is more representative of the findings of the study and would have been more balanced as a topic for the study given that the original topic went on to be picked up by news outlets such as News24 (2022) and Scientific American (2022).

The above highlights how in the immediate aftermath of a flooding event, discourse around climate change dominates, where otherwise a more complete evaluation of the flooding event could offer solutions towards resilience should the emerging trends of variability in rainfall intensity and patterns materialise, as is currently projected.

2.4.1.2 *Regionalisation Methods*

For rainfall-based events, when the observation period is less than the desired return period, the information is likely to be inaccurate. Even more observations are required if the particular interest is regarding extreme events. Often, inadequate rainfall data on individual sites gives rise to regional approaches which are adopted in the frequency analysis of hydrological data (Smithers, 2012). Smithers and Schulze (2002) discuss a Regional L-Moment Algorithm (RLMA) as used by them in the frequency analysis of hydrological data to estimate short (<24hr) and long (1 to 7 day) duration rainfall depth in South Africa.

This regionalisation approach results in the development of a frequency curve applicable anywhere within the defined region. This approach also helps standardise the major design input (rainfall) and as a result more standardised designs are attained. The critical component of RLMA is the formation of the regions which is recommended to be done using site characteristics. The study by Smithers & Schulze (2002) included the following characteristics: *latitude, longitude, altitude, a monthly index of concentration of* *precipitation, Mean Annual Precipitation (MAP), index of rainfall seasonality, distance from sea*.

The outcome of the regionalization was the identification of 15 homogenous short-duration rainfall clusters. However, Bobee & Rasmussen (1995) *have cautioned that L-moments may be too robust, and outliers given too little significance*. Smithers & Schulze (2002) also evaluated their design estimates against other methods such as in the DWAF Report TR102 (Adamson, 1981) and the HRU Report (Midgley & Pitman, 1978) and found no obvious anomalies in the data.

Dely & Esberto (2018) carried out distribution fitting using a simulation model which was able to accommodate 60 probability distribution functions whose fitness was determined using the Chi-square and K-S goodness-of-fit tests. Different geographic regions (stations) of the Philippines were reviewed for the study; however, for a single station for a period of 26 years, the monthly rainfall data could be best described by up to 19 theoretical probability distribution frequencies. This highlights the difficulty in selecting a distribution that is the best fit for the rainfall of a region as would be the case when developing a deterministic method which utilises regionalisation.

2.4.1.3 *Intensity Duration Frequency Relationships*

Under section 3.5.1.3 of the SANRAL Drainage Manual (2006), reference is made to Figure 3.6 of that same manual, which is the depth-duration-frequency (IDF) relationship for the determination of point rainfall. Abubakari, et al., (2017) found that the difference between intensities from existing and new IDFs for major cities and Towns in Ghana were consistent with explanations of climate change. The results showed that for shorter durations (12 mins and 24 mins), the new IDF curves gave higher intensities for the same return period with a percentage increase in the ranges of 2% to 25%.

In the study, Abubakari, et al., (2017), also found that for longer durations (42 mins, 1 hour-24 hours), the new IDF curves gave lower intensities for the same return period with percentage decrease ranging from 3% to 49% when compared to existing IDFs.

2.4.1.4 *State of Rainfall Data Collection*

The drainage design methods highlighted above rely on the quality and availability of rainfall and other hydrological data. Van Vuuren, (2011) showed a trend of decreasing flow gauging structures and rainfall stations in South Africa. This has a direct impact on the homogeneity of collected data which might affect its application in certain climatic models and flood estimation methods. Figure 2.7 and Figure 2.8 below show the declining numbers of flow gauging and rain gauging stations, respectively.

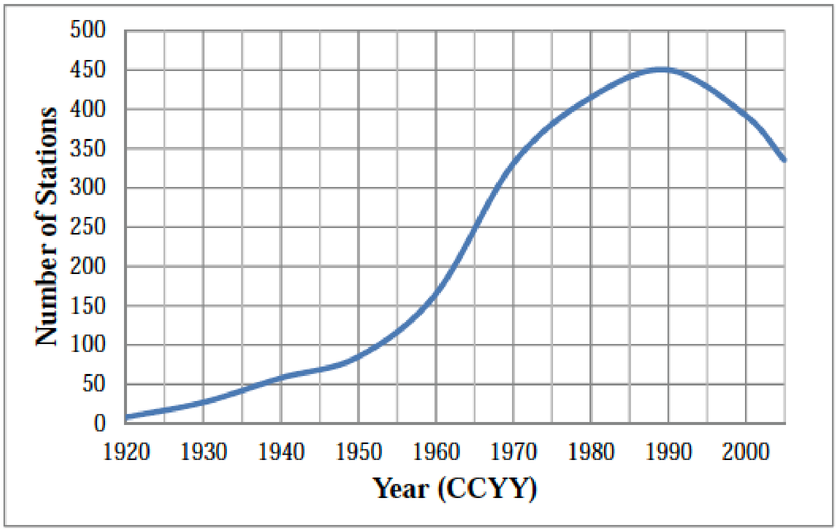


Figure 2.7: Number of Flow Gauging stations in South Africa (Van Vuuren, 2011) cited in; (van Vuuren, et al., 2013).

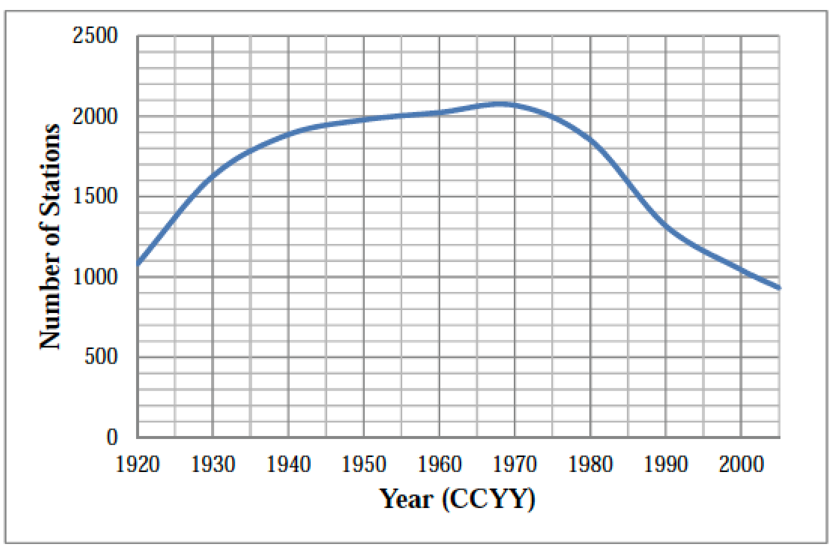


Figure 2.8: Number of Rainfall Stations in South Africa since 1920 (Van Vuuren, 2011) cited in (van Vuuren, et al., 2013).

2.4.2 *Catchment Response Time*

"...the catchment area is the single most important geomorphological variable, as it demonstrates a strong correlation with many flood indices affecting the catchment response time" (Smithers & Ockert, 2014). McCuen, (2005) explains how a particular runoff volume, without information on its temporal distribution, does not inform on the presence or absence of a flood hazard nor the overtopping of a hydraulic structure.

"T_C is the time required for runoff, as a result of effective rainfall, with a uniform spatial and temporal distribution over a catchment, to contribute to the peak discharge at the catchment outlet, in other words, the time required for a "water particle" to travel from the catchment boundary along the longest water-course to the catchment outlet" (Smithers & Ockert, 2014).

In design flood estimation, T_C is used to estimate the critical storm duration of a specific design rainfall event used as input to deterministic methods (Smithers & Ockert, 2014). The drainage manual defines T_C as the time from the start of total runoff (rising limb of hydrograph) to the peak discharge of total runoff (SANRAL, 2013).

McCuen (2009) cited in; Smithers & Ockert, (2014) highlighted that, due to differences in the roughness and slope of catchments (overland flow) and main watercourses (channel flow), Tc estimates, such as those which consider only the main watercourse characteristics, are underestimated on average by 50%. This in-turn leads to an overestimation of the peak discharge by between 30%-50%. *"…Because of differences in roughness, flow depth, and slope, the timing of runoff from most parts of a watershed differs from that along the principal flow path, which is generally used to compute times of concentration. Thus, times of concentration computed from principal flow path information do not accurately reflect the timing of runoff from most parts of a watershed…"* (McCuen, 2009)

Smithers & Ockert, (2014) note that the time variables related to peak rainfall intensities and discharge provide better estimates for smaller catchments where the exact occurrence of the maximum peak discharge is of more importance unlike in larger catchments.

The Drainage manual also references the influence of the catchment size on the rainfall/runoff relationship. It notes that the soil infiltration rates are more significant in smaller catchments whereas the overall storage in larger catchments has more significance on the peak discharge. (SANRAL, 2013). The drainage manual proposes limits to the catchment areas within which flood peak methods are applicable as shown in Table 2.3 below.

Smithers & Ockert, (2014) note that empirical methods should be limited to their original developmental regions if there are no local correction factors used to avoid observation, spatial and temporal errors. They further state that in South Africa, practitioners frequently apply not just the empirical time parameters outside their bounds but also some of the deterministic flood estimation methods.

Gericke, (2018) recommended that the simplified catchment response time (USBR T_c equation) and the design rainfall (modified Hershfield/TR102 DDF approach) be replaced by the empirical G&S T_c equation (Gericke & Smithers, 2016) and the Regional Linear Moment Algorithm and Scale Invariance (RLMA&SI) (Smithers & Schulze, 2000) DDF approach when deterministic design floods are estimated in South Africa.

Table 2.3: Application and Limitation of Flood Calculation Methods (SANRAL, 2013)

Method	Input Data	Recommended maximum area (km^2)	Return period of floods that could be determined (years)
Statistical method	Historical peak flood records	No limitation (Larger areas)	2-200 (depending on the record length

Method	Input Data	Recommended maximum area (km^2)	Return period of floods that could be determined (years)
Rational method	Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity	<15	2-100, PMF
Alternative Rational method		No limitation	2-200, PMF
Synthetic Hydrograph method	Catchment area, watercourse length, length to catchment centroid(centre), mean annual rainfall, veld type and synthetic regional unit hydrographs	15 to 5000	2-100
Standard Design Flood method	Catchment area, slope, SDF basin number	No limitation	2-200
Empirical methods	Catchment area, watercourse length, distance to catchment centroid, mean annual rainfall	No limitation (large areas)	10-1000, RMF

Van Vuuren, et al., (2013) identified the empirical estimates of T_L (Lag Time) as provided in the SANRAL Drainage Manual (2006) to be unverified against local data. (Lag Time: "*the time delay between the time runoff from a rainfall event over a catchment begins until the runoff reaches its maximum and is generally defined as the time between the centroid of effective rainfall and the resultant direct runoff hydrograph").* Smithers & Ockert, (2014) state that T_L=T_C. Gericke & Smithers, (2016) found that the estimates of flood estimation as applied in modern flood hydrology practice in South Africa were inconsistent and varied widely. They concluded that lower T_c values led to overdesign whilst the opposite would result in inadequate designs.

2.4.2.1 *Land Use Patterns*

The topography of a catchment has a major influence on runoff and the catchment response time. As a result, human interaction with the physical environment can impact urban flooding. Whether this impact is by way of attenuating or accelerating the peak flood would have to be determined for a given drainage structure to ensure an appropriate balance between the risks and economic considerations of overdesigning.

Van Vuuren, et al., (2013) argues that the impact of certain types of developments (particularly, high boundary walls) serves to attenuate the floods and highlights the difficulty in capturing the catchment characteristics. Van Vuuren, (2011) states that, " *the general notion that urban development will increase the flood peaks as well as the volume of discharge is unfounded, because the effect of temporal storage created by artificial barriers along the normal flow path and at all hydraulic structures designed for short design recurrence intervals is not considered."* There are no further studies which evaluate this phenomenon against the counter effect of weepholes, grid inlets and internal storm systems of which some directly drain from paved surfaces and roofs negating the influence of boundary walls.

In an interview the Ward 20 councillor in Ekurhuleni and the Municipality spokesperson highlighted challenges with concentrated runoff from properties as significant, particularly amongst neighbours of upstream and downstream properties (Dlamini & Humphreys, 2022). The SANRAL drainage manual (2006) also acknowledges that boundary walls have an influence on the peak discharge but concedes that this is site specific and depends on the topographic conditions of a given catchment.

The primary response to changes in the catchment due to urban developments is the development of drainage systems to compensate for the streams, waterways and infiltration lost to or altered by the developments. In the case of EMM, some streams have been routed underground into channels such as in Nigel, Alberton, and Springs (Piketh, et al., 2014). Most of the drainage systems in EMM have been in existence for a while now and modifications to the catchments they were designed for increases the risk for the failure of these systems.

Armitage, et al., (2014) introduced a Water Sensitive Urban Design (WSUD) approach. The WSUD approach entails an integrated approach to planning at a macro-level to ensure that the risks associated with catchment alterations are mitigated, mostly by interventions which mimic the undisturbed catchment. The adoption and successes of WSUDs will be critical in increasing the catchment response time or time of concentration.

WSUDs represent a more complete response to countering the effects of land use as they seek to both attenuate and reduce the overall runoff volume by enhancing infiltration. Van Lierop, (2015) termed this, hydrological and hydraulic neutrality and showed that despite benefits such as reduced erosion and improved water quality, attenuation systems that are not managed at the catchment level could be dangerous, particularly for downstream properties. The Queensland Urban Drainage Manual (2013) highlights the effect of coincidental peaks where uncoordinated attenuation systems could synchronise with peak flows from other sections of the catchment at critical locations downstream resulting in excessive flooding. As such, the manual limits the use of detention ponds in the lower third of any given catchment to counteract this effect.

The SANRAL Drainage Manual (2006) has developed empirical relationships which attempt to capture the influence of urbanisation on the peak discharge from a catchment. A study by Roodsaria & Chandler (2017) highlighted that for smaller catchments, the relationship between the relative nearness of the impervious surface to the catchment outlet yielded higher peak flows for impervious surfaces closer to the catchment outlet. The study also concluded that greater monitoring of peak discharge in small peri-urban catchments was critical to support flood prediction at the local scale (Roodsaria & Chandler, 2017).

Figure 2.9: Possible Influence of % of Impermeable Surface Area on Peak Flow, for Different Return Periods, Expressed as Multiples (adopted from SANRAL Drainage Manual (2006))

Return Period (years)	**Percentage Area Consisting of Man-Made Impermeable Surfaces**				
	1%	10%	25%	50%	80%
2-yr	1.0	1.8	2.2	2.6	3.0
5-yr	1.0	1.6	2.0	2.4	2.6
10-yr	1.0	1.6	1.9	2.2	2.4
25-yr	1.0	1.5	1.8	2.0	2.2
50-yr	1.0	1.4	1.7	1.9	2.0
100-yr	1.0	1.4	1.6	1.7	1.8

2.4.2.2 *Maintenance of Drainage Structures*

SANRAL has routine road maintenance contracts to ensure that national routes are kept at an optimal level of service. SANRAL relies on the route managers to correctly identify problems and acknowledges that the inability to correctly identify problems and understand the cause, can and has resulted in unnecessary or wrong repair methods being used (SANRAL, 2009).

SANRAL developed a Routine Road Maintenance Manual which, according to the agency, provides information to help its route managers recognise the common problem areas and to have an appreciation of probable causes.

Table 2.4 below is an excerpt from the routine route maintenance manual which indicates that SANRAL inspects drainage structures monthly. Regarding culverts, SANRAL states that, "*insufficient capacity requires enlarging of the structure or the construction of extra drainage structures. Normally this work will not be done under routine maintenance and the Route Manager must inform SANRAL so that appropriate action can be taken"* (SANRAL, 2009). The foregoing suggests that there are structural limitations presumably on the value of work which can be undertaken as part of routine maintenance. It is also unclear how the route manager would be able to attribute the cause of any flooding as insufficient capacity.

Table 2.4: Frequency with which various road elements are monitored according to the SANRAL Routine Route Maintenance Manual (SANRAL, 2009).

Road Elements	Frequency	Road Elements	Frequency
Signs	Yearly	Instabilities	Dependent on degree of problems
Road marking	Yearly		
Guardrails	Weekly		
Structures	Yearly	Informal Settlements	Weekly
Flexible road condition	Yearly	Illegal Access	Weekly
		Fencing	Monthly
Rigid road condition	Yearly	Illegal Signage	Weekly
Drainage	**Monthly**		

The scope of routine maintenance contracts often involves the mowing of overgrown verges, cleaning of culverts and other drainage structures. Figure 2.10 below shows the typical scope of work for a routine maintenance contract by SANRAL. SANRAL has reported collecting close to $10m^3$ of litter per day in some areas (SANRAL, 2018). The routine maintenance contact for section 19 of the N12 which covers the study area could not be found on the SANRAL website, it is an assumption of this study that the terms of such an appointment would be like the presentation in Figure 2.10 below.

PART C3: SCOPE OF WORKS

NATURE OF WORKS

The nature of work to be carried out under this contract includes:

- Establishment of camps on site
- Inspection of the road, structures, waterways, cutting/fill slopes and night inspections
- Accommodation of traffic
- **Pavement layers repairs**
- Crack sealing and patching of asphalt pavements
- Repair edge breaks and edge drops
- **Gravel shoulder repairs**
- Repair of slope failures and washaways
- Stabilisation of slopes
- Construction of drainage works to combat erosion
- **Cleaning of all drainage structures, including removal of grass and debris from grids, as well as clearing bridge drainage ports and scuppers**
- **Repairing damaged fencing**
- Clearing refuse from the road reserve, lay-byes and interchanges
- Repairing damaged road signs
- Cleaning of road signs
- **Installation and replacement of roadstuds**
- Repairing damaged guardrails and balustrades
- **Road marking**
- **Regular mowing of grass in the road reserve including the median and the removal of grass cuttings**
- Application of herbicide on road edges and around road signs
- **Eradication of weeds and undesirable plant growth**
- Burning or cutting of firebreaks and assistance with veld fires
- **Maintenance of trees and shrubs**
- Supply and spreading of topsoil
- Emergency assistance
- Removal of wrecks and abandoned vehicles
- Minor road works done under daywork

Figure 2.10: Excerpt from a SANRAL Tender Briefing for Routine Maintenance of National Route N1 and R34 (SANRAL, 2020)

2.4.2.3 *Repeat Photography*

This study will use re-photography to assess the changes in land use and maintenance aspects at the study area. Kull (2005) found that repeat photography, *if pursued opportunistically alongside other fieldwork, the method is efficient in time and cost; it can be a useful way to identify environmental trajectories worthy of further investigation, to corroborate other studies, and to illustrate changes*.

Pupo-Correia, et al., (2014), found landscape repeat photography to be an effective tool to quantitatively assess vegetation change. They also note temporal and spatial bias as key issues to the efficient use of this method. In this study, the results of the repeat photography will be used to support the investigation into the causes of flooding at the study area emanating from maintenance and land use patterns.

2.5 Flooding Background at the Study Area

Desktop and field investigations of the study area were done to inform the investigation into the cause of flooding. This was meant to establish the remedial work and any interventions that may have been carried out at the study area, given that there had been at least 3 instances of flooding recorded at the study area. Further, the authoring of this report was at least four years since the reporting of the last major flooding event at the study area.

2.5.1 *Previous Studies Investigating the Causes of Flooding at the Study Area*

There is evidence of studies investigating the causes of flooding at the study area. Although these studies were not available for a detailed review when carrying out this research, their conclusions were available from various sources where the Ministry of Transport and SANRAL, the responsible authorities, reveal their proposed remedial actions.

In a parliamentary report (National Assembly, 2017), the Minister of transport confirmed that SANRAL had instituted a specialist study which found that, "*The R24/N12 culvert system is inadequate to handle the recommended design flood (1:80). The inlet configuration, changes in catchment area and proximity of the commercial developments all played a role in the problems experienced at this site*." The parliamentary report further recommended that an additional four 3x1.8m culverts be added at the section including the addition of a retention/stilling pond upstream, linking the existing channel to the new culvert system.

The only recorded remedial action to date was confirmed in a parliamentary report which stated: “*In August 2019, SANRAL completed upgrades to the inlet of the existing N12 drainage culvert to improve water-flow. To date this has resolved the frequent flooding that occurred along the Westbound part of the N12*" (National Assembly, 2020).

A review of the SANRAL tender archives revealed that a tender, SANRAL N.012-190-2016/1F had gone out for the remedial works at the interchange in August 2020. The tender scope is shown in Figure 2.11 and highlights the need for collaboration with the

Ekurhuleni Metropolitan Municipality in the planning and implementation of the remedial works through a memorandum of understanding. This represents an integrated approach to the management of the catchment and is a key aspect which could have negatively influenced the initial design. The drainage structure falls under the management of SANRAL, yet the flood it is designed to pass is generated from a catchment managed by the Ekurhuleni Metropolitan Municipality.

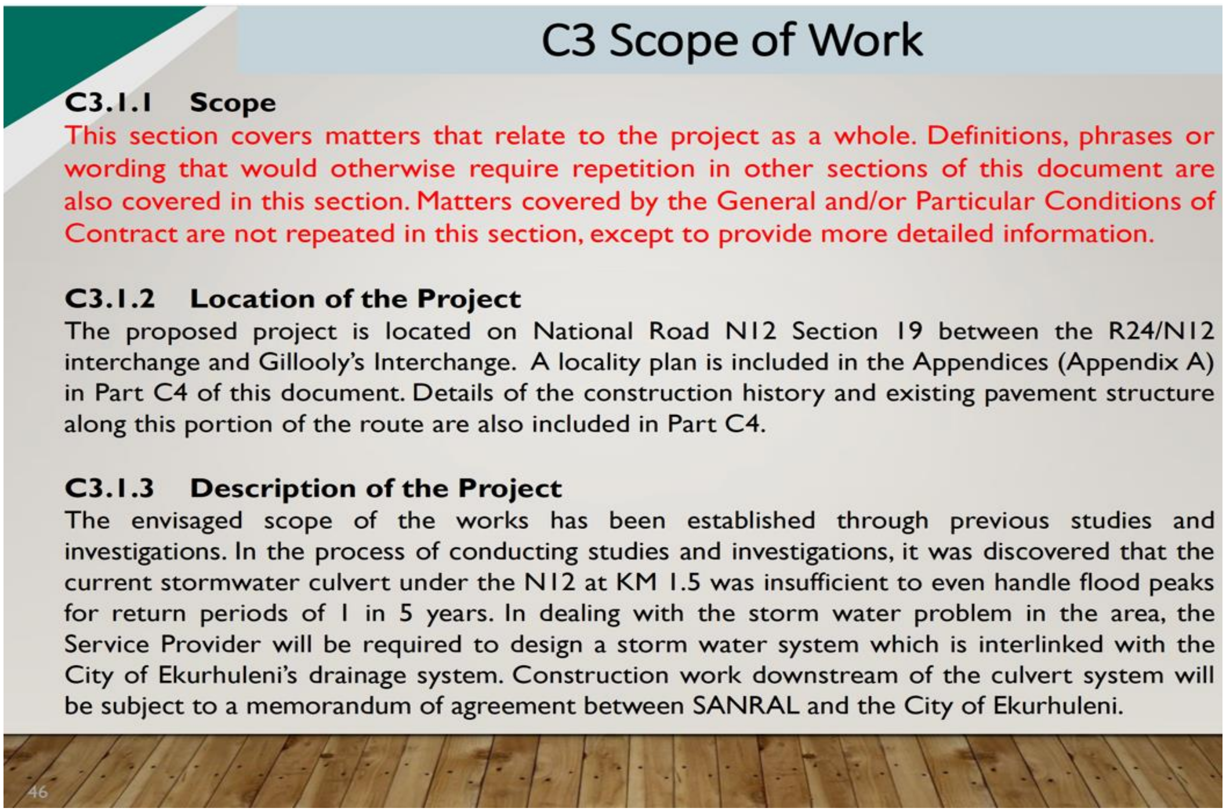


Figure 2.11: Excerpt of Scope of Works for the Tender for Remedial Works at the Study Area

2.6 Risk

In the IPCC climate change technical summary, Pörtner, et al., (2022), define risk as, "*the potential for adverse consequences for human or ecological systems, recognising the diversity of values and objectives associated with such systems.*" Most notable and aligned with the objectives of this study is the definition of resilience by Pörtner, et al., (2022) as, "*the capacity of social, economic and environmental systems to cope with a hazardous event or trend or disturbance, responding or reorganising in ways that maintain* *their essential function, identity and structure whilst also maintaining the capacity for adaptation...*"

Climate change remains a complex and dominant topic in flood modelling as well as a significant contributor to uncertainty in flood modelling. This is due to the influence of climate change on the flood design parameters already discussed in this chapter and summarised below.

- Topographic factors: such as loss of land cover, canopy, and erosion.
- Developmental influences: such as climate change influenced migration or over abstraction of existing water sources or the altering of existing water courses.
- Climatic Variables: such as changing rainfall patterns and trends.

2.6.1 *Hydraulic Reliability*

For the purposes of this study, hydraulic reliability may be defined as the consistency with which a drainage structure will maintain a design capacity greater than the peak flood demand. Sharma, et al., (2021) have defined reliability for a limit state function G, where q is the pipe flow and Q_p is the pipe flow capacity and Q_y is the flow load and P_f is the probability of failure, see Table 2.5 below.

Table 2.5: Reliability of Hydraulic Structures (as adopted from (Sharma, et al., 2021)

Limit State	**Probability of Failure**	**Hydraulic Reliability**
$G(q)= Q_p-Q_y$	P_f= probability [G(q)<0]	$R=1-P_f$

2.6.2 *Risk in the SANRAL Drainage Manual*

Risk is loosely defined in the SANRAL drainage manual, which implies that a lot of discretion is left to the designer. There is no discussion surrounding climate change and the applicable risk adjustments for it in the drainage manual. For class 1,2 and 3 roads, which includes the class 1 road being investigated for this study, the manual prescribes a freeboard requirement such that, "*Freeboard to shoulder breakpoint shall be zero or positive for a flood having twice the recurrence interval*" (SANRAL, 2013).

The foregoing implies that drainage structures for class 1,2 and 3 roads should only experience overtopping for a rainfall event twice the designed return period. Other findings in literature which only account for the risk to climate uncertainty (i.e., excludes other risks such as the road class and associated damage) estimate a slightly less conservative safety factor of between 1.4-1.7 to the pipe diameter (Sharma, et al., 2021). The drainage manual also cautions designers to consider the effects of debris and prescribes that at least 0.3m of freeboard be provided if significant debris is expected.

Table 2.6 shows the factors to be considered in determining the risk category for road drainage design according to the SANRAL Drainage Manual. The drainage manual refers to Q_D (the design flood) which is the flood in m^3/s, a given structure would have been designed to pass.

Table 2.6: Adopted from Table 2.3 of (SANRAL, 2006).

FACTORS TO BE CONSIDERED	**RISK CATEGORY**		
	1	**2**	**3**
Extent of possible damage			
Potential damage to the road and associated cost of repairs	Low	Medium	**High**
Potential other damage such as saturation of agricultural land, etc,	Low	Medium	**High**
Extent of loss of use			
Time needed for repairs to make route trafficable again	Short	**Medium**	Long
Availability of detours	**Good**	Medium	None
Obstruction of traffic flow			
Period of flooding	Short	**Medium**	Long
Traffic density	Low	Medium	**High**
Depth and velocity of floodwaters	Low	Medium	**High**
Strategic and economic importance of route			
Strategic and economic importance: military, police, fire brigade, medical services, etc.	Low	Medium	**High**
Economic importance	Low	Medium	**High**

Table 2.7 uses the 20-year return period as the basis for design according to the road classifications as outlined in the (TRH 26, 2012) South African Road Classification and Access Manual. The manual recommends that the outcomes of the findings in Table 2.7 be risk adjusted based on the outcomes of Table 2.6. Additionally, the Drainage Manual mentions the need for an independent adjudication of the risk and economic factors thereof.

Table 2.7: Proposed Design Return Period, T, For Different Road Classes, adopted from Table 2.2 of (SANRAL, 2006)

Road Class		**Proposed Return Period (T) Based on the Magnitude of the Q_{20} flood**		
		$Q_{20}<20m^3/s$	**$20m^3/s<Q_{20}<150m^3/s$**	**$Q_{20}>150m^3/s$**
1	Primary Distributor	50	$T=42.31+0.385Q_{20}$	100
2	Regional Distributor	20	$T=15.39+0.231Q_{20}$	50
3	District Distributor	1	$T=8.46+0.077Q_{20}$	20
4	District Collector	5	$T=4.231+0.0385Q_{20}$	10
5	Access Roads	2	$T=1.539+0.0231Q_{20}$	5
6	Non-motorised/ Access ways	2	$T=1.539+0.0231Q_{20}$	5

SANRAL has created a nomogram based on Table 2.7 for ease of use when determining the acceptable design flood frequency for a drainage structure for a particular road class. For a given road class, an acceptable risk of failure for hydraulic structures can be estimated using Figure 2.12 below.

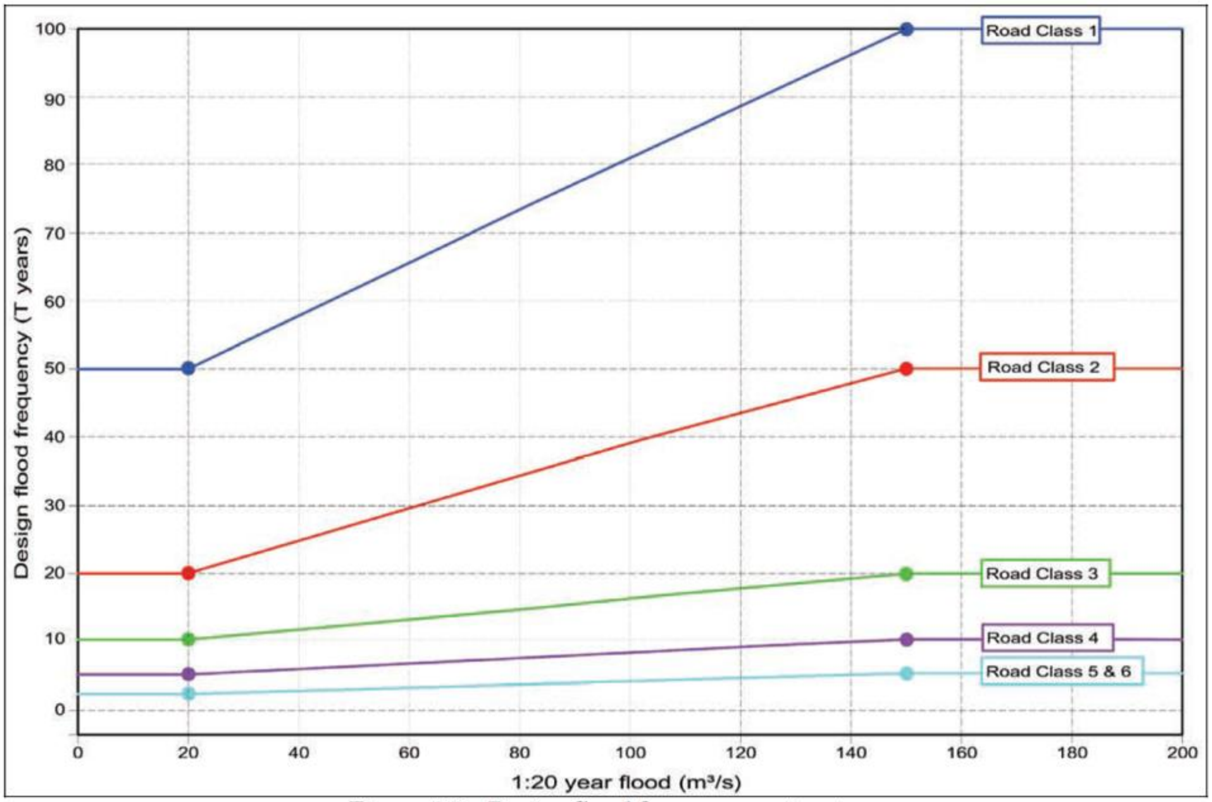


Figure 8.2: Design flood frequency estimate

Figure 2.12: Design Flood Frequency Estimation Adopted from (SANRAL, 2013)

2.7 Summary

The literature review has presented the background of urban road flooding to contextualise the research question. Drainage design methods in use in South Africa were reviewed with reference to the SANRAL Drainage Manual. The SANRAL Drainage Manual recommends the Rational and the Standard Design Flood (SDF) methods for flood peak design of the structures managed by SANRAL therefore, the two methods were reviewed in more detail.

This review included the assessment of key design parameters of both the Rational and SDF methods. As highlighted in the reviewed literature, there are gaps in knowledge on the understanding of the interaction between rainfall and catchment response time. These gaps complicate the response to uncertainty in flood estimation resulting from anthropogenic climate change. The chapter has also reviewed the maintenance procedures of drainage structures recommended by SANRAL since inadequate maintenance often leads to blockage and the inefficiency of drainage structures.

The reviewed literature does not suggest that there is an up-to-date catchment masterplan available to designers working on drainage designs within Ekurhuleni. The spokesperson of the city of Ekurhuleni, Mr Zweli Dlamini as quoted in Dlamini & Humphreys (2022) said, "*The formal Stormwater management guidelines and by-laws are under review*." Several draft guidelines were found through a google search, but they were not listed on the Ekurhuleni website along with other by-laws and guidelines.

As informed by the background into the flooding at the study area, presented in section 2.5 of this chapter, there were some changes made to the drainage structure at the study area by SANRAL. This study will evaluate the extent to which these changes address the flooding problem. Although the parliamentary report states that the remedial work carried out in 2019 was for the short-term, up to 3 years later, no major work has been carried out yet.

CHAPTER 3

The Study Area

3.1 Overview

This chapter presents the study area. The catchment area is defined, and the climatic and hydraulic parameters are specified. This enables the determination of the capacity of the installed drainage structures in subsequent chapters of this research. An assessment of changes in land use within the catchment and maintenance aspects at the study area is also carried out. This chapter partially fulfils Research Aim 1 and Research Aim 2

3.2 Description of the Study Area

The literature review has highlighted how catchment parameters play a significant role in the determination of the design flood and the design of drainage structures. In describing the study area, parameters relevant to the SDF and Rational Methods will be identified i.e., the catchment area, longest water course, slope of longest water course, mean annual precipitation and the runoff coefficient.

The locality of Section 19 of the N12 between the N12/R24 and Gillooly's Interchanges, is shown in Figure 3.1 below. The study area is located within the Ekurhuleni Metropolitan Municipality in the Gauteng province of South Africa. Figure 3.1 also shows the proximity of the N12/R24 and Gillooly's interchanges with the latter also having been flooded during the storm of November 9, 2016. Gauteng is the smallest but most populous province in South Africa with 25.4% of the national population residing in it (Stats SA, 2018). This is driven by its economic attractiveness both within the country and the Southern African Development Community (SADC), contributing over 34.8% of the country’s Gross Domestic Product (Stats SA, 2017).

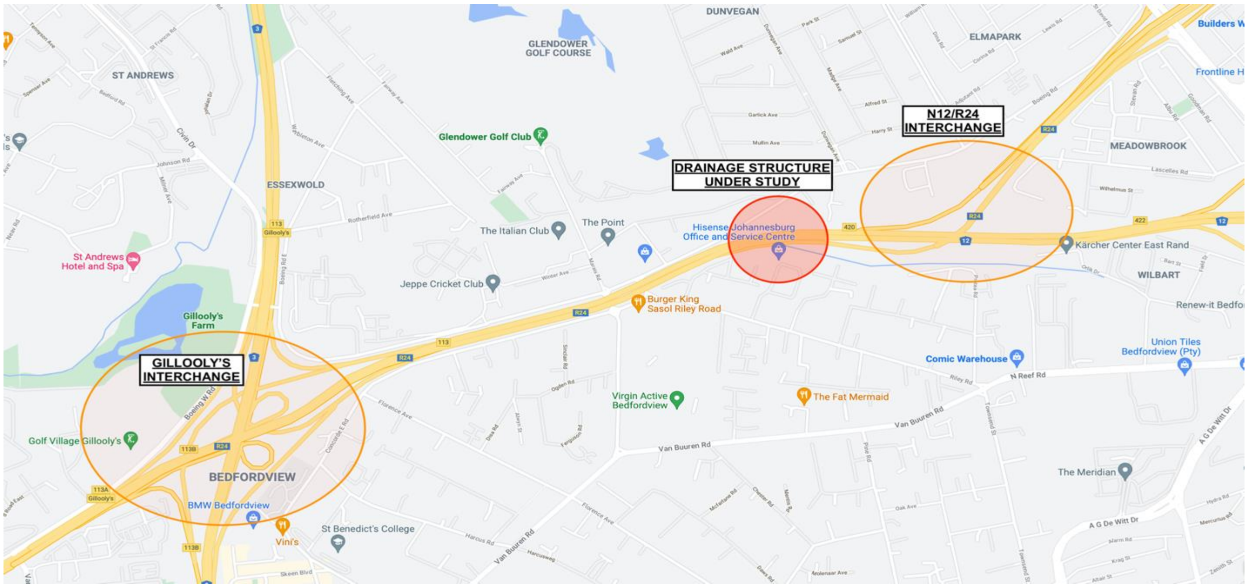


Figure 3.1: N12 Road Section Between the N12/R24 and Gillooly's Interchanges in Red. (Google-maps, 2018)

3.2.1 *Drainage Structures at the Study Areas*

The drainage structures at the study area are outlined in Figure 3.4 below. Figure 3.2, Figure 3.5 and Figure 3.6 highlight changes made by SANRAL (National Assembly (2020) at the inlet of the drainage structure. In chapter 5 of this study, a hydraulic simulation is conducted to assess the effectiveness of these changes, as stated in section 2.5.1 of this study.

Figure 3.2: Showing the inlet at the drainage structure with a clear construction line on the wingwall indicating when increased height (See B in Figure 3.4)

Figure 3.3: Section Showing Study Area (Google, 2017)

Figure 3.4: Photographic Summary of the Study Area

Figure 3.5: Upstream of the channel also showing the extent of the increased wingwalls (see A and B in Figure 3.4).

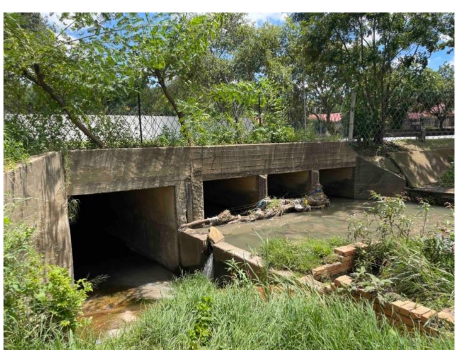
Figure 3.6 below shows that the elevation of the wing-walls of the drainage structure on the upstream-end used to be flush with the elevation of the channel. When compared to Figure 3.5 above, the extent of the upgrade work carried out by SANRAL is clear.

Figure 3.6: Upstream of the Drainage Structure Before Upgrades *(Google Earth 2018)*

During the field investigations, it was observed that Boeing Road runs parallel to the N12. The N12 passes through the drainage structure under study. Boeing Road is just 73m downstream of the drainage structures being reviewed and is serviced by much larger drainage structures as seen in Figure 3.7 below. From the catchment analysis carried out in this study, there is no obvious reason for the structure across Boeing Road to be significantly larger (>100% in area) than the N12 drainage structure. These differences suggest that the catchment drainage is likely not managed in an integrated manner.

(a) Locality Map of the two drainage structures Being Compared

(b) Inlet of Parallel Road to the N12, Boeing Road



(c) Outlet of Study Area

Figure 3.7: High Level Comparison of the Drainage Structures Near the Study Area

3.3 Climatic Parameters

Figure 3.7 of the drainage manual shows an average annual rainfall of between 600-800 mm. The more conservative choice to be used for the computation using the rational method is 800 mm. Various other values of MAP were available in literature including from the design rainfall model by Smithers & Schulze (2002) which averages to 742,5 mm, as presented in Appendix 1. Deference to the Drainage Manual is in keeping consistency with the assumed design procedure for drainage structures for a SANRAL route.

3.4 The Catchment Area

Defining the catchment area will be useful in the determination of the flood peak which will inform the sizing of the drainage structures. Catchment delineation is necessary to:

1. Determine the area with an influence on the drainage structures under study.
2. Determine the key parameters of the catchment such as the slope, longest watercourse and the coefficients of runoff which are necessary for the determination of the peak flood.
3. Find out the drainage structures within the catchment which impact on the flood characteristics at the study site.

The analysis of the catchment was carried out using AutoCAD Civil 3D's watershed and water-drop tools. The watershed analysis generates, from given elevation data, sub-catchments, and their respective drainage points. The water-drop tool simulates the path a drop of water falling within the study area would take and this is useful when combining the various sub-catchments. The elevation and cadastral data were obtained from Ekurhuleni Metropolitan Municipality through Calliper Consulting Engineers (2018). The elevation data was dated 2009, after the completion of the 2010 Gauteng Freeway Improvement Project. The most significant road upgrade project at the study area to date.

The accuracy of the elevation data was checked by evaluating how well indicative discharge points aligned with the discharge points as observed at the study area. Additionally, further comparison of the elevation data with data from the digital elevation model (DEM) of Global Mapper® (2019) and the catchment files of the Water Resources South Africa study (2012) was also done. The outcome of this process, the catchment delineation, is shown in Figure 3.8 which shows the extent of the catchment in magenta and the longest water course, in yellow. The drainage point of the catchment corresponds to the drainage structure at the study area and is highlighted by the orange circling. The size of the catchment is 538.9ha (5.4km^2). The profile of the longest stream is presented in Figure 3.9. From the longest watercourse in the catchment, the 1085 slope S_o. was determined as shown in Table 3.1.

Ideally, for a drainage design, up to date and detailed elevation data would have sufficed particularly for the determination of invert levels. However, this is not always feasible, catchment-wide, even in practice. The drainage masterplan for the area would have given more insight into the management of the catchment, particularly the piped drainage systems upstream of the study area. Inaccuracies in the elevation data may be mitigated by the fact that, post the Gauteng Freeway Upgrade project of 2008 (2008), no significant changes to the study area could be found in literature. This is also supported by the findings of the field survey carried out for this study.

Table 3.1: Data on the Longest Stream in the Catchment

DISTANCE from Drainage Structure (m)	ELEVATION (m)
0	1743,9
409,02	1729,93
3476,67	1617,8025
4090,2	1612
S_O	0,036552

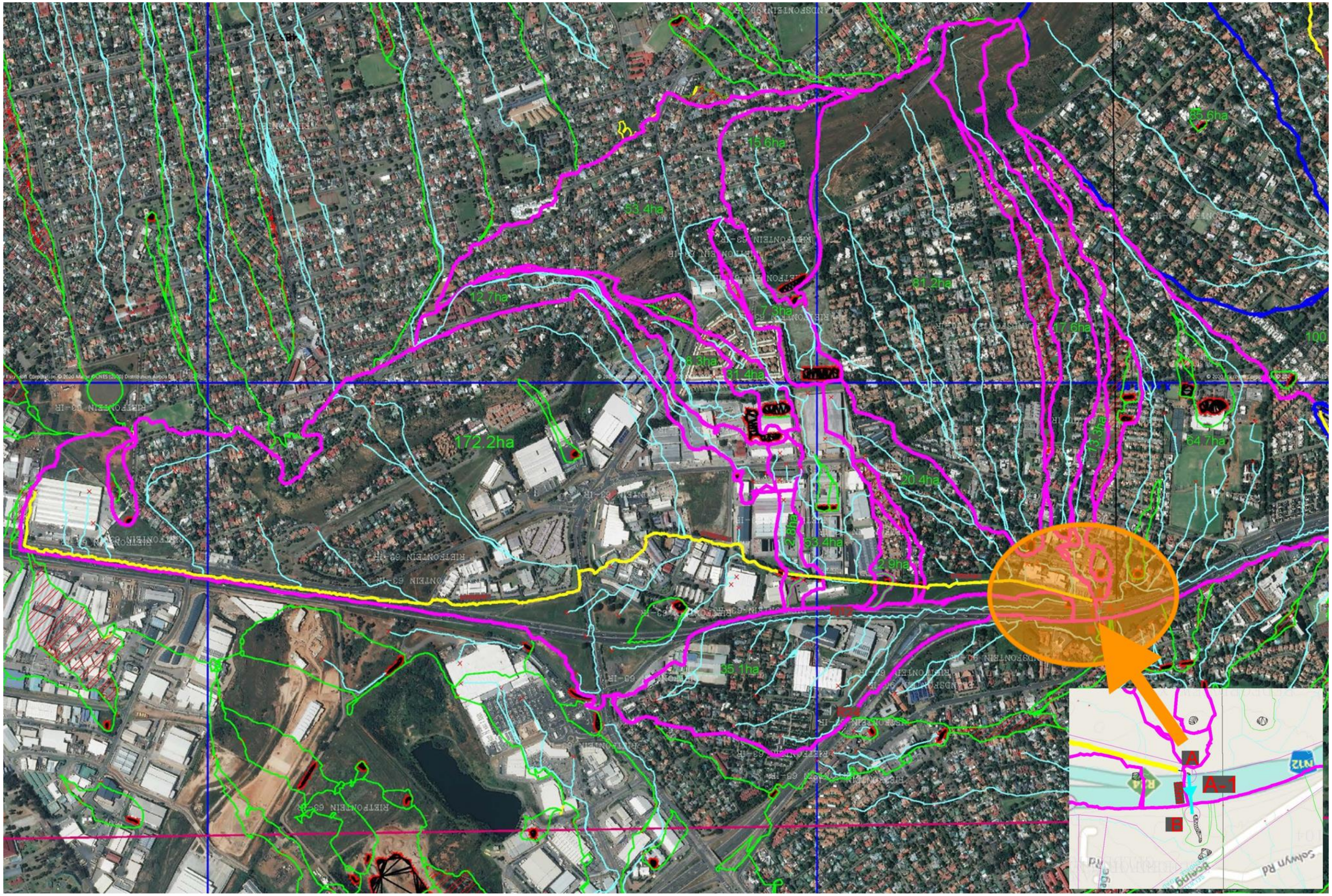


Figure 3.8: Catchment Delineation (catchment extent is shown in magenta/pink)

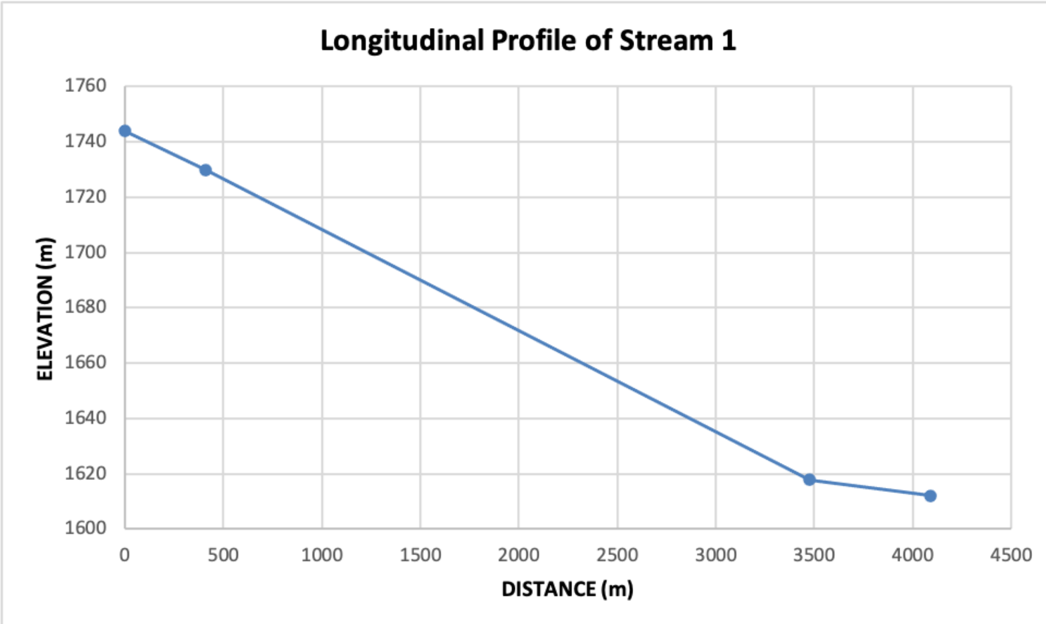


Figure 3.9: Profile of the Longest Stream in the Catchment

3.5 Hydraulic Parameters

This section presents the hydraulic parameters of the drainage structure at the study area. From the field survey, the dimensions of the drainage structure as presented in Figure 3.10 below were determined. Figure 3.10 should be cross-referenced with Table 3.2 below.

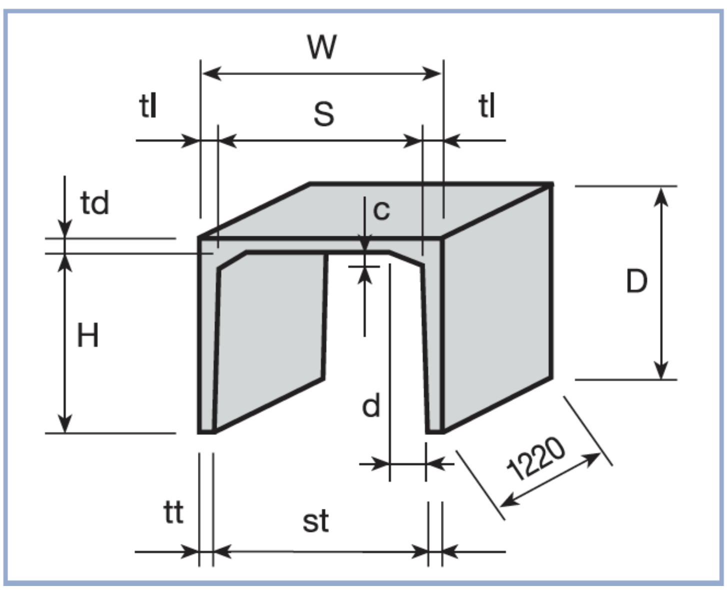


Figure 3.10: Rectangular Culvert Schematic (ROCLA, 2019)

Table 3.2: Measurements taken at the Study Area referenced to the Nearest Standard Culvert Dimension.

Nominal Size	W	D	St	td/tl	tt	c	d	area
S (mm) x H (mm)	mm	mm	mm	mm	mm	mm	mm	m^2
2400 x 1200	2720	1360	2480	160	120	100	300	2.91

The dimension of the existing drainage structure is further defined as follows:

- **Upstream**=4800 mm width x 1200 mm Height. The final level of the travel way above bed of channel is 2500 mm
- **Downstream**= 4800 width x 1200 mm Height. The final Level of travel way above the bed of the channel is 3500 mm.
- The road is cambered towards the inlet at the point the drainage structure crosses the N12, as such, the difference in elevation between the channel bed and the final road level is higher at the outlet of the channel than the inlet.
- Average roughness coefficients are obtained from literature for rough, turbulent flow. The channel is concrete lined with float finish as adopted from SANRAL's Fig 4.8 b on page 183 of the drainage manual. Therefore, K_s is 0.004m
- S_0= Slope in direction of flow: Upstream=0.0145m/m and Downstream of drainage structure=0.026m/m

Some measurements have been obtained during the field survey whilst other elevation differences have been obtained from the available elevation data.

3.6 Significant Land-use and Catchment Changes

This section uses repeat photographs to demonstrate changes within the catchment which could have an impact on the reliability of the drainage structure at the study area. The section will highlight observed changes in land-use, upstream of the catchment.

It was established that no other work has been done to upgrade the drainage structure at the study area except for the upgrades reported in the National Assembly (2020). As such, changes to the catchment area highlighted in this section have not been accompanied by any direct improvements of the drainage structure at the study area. This is also supported by findings in Figure 4.11.

3.6.1 *Land Use Changes Within the Catchment*

Aerial photographs upstream of the study area and within the catchment area defined in Figure 3.8 were compared for a 20-year period between 2001 and 2021 using Figure 3.11 and Figure 3.12 below. The findings show densification within the catchment area and the introduction of increased hard surfaces. The figures show that most changes to the land use within the catchment consist of conversions of existing single-family dwellings into multi-family apartments and office blocks.

The dual drainage system of minor systems (urban formal drainage networks) and major systems (conduits, natural or artificial channels, overland flow on roads) is defined as the common drainage system in South Africa (2005). The road networks in Figure 3.11 do not appear to have changed in the period under analysis and it may be assumed that the capacity of the minor systems associated with the road network has also remained relatively unchanged despite the increase in impervious surfaces. This study has demonstrated in chapter 3.2.1 above, how the major system has been modified in 2019 by raising wingwalls.

In Figure 3.11, the area marked "B", adjacent to the stream at the study area has been developed and this could have altered the runoff so close to the drainage point of the catchment which is at the study area. Roodsaria & Chandler, (2017) discuss the influence of relative nearness of imperviousness to the catchment outlet and found that this correlates positively with increased runoff (and decreased time to peak). The study also found that the impact of impervious surface distribution was more pronounced in smaller catchments (area<40km^2). Figure 3.12 shows "A" which appears to have been an attenuation feature such as a detention pond and has now been partially converted into a parking area. This study could not investigate the extent to which attenuation measures have been applied within the identified developments. However, it shows at least a 6.2% change in land use upstream of the catchment area. The areas in Figure 3.13, are compared to the total catchment area of 538.9ha to determine the % change in land use.

In the literature review, Van Vuuren, (2011) informed that urbanisation would not necessarily lead to increased peak flows since artificial barriers (boundary-walls, etc.) would increase temporal storage and attenuate flood peaks. However, this conclusion did not consider the phenomenon presented here where there is a re-densification of existing urban areas with the same barriers but with increased hard surfaces. The findings here support those of Piketh et al., (2014) who highlighted topography and impacts of urbanisation as the major causes of flooding in EMM.

3.6.2 *Assessing the Influence of Land Use Changes on the Peak Flood*

The estimated influence of a 6% change in land-use on the peak flow is by a factor of 1.24 as derived from Table 3.3 of the SANRAL Drainage Manual (2006), also see Figure 2.9 of this study. This assumes that the 6% change in land use represents a portion of the catchment area completely converted to an impermeable surface whilst ignoring the influence of attenuation.

The assumption that the change in land use is predominantly the conversion of the given catchment surface area into an impervious surface is supported by the observations presented in Figure 3.11 below since most of the changes in land use are from the construction of buildings on greenfield sites.

The factor of 1.24 to account for the influence of changing land use will be applied to the theoretical peak flood to be calculated in the next chapter to determine the influence of the change in land use on the peak flood.

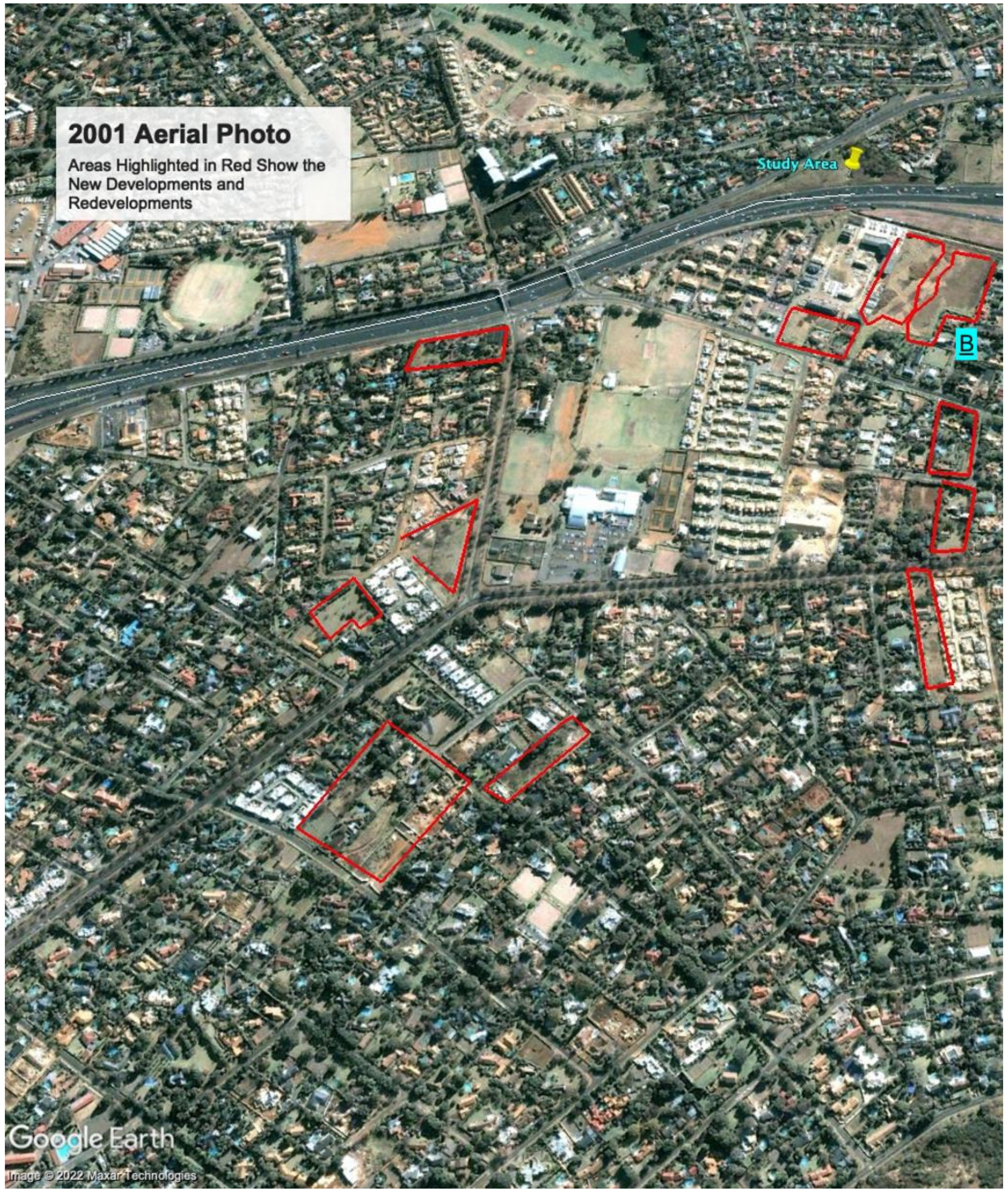


Figure 3.11: Summary of Land Use Changes Within the Catchment Westbound of the Study Area.

Figure 3.12: Summary of Land Use Changes Within the Catchment Eastbound of the Study Area.

Figure 3.13: Summary of Areas of Land Parcels with Recorded Changes Within the Catchment Westbound of the Study Area.

3.6.3 *Drainage Structure Maintenance at the Study Area*

Photographs were taken at the study area on separate occasions to document the maintenance regime applied. SANRAL's maintenance procedures have previously been discussed in section 2.4.2.2 of this report where it was established that the monitoring of drainage structures is carried out monthly.

Field surveys were carried out at two different points in time, 13-months apart. The photographic evidence presented in Figure 3.14 below highlights a lack of maintenance at the study area. The different points in time between the series of photographs presented in Figure 3.14 gives insight into the maintenance practices at the study area, with the more recent pictures showing evidence of neglect and the continued accumulation of debris. This suggests that the monthly monitoring schedule by SANRAL might not be translating into the corresponding maintenance actions for the drainage structure at the study area.

Location Based on Figure 3.4	**November 2020**	**January 2022**
D		

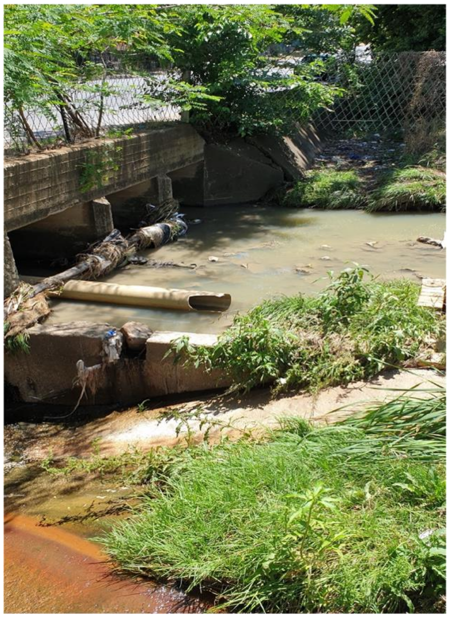
Location Based on Figure 3.4	November 2020	January 2022
E		
F		

Figure 3.14: Re-photographs Illustrating a Deteriorating State of Maintenance.

More frequent rephotographs would confirm the findings of Figure 3.14 above. Photographs could be integrated into the reporting template of route managers to improve the information gathered during maintenance inspections. The photographs can also be used to compare the before and after state of drainage structures following precipitation events. This could be useful for structures under investigation or could serve as a warning for the hydraulic system if the photographs are taken before and after recorded precipitation events of various magnitudes.

Overgrown vegetation is noticeable in the photographs taken in January 2022. Location E and F also show significant debris trapped within the gabion mattresses and at the inlet of the drainage structure at Boeing Road for the entire period under analysis. Given that both pictures were taken within the rainy season for the study area, it suggests that the inspection regime for drainage structures which was listed as monthly in Table 2.4 may either not be adhered to for the study area and would need to be updated or that the recommendations of these inspections are not implemented.

There is a wider problem of flooding within EMM. A simple online search of, "Flooding in Ekurhuleni," returns results of severe flooding for almost every rainy season ((News24, 2022), (EWN, 2021), (Alberton Record, 2020), (EWN, 2019)**,** (City of Ekurhuleni, 2018). There is a theme from these reports which suggests that the catchment is not proactively managed with most of the effort going towards reactive response to the flooding. Some of the flood warnings suggest that the Ekurhuleni Emergency Services are aware of the vulnerable areas, but little is done to address these issues before the next rainy season or event.

3.7 Summary

This chapter has presented the study area and defined the catchment including identifying parameters relevant to the computation of the design flood using the SDF and Rational methods.

Figure 3.2 has shown, and verified the changes highlighted in the National Assembly (2020) report where SANRAL increased the height of the wingwalls and headwall at the inlet of the drainage structure by an average of 1m for a length of 26m. The effectiveness of these changes will be analysed in Chapter 4 and Chapter 5 of this study.

Changes in catchment land use that could impact the rainfall-runoff characteristics and subsequently flooding at the study area were also reviewed using desktop techniques. Changes in the land uses within the catchment were analysed over a 20-year period using satellite images. Although the physical impact appears nuanced, at >6% of the total catchment area, the resulting impact on the peak flow is estimated at 24% using guidelines of the SANRAL drainage Manual.

The drainage structures along Boeing Road were presented in Figure 3.7. An eyeball test of the difference in the drainage structure sizes at Boeing Road and at the study area could have informed the designers of the drainage structure under study to re-evaluate their design. This highlights the importance of a field survey in the design of and determination of causes of flooding at drainage structures. Catchment management is an integrated process which should consider the influence of the entire catchment.

CHAPTER 4

Historic Flooding at the Study Area and the Determination of Adequacy of the Existing Drainage Structures

4.1 Overview

In this chapter, the theoretical capacity of the drainage structures required at the study area is determined using the SANRAL Drainage Manual Guidelines. A comparison of the theoretically determined design flood and the design capacity available at the drainage structures at the study area is made. This comparison enables the determination of the adequacy of the current configuration of the drainage structure. Historic flooding events at the study area are analysed and characterised and their return periods estimated. The chapter also determines whether the recorded precipitation events would have been expected to cause flooding based on the peak flood associated with their return periods in comparison to the peak flood that can currently be accommodated at the study area. This chapter fulfils research Aims 1 and 2.

4.2 Theoretical Design Flood Determination

The catchment parameters gathered thus far in this report are sufficient for the calculation of the theoretical design flood as would have been done in a typical drainage design. The guidelines of the SANRAL Drainage Manual will be followed in determining the design flood and sizing for this design flood. The theoretical design flood obtained here will be compared to the design flood capacity for the measured drainage structures at the study area. This responds to Research Aim 1 by determining the adequacy of the currently installed drainage structures.

4.2.1 *Design Flood*

The results of the peak flood calculation, using the catchment parameters that were presented earlier in Chapter 3, are presented in Table 4.1. Detailed calculations for the SDF and Rational methods are presented in Appendix 2 of this study. The results show that the rational method yields the highest design flood for the calculated return periods.

Table 4.1: Theoretical Design Flood Determined for the Study Area

Method		Design Flood (m^3/s)						
	T_c	1:2	1:5	1:20	1:50	1:64*	1:80*	1:100
Rational	0.9304 hrs	41.5	57.3	98.76	135	140.2	147.45	187
SDF	0.7014 hrs	6.67	23.26	56.84	83	89.79	96.81	104.95
% Difference	28%	145%	85%	53.8%	47.7%	43.8%	41.46%	56.2%

Determined for section 4.3.1 below after applying risk factors

The results show an average difference of 67.6% between the design flood determined using the two methods. Grimaldi, et al., (2012) found in their study that the T_c obtained by different equations varied by up to 500%. They also found that 75% of the total error in design emanates from the time of concentration calculations. In this study, the percentage difference in the peak response times between the SDF method and the Rational method was 28% which is far less than the 500% found by Grimaldi, et al., (2012).

The discrepancies highlighted in Table 4.1 above are, in some instances, consistent with some of the findings in the literature review such as Gericke & Smithers, (2016) who found that the estimates of flood estimation, as applied in modern flood hydrology practice in South Africa, were inconsistent and varied widely. They concluded that lower T_c values led to overdesign whilst the opposite would result in inadequate designs. However, as observed in Table 4.1 above, the higher T_C of the Rational method has provided a higher design flood than the lower T_C of the SDF method which could be attributed to other parameters within the methods such as the determination of the point rainfall.

4.3 Sizing Drainage Structures for the Theoretical Design Flood

In this section, drainage structure sizes for the determined theoretical peak design floods in Table 4.1, is determined. This enables a comparison between the currently installed drainage structures and the drainage structure sizes based on the theoretically determined design flood using guidelines of the SANRAL Drainage Manual.

4.3.1 Determination of the Recommended Design Flood Frequency at the Study Area

To size the drainage structures, the SANRAL Drainage Manual requires that the designer applies a risk factor which is a function of the road class to be serviced by the drainage structure being designed. The road class would have considered aspects such as traffic volumes and the intended level of service of the road. The design flood frequency for a class-1 road, as is the N12 which passes through the study area is done deterministically using Figure 8.2 of the SANRAL Drainage Manual reproduced as Figure 4.1 below.

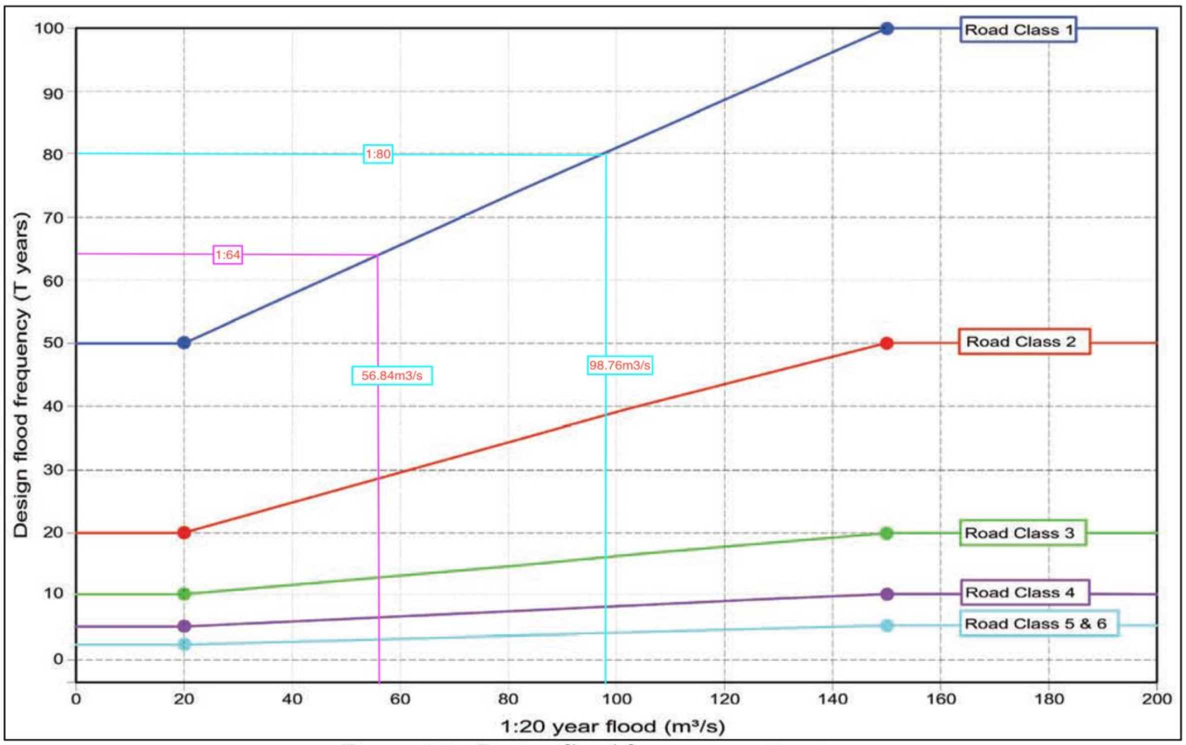


Figure 8.2: Design flood frequency estimate

Figure 4.1: Determination of the Design Flood Frequency at the Study Area

To obtain the design flood frequency, the designer initially determines the 20-year design flood. From Table 4.1 and Figure 4.1 above, the design flood to be used for the drainage structure at the study area is a 64-year (89.79m^3/s) design flood and an 80-year

($147.45m^3/s$) design flood using the 20-year peak floods of the SDF and Rational methods, respectively.

The results of the rational method confirm SANRAL's findings that the design return period at the drainage structure is 80-years, "*The R24/N12 culvert system is inadequate to handle the recommended design flood (1:80)"* (National Assembly, 2020). Therefore, for the purpose of this case study, the drainage structure will be sized for the 80-year design flood. This is also the more conservative design.

4.4 Checking for Adequacy Based on the Theoretical Design Floods Obtained from the SDF and Rational Methods

The theoretical sizing for the drainage structures is done for a design flood of $147.45m^3/s$ as described above. A comparison of the normal flow depths of the SDF and Rational methods has been carried out using the Manning, Chezy and Energy Equations. Table 4.2 below summarises the normal flow depths at the study area for the design flood obtained using the Rational and SDF methods for the current configuration of the drainage structure.

The current configuration of the drainage structure consists of a total available normal flow depth of 2.5m and was presented in Figure 3.2. This configuration is hereafter referred to as the post-upgrade configuration following the modification to the wingwalls and headwalls carried out by SANRAL in 2019.

The pre-upgrade configuration consists of reduced height of the current headwalls and wingwalls by 1m, which is what existed prior to the 2019 upgrades, as was presented in Figure 3.6. Table 4.2 shows that none of the normal flow depths can be accommodated by the pre-upgrade configuration, however, the normal flow depth of the SDF method could be accommodated at the post upgrades configuration. The determination of adequacy is carried out using the design flood calculations.

Table 4.2: Summary of the Upstream and Downstream Normal Flow Depths

	Sizing for the Design Flood obtained using the SDF Method: 89.79m^3/s	**Sizing for the Design Flood obtained using the Rational Method: 147.45m^3/s**	**Post-Upgrades Allowable Flow Depth**	**Pre-Upgrades Allowable Flow Depth**
1. Upstream Flow Depth	*Upstream Flow Depth (Y_u)*= **1.82m** *Area*= (Y_u*4.8) = 8.736m^2 Froude Number (5.92)>1 thus flow is supercritical. *Upstream Energy Head (H1 for Inlet Control):*7.20m	*Upstream flow Depth (Y_u=)* **2.64m** *Area*=(Y_u*4.8) =12.67m^2 Froude Number (5.23)>1, thus flow is supercritical. *Upstream Energy Head (H1 for Inlet control)*: 9.54m	***2.5m*** *(Exceeded by Rational Normal Flow Depth by 0.14m)*	***1.5m*** *(Exceeded by Rational Normal Flow Depth by 1.14m)* *(Exceeded by SDF Normal Flow Depth by 0.32m)*
2. Downstream Flow Depth	*Downstream Flow Depth (Y_D)* = **1.60m** Area= (Y_u*4.8) = 7.68m^2 Froude Number (8.722)>1 thus flow is supercritical.	*Downstream Flow Depth (Y_D)* = **2.31m** *Area*= (Y_u*4.8) = 11.1m^2 Froude Number (5.23)>1 Thus, flow is supercritical.		

4.4.1 *Determining the Capacity of the Current Drainage Structure Configuration*

The adequacy of the drainage structures cannot be fully defined by only evaluating the normal flow depths. This is because the influence of the headwater upstream of the drainage structure must be considered i.e., inlet control.

SANRAL specifies a headwater to internal drainage structure depth ratio (H_w/D) of no more than 1.2 where H_w is the total depth of water (measured from the invert of the drainage structure), and D is the interior height of the drainage structure barrel. This design guideline helps reduce the risk of overtopping and excessive velocities which result in supercritical flows and a potential for hydraulic jumps and excessive erosion.

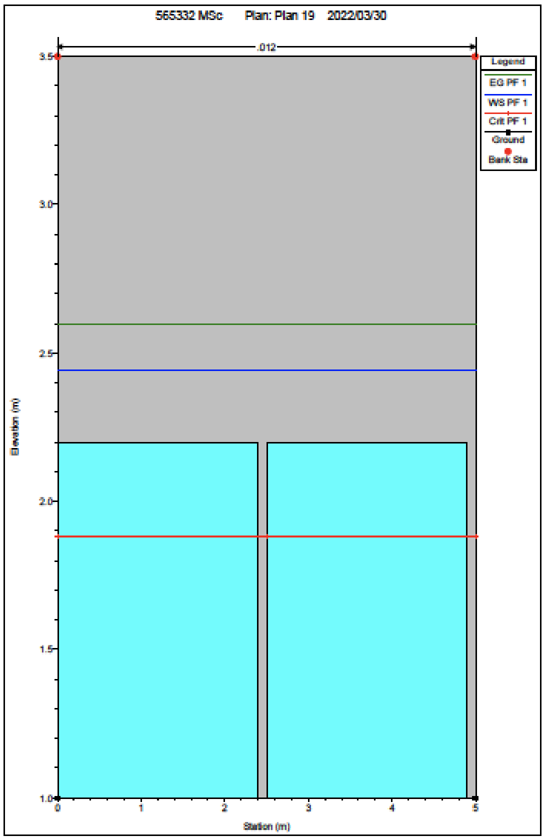
Given that the depth of the existing drainage structures (D) is 1.2m for the currently installed drainage structure, the acceptable H_1 should be 1.44m to satisfy an H_w/D of 1.2. The determined normal flow depth for 64-year and 80-year design floods of 1.82m and 2.64m respectively (in Table 4.2 above), exceed this available capacity of the current configuration of the drainage structure at the study area. The current configuration with the extended wingwalls was presented in Figure 3.2 and Figure 3.5

The results of the hydraulic modelling in HEC-RAS presents the cross sections (at the drainage structure inlet) to indicate the water surface profile/level (WS-PF). This is a key outcome of the hydraulic modelling carried out in this study since flooding is defined at the limit state or the overtopping of the travelled way by the flood water. In addition, the longitudinal profile is also presented to show the behaviour of the flow regime across the drainage structure.

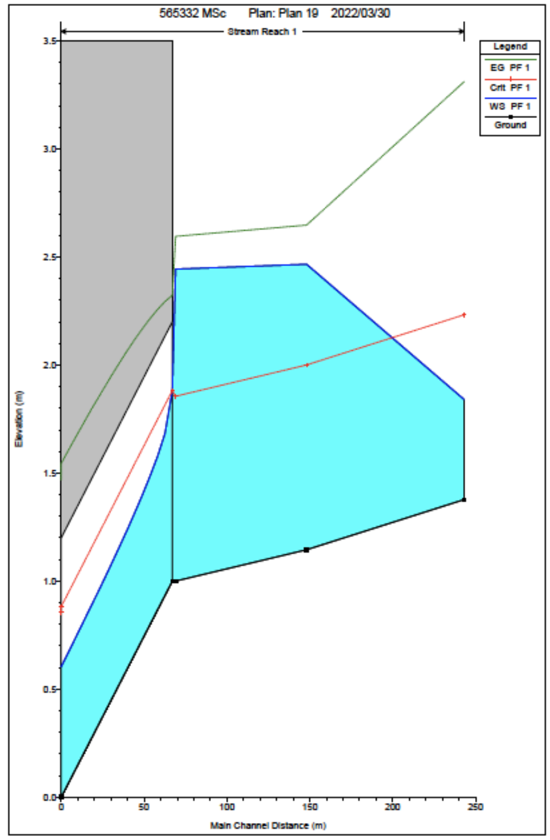
The available capacity at the drainage structure (post-upgrade) for the ideal scenario are determined below where: a) the H_w/D ratio is observed and, b) for the limit state scenario which checks for the maximum available capacity without overtopping, respectively.

a) Determining the Post-Upgrade Capacity Whilst Observing H_w/D

From the energy equation, the relationships for flow rate under inlet control and the upstream permissible headwater depth of 1.44m, the flow rate is determined as 12.45m^3/s. This design flood is analysed in HEC-RAS for the post upgrade drainage structure configuration and the design flood capacity of the current configuration at the study area is confirmed as 12.45m^3/s. Figure 4.2 below shows the cross-section upstream of the drainage structure and the longitudinal profile for a flow depth of 1.44m and a design flood of 12.45m^3/s.



Cross-Section Upstream of Drainage Structure.



Horizontal Profile.

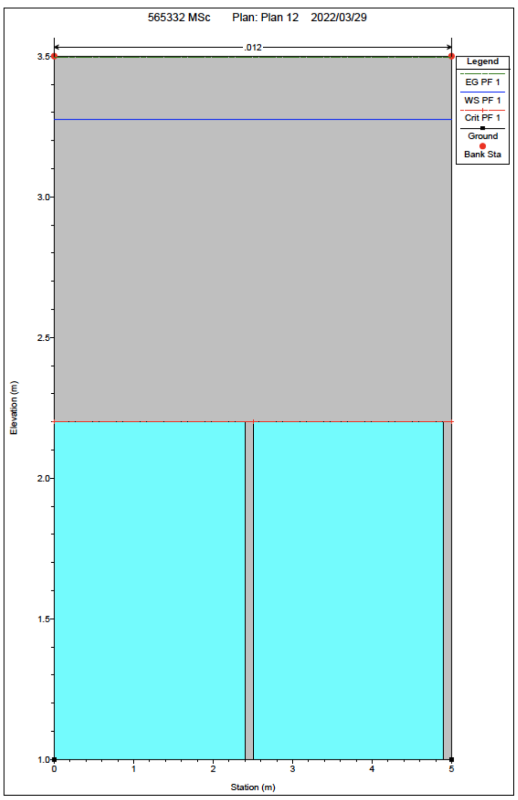
Figure 4.2: Water Surface Profile from HEC-RAS for a Design Flood of 12.45m^3/s

b) Determining the Post Upgrade Capacity for the Maximum Possible Flood Without Overtopping

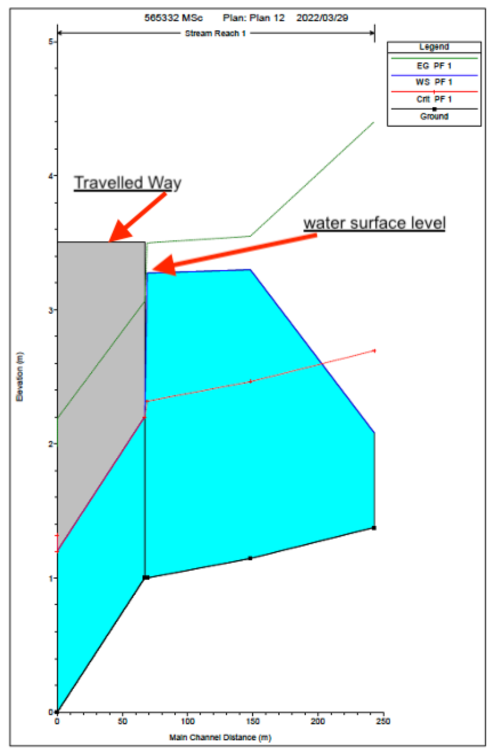
When the headwater to depth (H_W/D) ratio is ignored, the maximum allowable headwater upstream is 2.5m. This maximum allowable normal depth of flow of 2.5m is determined by the level of the travelled way shown in Figure 3.2 for the post-upgrade configuration. The hydraulic model gives an ultimate limit of

23.80m^3/s for the design flood which corresponds to a normal flow depth of 2.27m at the inlet of the drainage structure.

The upstream cross-section and profile of the drainage structure is shown in Figure 4.3 which shows a maximum water surface profile upstream of 2.27m. This is less than the available depth of 2.5m because the flow regime is outlet controlled, since a hydraulic jump occurs within the barrel, resulting in full flow conditions within the barrel. The post upgrade condition therefore accommodates a maximum design flood of 23.80m^3/s. Beyond 23.80m^3/s, overtopping of the roadway occurs for the post-upgrade condition. Figure 4.4 illustrates the overtopping of the travelled way if the flood flow is increased slightly to 23.90m^3/s.

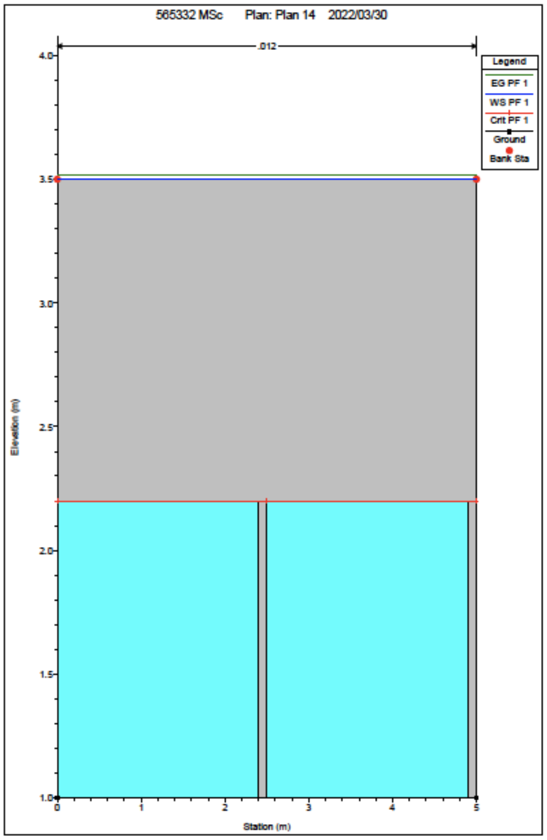


Cross-Section Upstream of Drainage Structure.

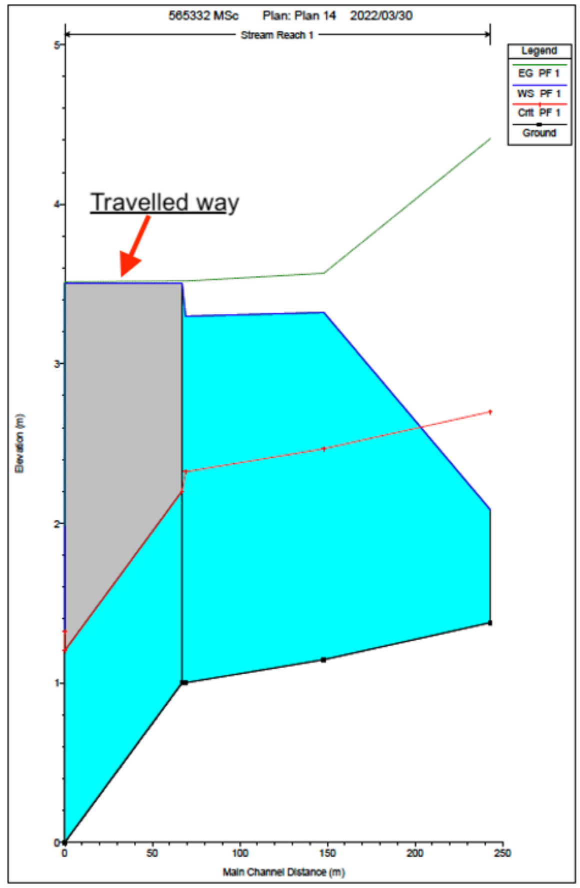


Longitudinal Profile.

Figure 4.3: Water Surface Profile from HEC-RAS for a Design Flood of 23.8m^3/s



Cross-Section Upstream of Drainage Structure.



Longitudinal Profile.

Figure 4.4: Water Surface Profile from HEC-RAS for a Design Flood of 23.9m^3/s

4.4.2 *Comparing Computed Theoretical Design Floods to the Available Capacity*

The available design flood capacity established above is compared to the design flood from the different return periods previously computed and presented in Table 4.1 using the Rational and SDF Methods.

The analysis in Table 4.3 below shows that the currently installed drainage structure is inadequate for a 2-year design flood as well as the recommended design intervals using the rational method i.e., 80-year return period. The SDF method shows that the post upgrade configuration has adequate capacity for a 2-year return period with marginal capacity for a 5-year return period. In the comparison in Table 4.3 below, the available capacity which disregards the headwater/depth ratio is adopted here to compare the overtopping (limit state) conditions for the case of the existing drainage structure.

Table 4.3: Comparing Available Capacity Against Design Peak Flows

Capacity of Existing Drainage Structure (m^3/s)	**Return Period (years)**						
	2	**5**	**20**	**50**	**64**	**80**	**100**
23.8	**Rational Method Design Flood (m^3/s)**						
	41.5	57.3	98.76	135	140.2	147.45	187
	% Difference						
	-54.2	-82.6	-122	-140	-142	-144	-155
	SDF Method Design Flood (m^3/s)						
	6.67	23.26	56.84	83	89.79	96.81	104.95
	% Difference						
	+112	+2	-82	-111	-116	-121	-126

An adequate drainage structure has also been determined to highlight the discrepancies in the required and available capacity. An adequate drainage structure for the site can have many configurations and some of these are described below.

The theoretical design flood using the Rational method (147.45m^3/s) is used as a conservative estimate for this sizing. There are nomographs provided in the SANRAL drainage manual which can be used as a guide when sizing drainage structures, and these suggest using a combination of 3m x 1.8m culverts for the required design flood at the drainage structure.

Table 4.4: Sizing for the Appropriate Drainage Structure

<table>
<tr><th>Step</th><th colspan="2">DESCRIPTION</th></tr>
<tr><td>1.</td><td>A. Calculate new allowable upstream flow depth, assuming that the vertical profile of the road is to remain unchanged i.e., $H_W/D=1.2$ for $H_W=2.5$ gives a maximum culvert depth dimension (D) of 2.08m.</td><td>B. The width for a flow depth of 2.08m and a Q of 147.45m^3/s is 7.78m from the Chezy relationship. Using the upstream slope.

The proposed channel dimensions at the entrance are now 2.08m x 7.78m.</td></tr>
<tr><td>2.</td><td>A. Calculate new downstream flow depth for a stream width of 7.78m</td><td>B. $Y_D=1.7$m.</td></tr>
<tr><td>3.</td><td colspan="2">If a diameter greater than 1.7m is chosen, which is greater than the downstream flow depth, then the inlet control conditions will likely prevail. A culvert with a depth of 1.8m is chosen. This changes the H_W to (1.2*1.8) =2.16m.</td></tr>
<tr><td>4.</td><td colspan="2">The available Q per square culvert for an H_W/D of between 0 and 2 as provided in Table 7.2 of the drainage Manual, is 14.6m^3/s.

If a culvert with dimension 3m x 1.8m is to be used, the number of culverts will have to be 10 i.e., 14,6x10 is 146m^3/s

4 culverts of a similar dimension are quoted in the National Assembly (2017) as a possible solution. Using 4 culverts of (3m x 1.8m), thus Q per culvert is 36.9m^3/s. This is not adequate for the upstream control(subcritical) flow regime.</td></tr>
<tr><td>5.</td><td colspan="2">Other configurations are however possible, but their applicability would be in consideration of other factors from the road authority. The</td></tr>
</table>

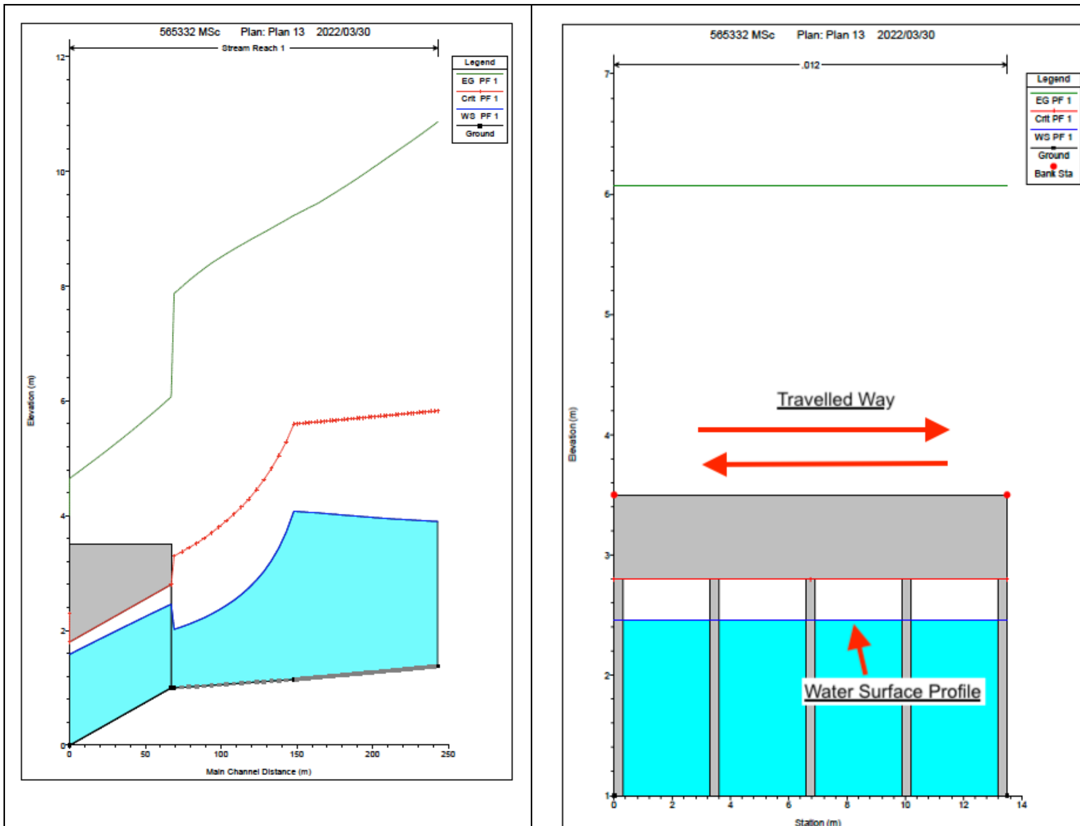
Step	DESCRIPTION
	assumption made here is that the vertical profile of the existing road is maintained, and the findings show that 10 by (3m x 1.8m) culverts would be required.

The study commissioned by SANRAL, National Assembly (2017), proposed that a new configuration of 4 culverts measuring 3m x 1.8m be installed. This configuration was hydraulically analysed using HEC-RAS and was verified to be adequate. However, a hydraulic jump forms inside the culvert making the outlet control conditions valid for this configuration.

The upstream water surface profile and cross-section for this configuration is shown in Figure 4.5 below and confirms that this configuration will not result in the overtopping of the travelled way at a peak flood of 147.45m^3/s for the 80-year return period determined using the Rational Method. In Figure 4.5, the sudden drop in the water surface elevation reflects the widening of the stream to accommodate the four 3m x 1.8m culverts which has been modelled by interpolating between the new drainage structure and the upstream station. However, the results of the modelling indicate that the upstream cross-sections would have to be extended vertically for the computed water surface to be accommodated, as such just replacing the culverts with the ones specified in the report by SANRAL in the National Assembly (2020) is not adequate without attending to the upstream depth of the channel.

The proposal by SANRAL is to also add a, "*retention/stilling pond upstream, linking the existing channel to the new culvert system*." This alteration of the stream could be designed to accommodate the elevated water surface of the proposed culverts. It was discussed in the literature review that for class 1,2 and 3 roads, which includes the class 1 road being investigated for this study, the drainage manual prescribes a freeboard requirement such that, "*Freeboard to shoulder breakpoint shall be zero or positive for a flood having twice the recurrence interval*" (SANRAL, 2013). This effectively requires that the proposed design would accommodate the 160-year flood at an estimated peak flood of 149.2m^3/s (SDF Method). The SDF method is used to determine the 1:160 flood because as highlighted in Table 2.3, the maximum flood from the rational method is the 1:100-year flood.

The results obtained for this study recommend 10 (3m x 1.8m culverts) culverts and a widening of the channel. These results are relevant for an inlet control condition with subcritical flow.



Long-section Profile

Upstream Water Profile

Figure 4.5: Water Surface Profile for Proposed Configuration by SANRAL in National Assembly, (2020)

4.5 Analysis of Historic Flooding Events

Previous flooding events at the study area are limited to 3 events for this investigation i.e., the events of the 2nd of March 2017, the 21st of January 2016, and the 9th of November 2016. These events were identified through an online search which brought up other flooding events at the study area. The investigation observed that media reporting is dominated by the flooding events at the Gillooly's Interchange, situated 1,9km west of the study area.

Reporting on flooding instances at the study area can be mischaracterized as flooding at the Gillooly's interchange given the proximity (see Figure 3.1) of the two locations and how traffic disruptions between them are interlinked. It was therefore necessary to also consider flooding reports at the Gillooly's interchange for this study.

Media reports of flooding have been corroborated using data obtained from the South African weather Services (SAWS) and will be presented here. The data requested was for the O.R Tambo Station, Johannesburg Botanical Gardens, Johannesburg Turffontein, and the Alexandra depot stations. This request was based on the proximity of the weather stations to the study area as shown in Figure 4.6 below. The stations for which there was data provided by the SAWS have been highlighted in green in Figure 4.6 and further clarified in Table 4.5 below.

Table 4.5: Summary of Data Provided by the SAWS

SAWS Station Name	SAWS Station Number	Information Requested (Yes/No)	Information Received (Yes/No)	Distance from Catchment Centroid (km)
ALEXANDRA DEPOT	0476156 X	Yes	No	8.3
JOHANNESBURG - TURFFONTEIN	0476044	Yes	No	14.1

SAWS Station Name	SAWS Station Number	Information Requested (Yes/No)	Information Received (Yes/No)	Distance from Catchment Centroid (km)
JOHANNESBURG INT W	0476399	Yes	Yes	7.7
JHB BOT TUINE	0475879	No	Yes	16.4
ZUURBEKOM AWS	0475528B7	No	Yes	38.0

Alexander (2001), cited in Gericke & DuPlessis (2012), states that, "*an eyeball estimate of the location of the catchment centroid is adequate*." The provided distances in Table 4.5 above are based on an eyeball estimate of the centroid of the catchment. The data provided was in 5-minute timesteps. As has been shown in Table 4.5 and Figure 4.6, the data from the Johannesburg Int WO is the most relevant to this study owing to its proximity to the centroid of the catchment of the study area.

Of the stations under analysis, only the Johannesburg Int WO station recorded a heavy rainfall event which was on the 9th of November 2016. A heavy rainfall event is defined as >50 mm of rainfall in 24-hours according to SAWS (2013). The relevant rainfall data as well as the maximum storm depths have been extracted from the data provided by the SAWS and are presented later in this chapter. A comparison with the gridded values, to be obtained using the design rainfall model by Smithers & Schulze (2002), is also carried out to categorise these rainfall events by their return period.

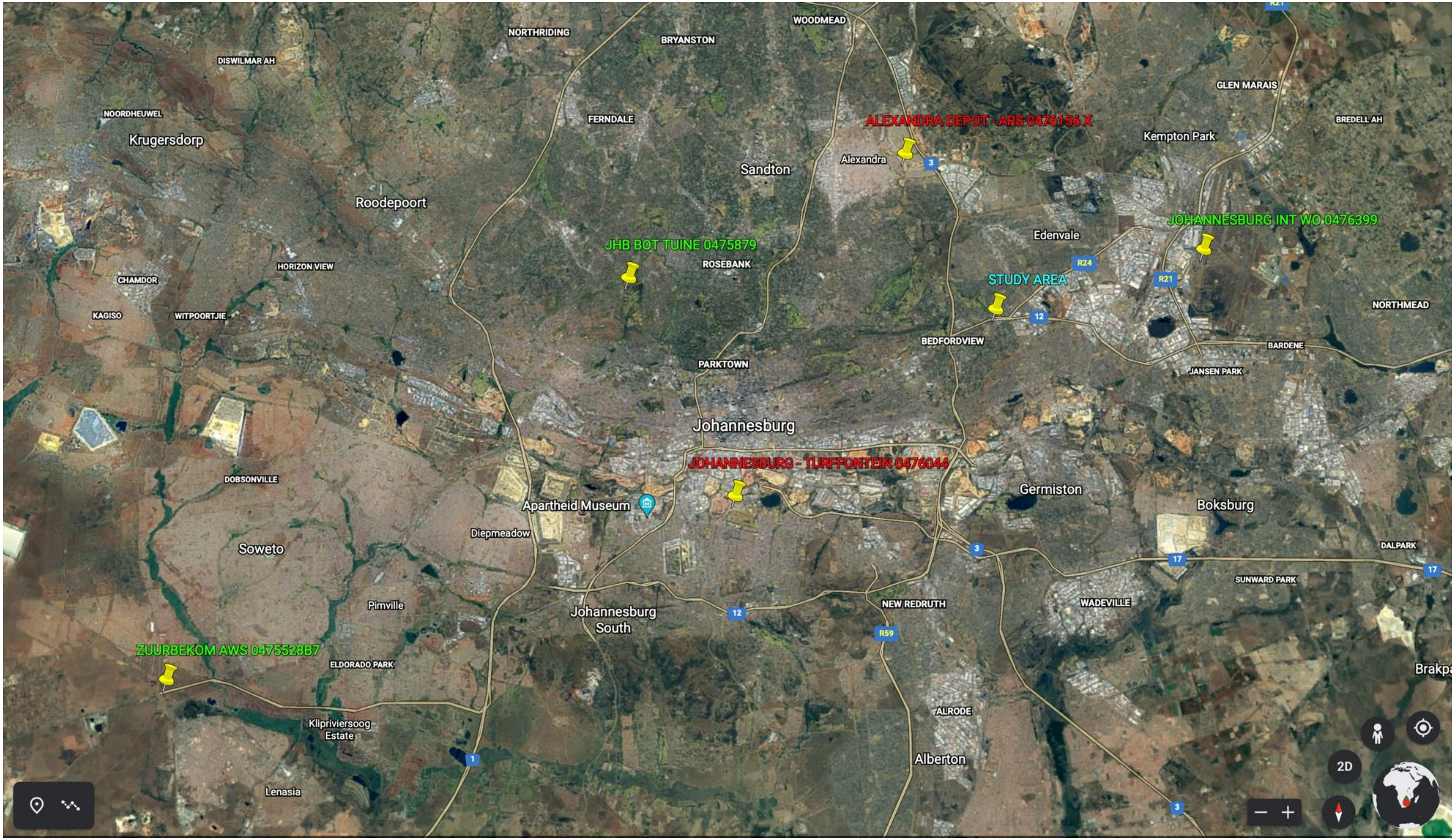


Figure 4.6: Proximity of Study Area to SAWS Stations (Provided Station Data is Shown in Green, Data Requested but Not Provided is in Red)

4.5.1 *The 21 January 2016 Flood*

Maximum values for the rainfall depths at 5-minute intervals were extracted from the time series provided by SAWS for a 1-hour duration. These values were plotted in Figure 4.7 below to give a presentation of the rainfall intensity. Of the 3 events being analysed, the event of the 21st of January 2016 had the least intensity (8.6 mm/hr) and corresponding reporting on the event is limited to the traffic disruptions as highlighted in Figure 4.7 below.

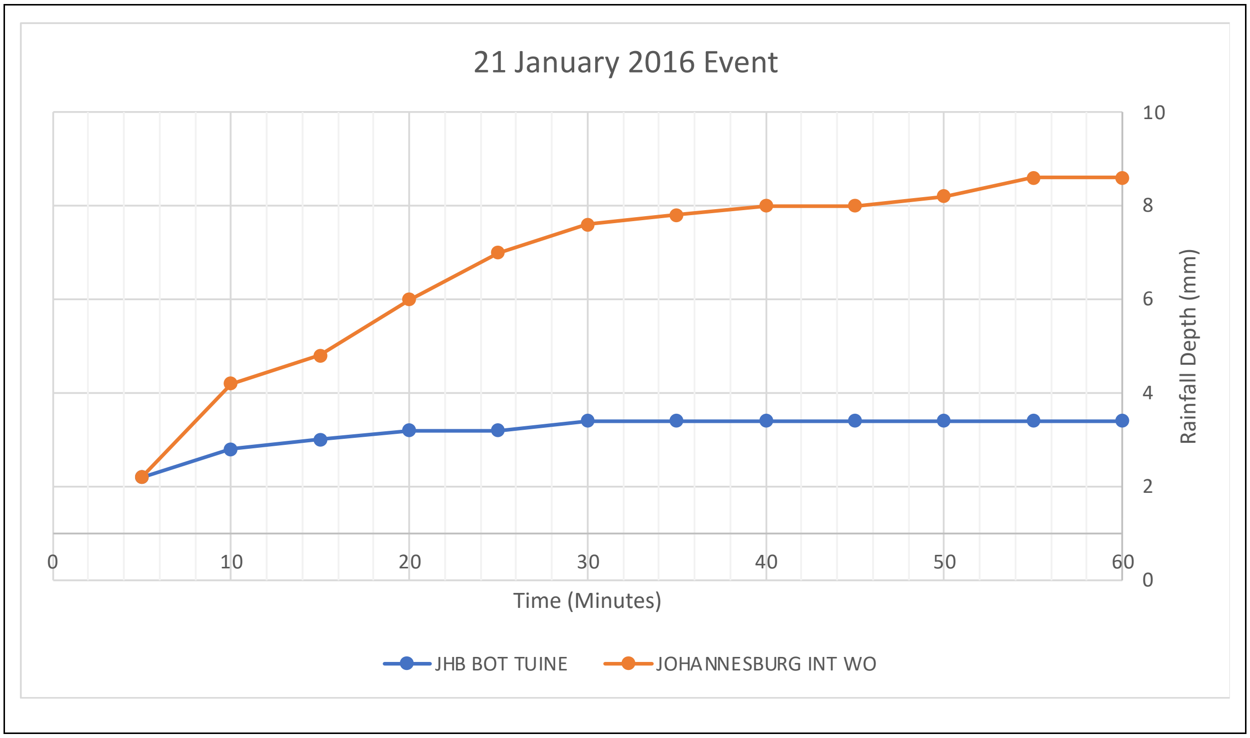


Figure 4.7: January 21, 2016, Rainfall Intensity

Several media reports on the flooding on this day are available online and an excerpt has been provided in Figure 4.8 below.

21 January 2016 19:03

Weather South Africa ?@WeatherSA issued an alert warning: "Multiple reports of flooding in parts of eastern Johannesburg. Bedfordview area particularly affected".

The flooding affected traffic on the N3/N12/R24 in all directions, at Gilloolys Interchange, causing heavy delays.

Rob Byrne ?@TrafficSA tweeted: "Avoid: Gilloolys Interchange – Flooding Heavy queues on N3 South, N12 West and R24 West – N3 North slow from Geldenhuis".

Figure 4.8: Extract from Media Report on the 21 January 2016 Event (Sowetan, 2019)

Figure 4.9 shows the flooding at the Gillooly's interchange due to this rainfall event.

(Twitter, 2016)

(Twitter, 2016)

(Twitter, 2016)

Figure 4.9: Flooding at the Gillooly's Interchange on 21 January 2016

4.5.2 *The 9 November 2016 Event*

This rainfall event was the most severe of the three with a recorded intensity of greater than 50 mm in a 24-hour period which fits the categorisation as a heavy rainfall event according to SAWS (2013). Figure 4.10 shows that during the rainfall event, the O.R Tambo station received up to 83.4 mm in 60 minutes.

This rainfall event, due to its severity, triggered the attention of the responsible authorities as outlined in Figure 4.11 below. Figure 4.11 below demonstrates that SANRAL largely blames the flooding on the intensity of the rainfall event. There are also suggestions to increase the capacity mentioned in Figure 4.11. Maintenance issues are however ruled out as a potential cause of flooding at the study area, which is contrary to the findings of section 3.6.3 of this study.

Figure 4.11 also indicates that during the 2010 upgrades of the N12, the drainage structure was not upgraded. Media notices like Figure 4.11 below, support the problem statement that, " *Changing rainfall patterns and intensity are often considered as the main causes of flooding events without due consideration to other causes of flooding*". Other studies have since been concluded regarding the flooding at the study area including a formal parliamentary response, (National Assembly, 2017), as was presented in chapter 2.5.1

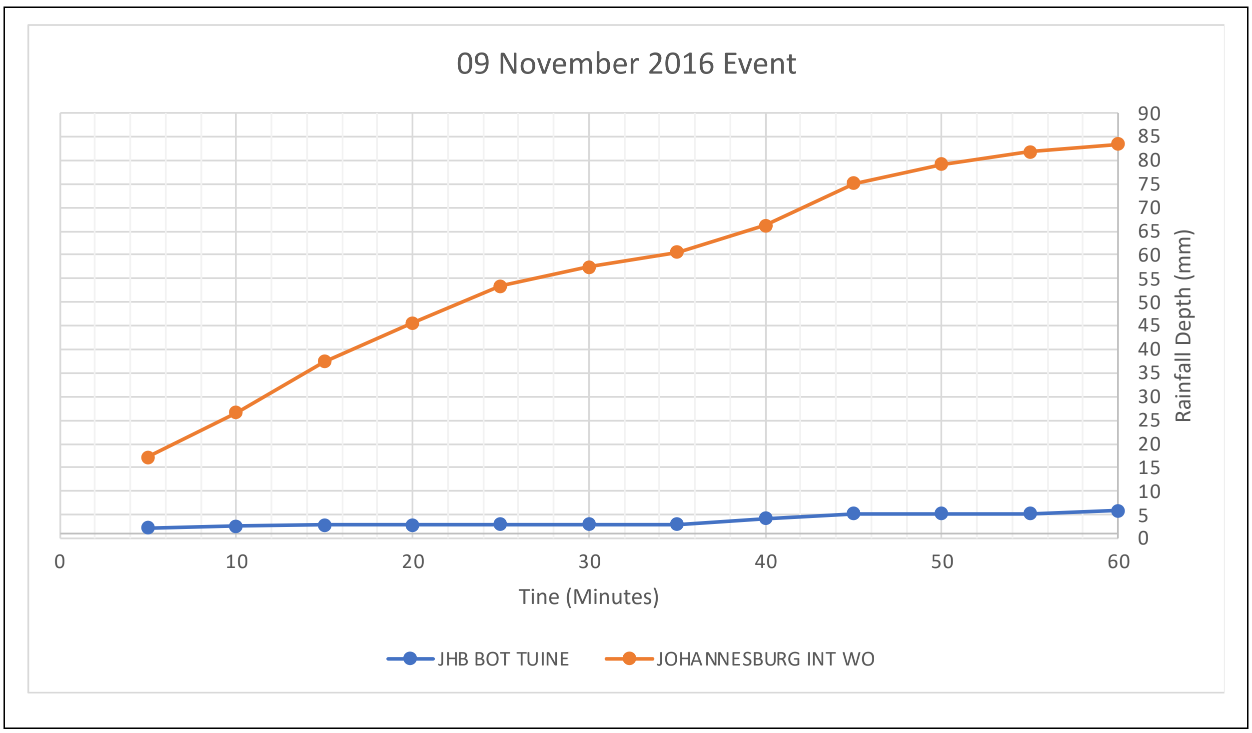


Figure 4.10: November 9, 2016, Rainfall Intensity

6 March 2018

"Emanating from the report is that the R24/N12 culvert system is inadequate to handle the recommended design of a one-in-80-year flood.

"The questions that I will be asking in Parliament is how was this allowed to happen and why the culverts were not upgraded at the time of the highway network upgrades, ahead of the 2010 World Cup?" Waters said.

Sanral responds:

In response to an enquiry by the NEWS, general manager for communications at Sanral, Vusi Mona, said a team of engineers inspected its network in Gauteng following the flash floods on November 9.

"The independent report confirmed that the flood was an exceptional 1:200 year (one in 200 year) flood," Mona said.

He also told the NEWS that maintenance of the culverts within Sanral's jurisdiction is regularly done but the culvert system is not designed for major floods such as those which occurred.

"After every significant rainfall event, culverts are checked and cleared of debris. The independent report did not highlight maintenance as a problem," Mona said.

"Improvements to the drainage systems is, however, being looked at. However, a holistic approach with all spheres of government has to be adopted, as solving upstream floods may cause flooding of houses downstream," he said.

Figure 4.11: Extract from Media Report on the 9 November 2016 Event (Edenvale News, 2018)

A study by Simpson & Dyson (2018) carried out a detailed analysis of the rainfall event of the 9th of November 2016. Their findings reveal that, "*this was the third-highest rainfall reported at the airport since 1978*" (Simpson & Dyson, 2018). The study by Simpson & Dyson (2018) further highlighted the variability of rainfall by indicating that the botanical gardens weather station, 24km west of O.R Tambo station, had received only 8mm of rainfall on the day. The study also shows that 8kms downstream of the study area, the Alexandra depot station received 16% less rain than the O.R Tambo Station over the 24hr period of the 9th of November 2016.

Simpson & Dyson, (2018) explained the meteorological origins of the event on the 9th of November 2016 as a combination of a rapidly developing, slow-moving thunderstorm with a strong updraft and elevated amounts of instability. The study also generated a Meteosat Second Generation (MSG) High Resolution Visible (HRV) image, valid for 9 November 2016 which demonstrated that the rainfall event was largely stationary and explained how this contributed to the excessive rainfall depths. Figure 4.12 shows photographs of the flooding at the N12 section directly passing the study area on the 9th of November 2016.

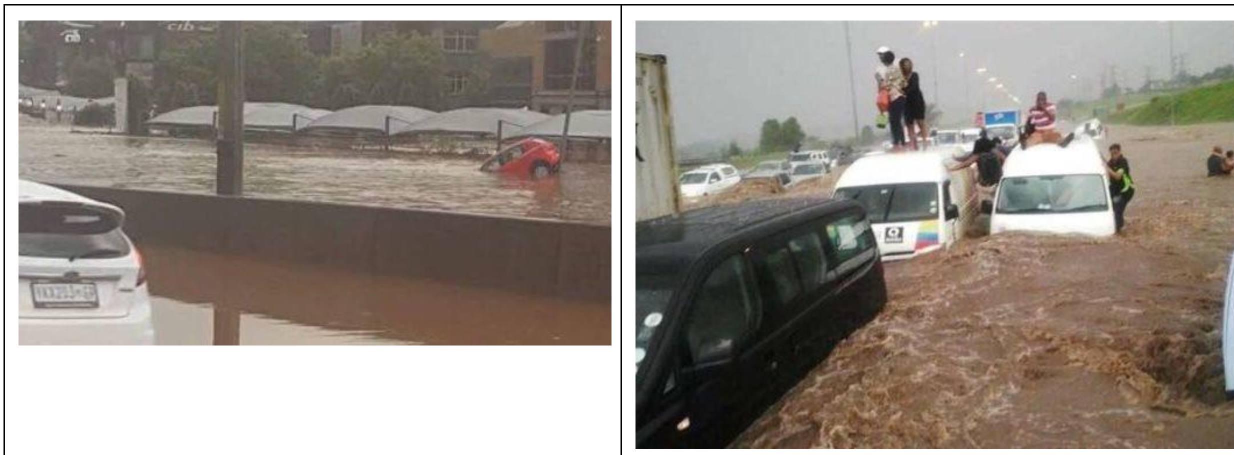


Figure 4.12: Flooding on the N12 near the CIB Insurance on Riley Road (Comaro Chronicle, 2017)

4.5.3 *The March 2017 Rainfall Event*

The intensity (20.6 mm/hr) of this event is midway when compared with the other two events being analysed in this study. The Johannesburg International Airport station recorded very minimal rainfall for this event as shown in Figure 4.13.

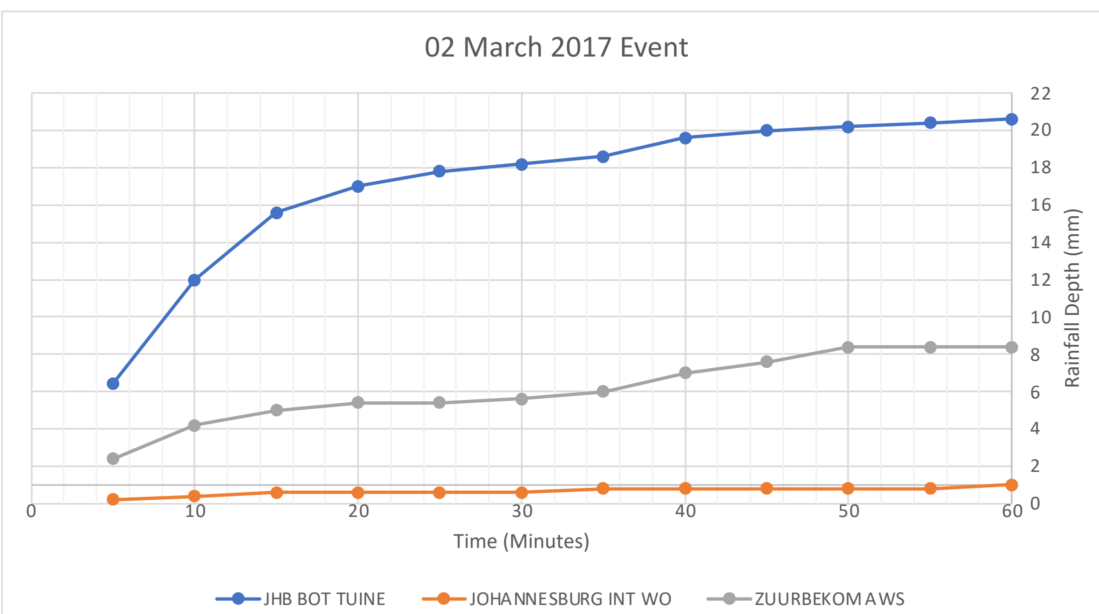
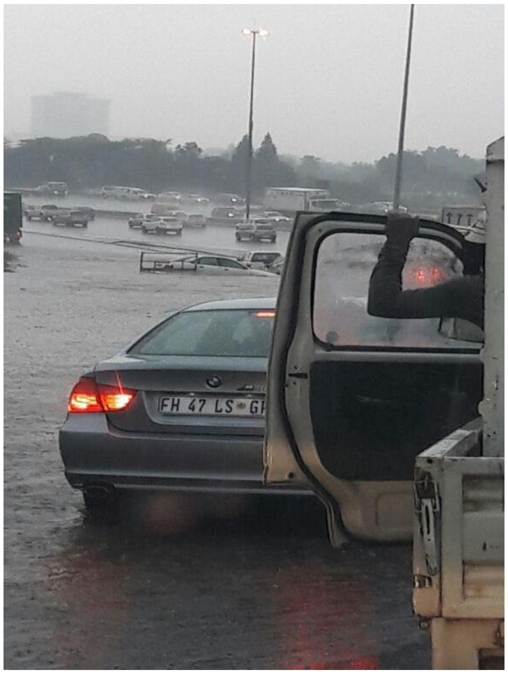


Figure 4.13: March 02, 2017, Rainfall Intensity



(Arrive Alive, 2017)

(Jacaranda FM, 2017)

Figure 4.14: Flooding at the Gillooly's Interchange on 2 March 2017

4.6 Categorisation of Rainfall Events

A technical report by Fourth Element (2017) compared the maximum values extracted from the time series of the November 9th, 2016 rainfall event with design rainfall by Smithers & Schulze (2002). Fourth Element (2017) was able to establish that the rainfall event exceeded the recommended 80-year design beyond the 30-minute period for the storm of the 9th of November 2016 using the data from the Johannesburg Int WO, see Table 4.6 below.

This study repeated the categorization of the storm depths as shown in Table 4.6 and extended this for the events of the 2nd of March 2017 and the 21st of January 2016 to categorise the return periods of these events as well.

Table 4.6: Summary of Maximum Storm Depths and Estimates of Equivalent Return Period (Fourth Element, 2017)

9-Nov-2017 Data		**Approximate Return Period based on (RLMA&SI)**
Max 5mins	17,2mm	21 years
Max 10mins	26,6mm	27 years
Max 15mins	37,4mm	50 years
Max 30mins	57,5mm	140 years
Max 45mins	75,2mm	>200 years (*325 years)* *
Max 60mins	83,4mm	>200 years (*325 years*) *

**Estimated from the plotted data in the Fourth Element* (2017) *report*

The parameters (gridded reference) of the data generated from the design rainfall model by Smithers & Schulze (2002) was unclear in the Fourth Element (2017) study. The researcher could not repeat the observations by Fourth Element (2017) for the November 9th event. The results of this study are presented in Table 4.7 and the percentage differences between this study and the Fourth Element (2017) study, for the 9 November event, are also presented below.

Table 4.7: Approximate Return Periods Based on RLMA by (Smithers & Schulze, 2002)

Duration	**Approximate Return Period based on Regional L-Moment Algorithm**				
	21 January 2016	09 November 2016	02 March 2017		% Difference for 9 Nov 2016 when compared to Fourth Element (2017)
Max 5mins	0,6 years	18,5 years	1,8 years		-13%
Max 10mins	0,7 years	43,2 years	1,9 years		46%
Max 15mins	0,6 years	46,1 years	2,0 years		-8%
Max 30mins	0,7 years	107,1 years	1,8 years		-27%

Max 45mins	0,7 years	196,9 years	1,7 years		-49%
Max 60mins	0,7 years	197,2 years	1,6 years		-49%

No meteorological analysis could be found in literature for the rainfall events of the 2nd of March 2017 and the 21st of January 2016. The analysis of the rainfall event on the 2nd of March 2017 was carried out using data from the JHB BOT TUINE station, 16,4 kms away from the centroid of the catchment. This was because on the day, this station received significantly more (+180% see Figure 4.13) rainfall than the Johannesburg Int WO. The assumption for this in this study is that the storm on the day may have originated west of the study area and that this may not have been a stationary storm as was the case for the storm on the 9th of November.

The data request in the design rainfall model was for a block with the coordinates; Latitude 26°10'S and Longitude 28°10'. This location is 370 m from the catchment centroid and is shown in Figure 4.15 below.

Figure 4.15: Location of Specified Grid in Design Rainfall Model in Reference to the Catchment Centroid and Study Area

From the analysis of the event rainfall, the events of the 2nd of March 2017 and the 21st of January 2016 do not exceed the 2-year return period at any point within the 60-minute duration as shown in Table 4.7 and Figure 4.16 further below, as such, flooding from these events would not have been expected at a structure designed for the 80-year flood for this catchment, based on the return period of the rainfall events.

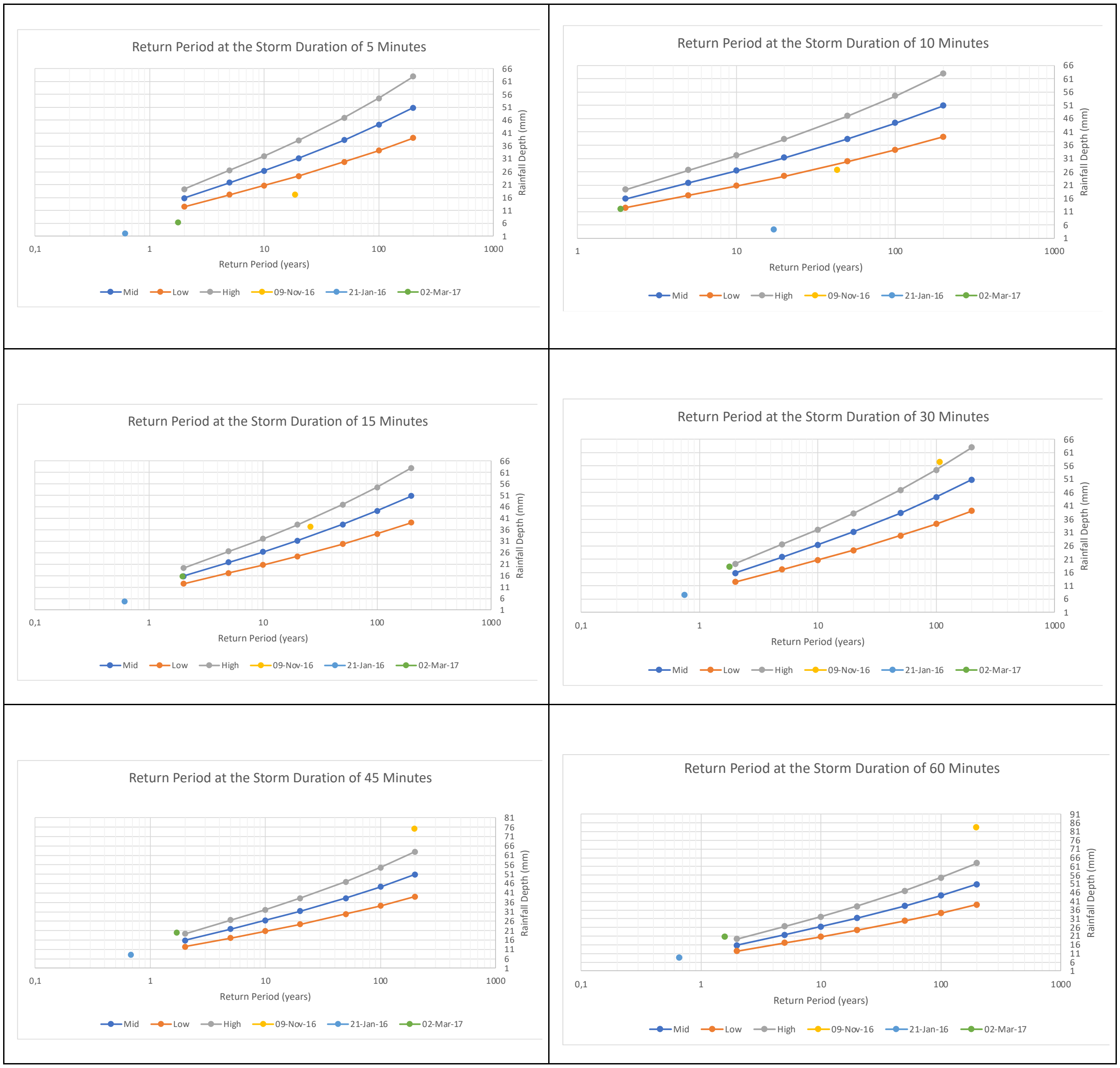


Figure 4.16: Determining the Return Periods for Event Rainfall Data from SAWS by Interpolating Between the Return Periods from the Design Rainfall Model.

As outlined in Table 4.7, the study also estimated the return periods for the event of the 9th of November 2016 which had been carried out by Fourth Element, (2017). This study showed huge differences to the findings of the Fourth Element (2017) study, particularly at the 10-minute, 45-minute and 60-minute duration where differences were ~50%. This difference might be due to the analysis by Fourth Element, (2017) using a different specified block and subsequently generating different gridded values from the design rainfall model.

Fourth Element (2017) did not specify the location for which the gridded values used in their analysis were generated. The event rainfall data was however the same as both data sets were obtained from the SAWS for the same station. The event of the 9th of November 2016 still exceeded the recommended design period of 80-years, beyond the 30-minute storm duration. The gridded values used in this analysis have been appended to this report as Appendix 1.

The design rainfall model by Smithers & Schulze (2002) assigns rainfall depths to a return period for the selected coordinates. This design rainfall model utilising the Regional Linear Moment Algorithm and Scale Variance (RLMA&SI) has already been discussed in the literature review in this report.

The design rainfall model generates for a given return period the lower, middle, and upper bound rainfall depths against the return period. Event rainfall maximum depths from SAWS were interpolated using the values generated from the design rainfall model to obtain the return periods for the rainfall events. Figure 4.16 plots the rainfall depth against the return period for all 3 events to demonstrate the return periods of the events being analysed in comparison to the lower, middle, and upper bounds from the design rainfall model for 5,10,15, 30, 45 and 60-minute maximum rainfall depths.

4.6.1.1 *Categorisation of Rainfall Events based on Rainfall Intensities*

The rainfall intensities from the event data at the 60-min duration are summarised in table Table 4.8 below. They have been cross-referenced to the rainfall intensities derived from the SDF and Rational methods for the corresponding return periods. This is necessary because a comparison of the event rainfall return period to the return period corresponding to the drainage structure design does not account for the land phase of the hydrological cycle. A rainfall event with a given return period does not necessarily translate into a peak flood of the same return period when accounting for the variables within the catchment.

Table 4.8: Estimating the Return Period of Rainfall Events from the Design Rainfall Methods

Event	**Intensity from SAWS**		**Return Period Estimated from Rational Method**	**Return Period Estimated from Rational Method**	**RLMA&SI from** Table 4.7
21 January 2016	8.6mm/hr		< 2-yr	< 1-yr	0.7-yr
02 March 2017	20.6mm/hr		< 2-yr	< 2-yr	1.6-yr
09 November 2016	83.4mm/hr		~ 41-yr	~ 27-yr	197,2-yr

Table Table 4.8 above suggests that the intensities of the rainfall events experienced at the study area were within the theoretical design limits of the drainage structure i.e., 41-years < 80-year return period.

4.7 Summary

This chapter has determined the adequacy of the existing drainage structure, as well as the theoretical design flood. The Rational method yielded higher values of the peak flow which would have resulted in a more conservative design. However, the design flood obtained using the SDF method, though less conservative, still cannot be accommodated at the study area with the current configuration of the drainage structure.

The results show that the drainage structure at the study area is inadequate by a magnitude of 168% when compared to the design flood of the rational method and that they are inadequate by a magnitude of 150% when compared with the design flood from the SDF method. Previous precipitation events have been investigated and apart from the event of the 9th of November 2016, the other two events would not have been expected to result in flooding had the drainage structure met the intended design return period of 80-years when categorised using the RLMA&SI method. However, when the intensities of the experienced rainfall events are cross-referenced with the SDF and Rational methods, flooding would not have been expected from all three rainfall events.

The impact, at the study area, of future rainfall events of a similar intensity to those analysed in this chapter was also established. This report has demonstrated that the current configuration of the drainage structure is inadequate for a 2-year flood, yet the categorisation of the rainfall events showed that at least 2 of the 3 events returned a return period of >2-years, which is more than the available capacity at the drainage structures. As such flooding would be expected for the post-upgrade configuration of the drainage structure from the same rainfall events except the January 2016 event.

The results of the SDF method however show marginal adequacy at the 5-year return period i.e., 23.26m^3/s available capacity <23.80m^3/s required capacity for a 1:5-year storm (see Table 4.3). When analysed according to the SDF method, none of the rainfall events would have been expected to result in flooding.

The literature review has discussed findings by Pregnolato, et al., (2017) who found that existing approaches fail to capture the interactions between floodwater and the transport system, noting that, "*typically a road is considered either fully operational or fully blocked which is not supported by observations"*.

The media reporting on the flooding events confirms these findings, to an extent. It is not until the event of the 9th of November 2016 that the responsible authorities respond to instances of flooding at the study area. This was because for that event, the road was fully blocked. For the analysis of the results in this report, flooding has been taken as the overtopping of the drainage structures and of the travelled way (limit state).

CHAPTER 5

The Influence of Maintenance and Climate Change on the Hydraulic Reliability of Drainage Structures

5.1 Overview

This chapter evaluates the influence of other considerations such as climate change, risk and catchment characteristics on the peak flow which was determined in chapter 4.2. The analysis carried out thus far has determined the pre- and post-upgrade design floods at the study area. In this chapter, hydraulic modelling in HEC-RAS is used to contextualise the vulnerability to flooding at the drainage structure from debris and blockages. This chapter responds to Research Aims 2 and 3.

5.2 Introduction

The previous chapters have informed that there have been some upgrades at the study area since the flooding events analysed in the previous chapter took place. The effect of the upgrades on the available capacity at the drainage structure will be evaluated using hydraulic modelling in HEC-RAS. Further, the hydraulic model will be used to investigate the effect of debris and blockages at the inlet of the drainage structures.

Analysis of the flooding at the study area, particularly during overtopping scenarios will be offered based on the hydraulic modelling and the field observations. This is intended to offer support to the approach towards investigating flooding proposed in this report. Each flooding scenario is unique in terms of the behaviour once the available capacity has been exceeded. Other aspects with an influence on the design flood are evaluated for their influence on the peak flow. Such aspects include variables which are not necessarily rigorously prescribed by the SANRAL Drainage Manual but are left to the discretion of the designer for application in accordance with the uniqueness of each drainage structure.

5.3 The Significance of the Proposed Methodology in this Report to Identifying the Causes of Flooding

The influence of debris on the reliability of drainage structures is often overlooked. Following the floods of the 5th of October 2020 which affected the M1 Double Decker Bridge (see Figure 5.1), a lot of concern was raised from various quarters (City Press, 2020), (Times Live, 2020). The Double decker Bridge is within a 12km radius of Gillooly's Interchange and is managed by the Johannesburg Roads Agency (JRA). The City Press reported that "*Parliament was asking the Johannesburg Roads Agency to account for the flooding given that a R169 million rehabilitation had recently been carried out at the bridge*" (City Press, 2020).

The report by the consultants on the project argued that the sizing of the drainage system was adequate (Calliper Consulting Engineers, 2020). However, from the images supplied in the report, Figure 5.2, the susceptibility of the inlets on the bridge to debris and litter is apparent, owing to the small diameter of the grid inlets.

Figure 5.1: Flooding of the M1 Double Decker Bridge Lower Deck (Southbound-05 October 2020) (Rosebank Killarney Gazette, 2020)

Figure 5.2: M1 Double Decker Bridge Kerb Inlet (Calliper Consulting Engineers, 2020)

The JRA now implements a rigorous cleaning regime particularly in the rainy season to ensure that the inlets for the drainage system are free of debris and litter, see Figure 5.3 below. There also could be some correlation between the high traffic volumes along the double decker and the slow speed of the traffic to the amount of litter thrown from moving vehicles in traffic. This could be a focus for future studies.

The review of the case at the M1 Double Decker Bridge shows that stormwater designs must be sensitive, not only to meet the design capacities but other elements of the environment in which the drainage system is to operate. This gives significance to the proposed methodology in this report to develop an approach to investigate the causes of flooding which considers the integrated nature of flooding.


Joburg Roads Agency · Dec 8, 2020
@MyJRA · Follow
Replying to @MyJRA
The Regional Operations team will focus on cleaning kerb inlets; addressing emergency issues such as flooding 'hotspots' and making safe pavement failures and potholes.

Joburg Roads Agency
@MyJRA · Follow

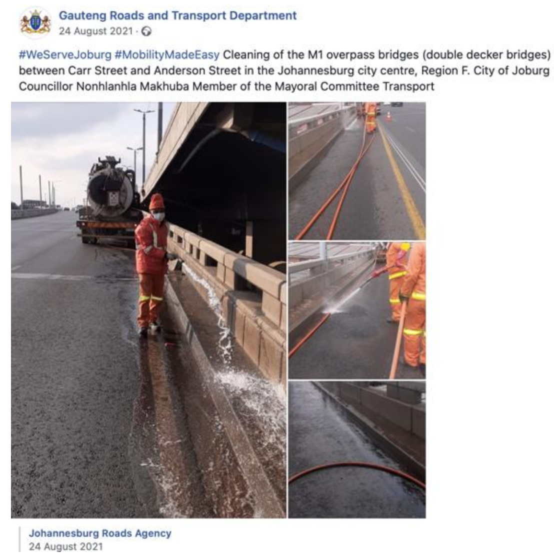
Motorways depot will continue to ensure daily cleaning of the M1 Double Decker, including litter picking at the M1 and M2.

6:45 PM · Dec 8, 2020

1 Reply Share

Read 2 replies

(JRA, 2020)



(GDRT, 2021)

Figure 5.3: Media Statements Reflecting the Implementation of Cleaning Regimes to Counter the Influence of Debris on the Reliability of Drainage Structures

The report on the flooding at the M1 double decker section also referenced other causes such as the M1 Double Decker Bridge lower deck having a drainage point midway along the bridge. This is in fact a low point of a much wider catchment which includes water incoming from the M2 bridges which had not undergone similar rehabilitation at the time. This highlights the need for evaluating the catchment when investigating flooding at an area of interest.

5.4 Analysis of the Improvements Done by SANRAL

From the available literature and investigations at the study area, it was established that the changes to the drainage structure consisted of raising the wingwalls and the headwall and were carried out in 2019. This was presented in Figure 3.2 and Figure 3.5 of this report. The modelled cross-section for this pre-upgrade condition is shown in Figure 5.4 below and shows that for the inlet, the maximum possible flow depth without overtopping is 1.5m which is less than the determined normal flow depths for the channel.

The headwater ratio is not applied in this analysis since the aim is to establish the available capacity just before overtopping. For an upstream water surface profile of 1.5m, the design flood is 12.7m^3/s. When this design flood is compared to the new capacity of 23.8m^3/s as was determined in section 4.4.1., it can be concluded that the current upgrades at the study area amount to an 87.4% improvement in the capacity of the drainage structure, from a design flood of 12.7 m^3/s to a design flood of 23.8 m^3/s.

This improvement limits the impacts from the less severe (<1:2-year) instances of flooding since according to Table 4.3, the new design flood capacity is still inadequate for the 2-year return period. However, for the SDF method, the upgrades improve the capacity up to a 1:5-year return period. This might explain why there have not been widely reported instances of flooding following the upgrades in 2019, if any.

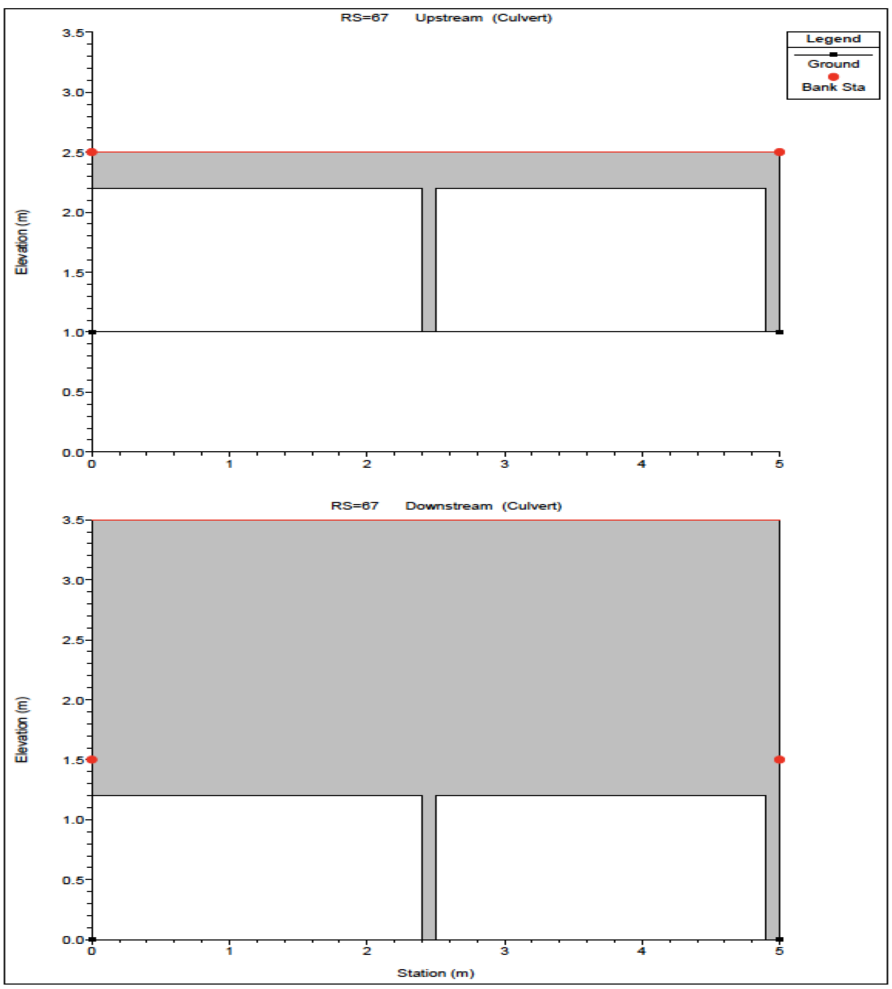
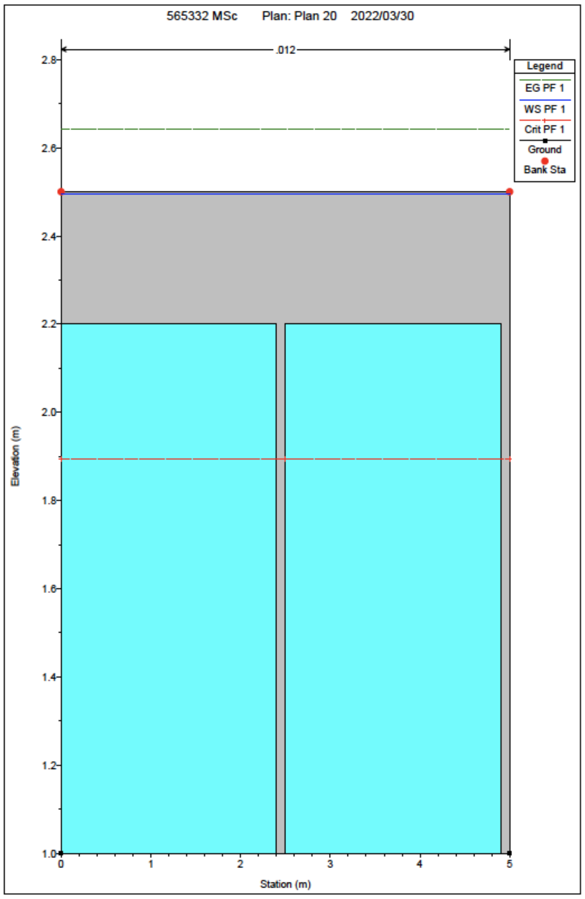
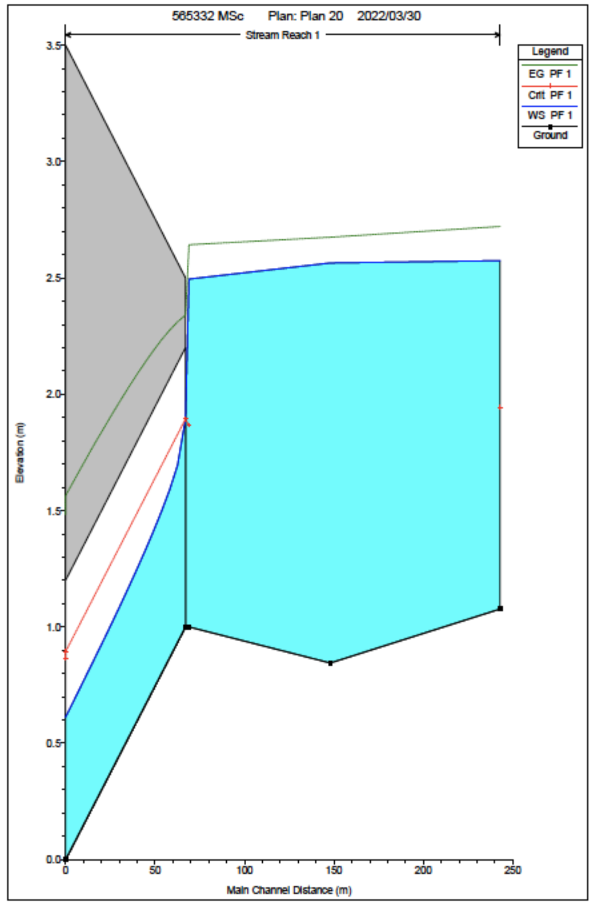


Figure 5.4: Cross-section Upstream and Downstream of the Drainage Structure at the Study Area (Before the Upgrades highlighted in Figure 3.5 and Figure 3.6)

Figure 5.5 shows the results of the hydraulic model for the pre-upgrade configuration at the inlet of the drainage structure. The figure also confirms that a design flood of 12.7m^3/s will be at the limit of overtopping of the structure in the pre-upgrade scenario.



Cross-section at Inlet Showing Water Surface Profile at 1.5m



Longitudinal Profile

Figure 5.5: Hydraulic Model Results for the Maximum Capacity for the Design Flood of 12.7m^3/s for the Pre-upgrade Configuration

5.5 Assessing the Vulnerability to Blockage and Debris

The effect of debris at the study area has been analysed using hydraulic modelling. In the aftermath of the flooding on the 9th of November 2016, SANRAL issued a statement which amongst other factors blamed the homeless "*who had used concrete blocks to build a shelter in a culvert at Gillooly's."* (SANRAL, 2016).

The analysis in this report based the depth of debris on the observed debris trapped along Boeing Road as was shown in Figure 3.14, location F. This is also the typical blockage from a tree branch during a storm. Debris has been observed at the study area during one of the inspections and this is shown in Figure 5.6 below. The entrance to that drainage structure was blocked by ~100mm of debris and this has been analysed using hydraulic modelling for the drainage structures at the study area. These results are presented in Figure 5.7 and they show that the upgraded drainage structure will not overtop the travelled way for a 100mm blockage.

 From Figure 3.4 Location B	 From Figure 3.4 location F

Figure 5.6: Debris Build Up at the Inlet of the Study Area Drainage Structure (See Figure 3.4)

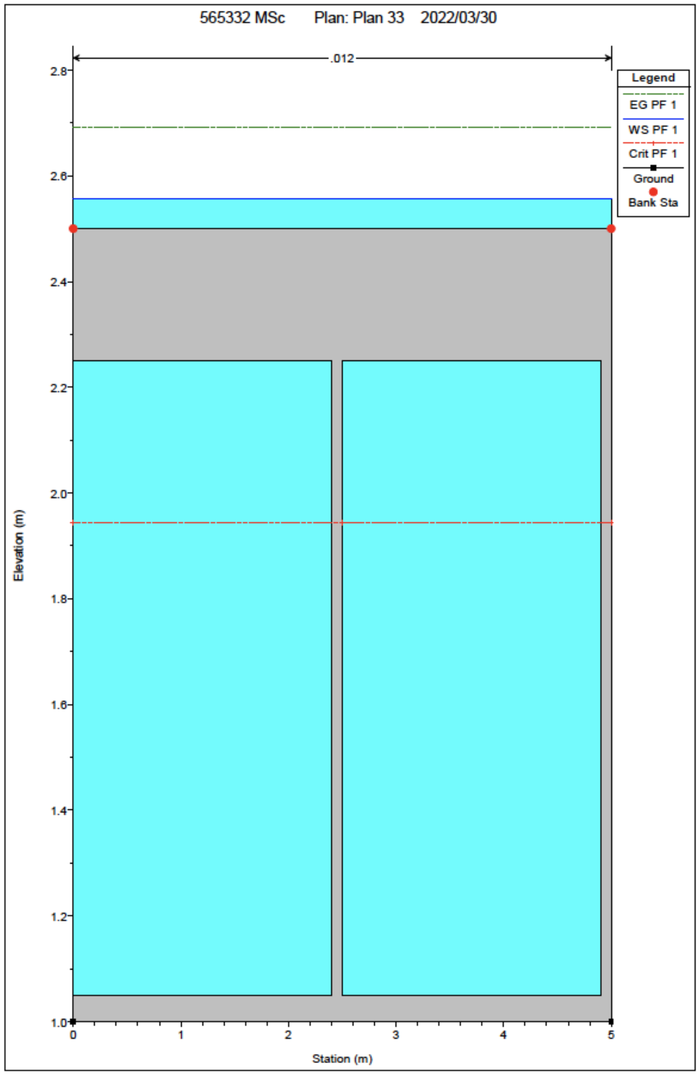
The pre-upgrade configuration will result in overtopping when analysed for a design flood of 12.7m^3/s which was deemed as the available capacity. The findings show that the pre-upgrade configuration will result in overtopping if there is to be a blockage of either 50mm or 100mm across the width of the inlet of the drainage structure.

HEC-RAS has the functionality to model blockages; however, this was not used since doing so was adding to the model of the cross-section a 100mm blockage on the downstream end as well. Instead, a 100mm blockage was modelled by editing the upstream invert elevation as shown in Figure 5.8 below. The use of a depth of 10 cm was adopted from Figure 3.1 (b). The debris is modelled for the pre-upgrade and post-upgrade scenarios using the determined available capacities of 12.7m^3/s and 23.8m^3/s, respectively.

This blockage does not impact the post-upgrade scenario significantly regarding the water surface profile. However, from a 50mm blockage, the pre-upgrade scenario results in the overtopping of the travelled way. Table 5.1 computes the differences between the water surface profile levels with those obtained in Figure 4.3 and Figure 5.5 for the post-upgrade and pre-upgrade configurations of the drainage structure, respectively.

Table 5.1: Comparison of Water Surface Profiles with Debris Modelled

Configuration	**Water Surface Profile Without Blockage**	**Water Surface Profile With 50mm Blockage**	**Water Surface Profile With 100mm Blockage**
Post Upgrade (simulated design flood: 23.8m^3/s)	2.5m	2.42m	2.27m (outlet control)
Pre-Upgrade (simulated design flood: 12.7m^3/s)	1.50m	1.56m (overtopped the 100mm blockage and the result here is for 50mm blockage)	Overtopped



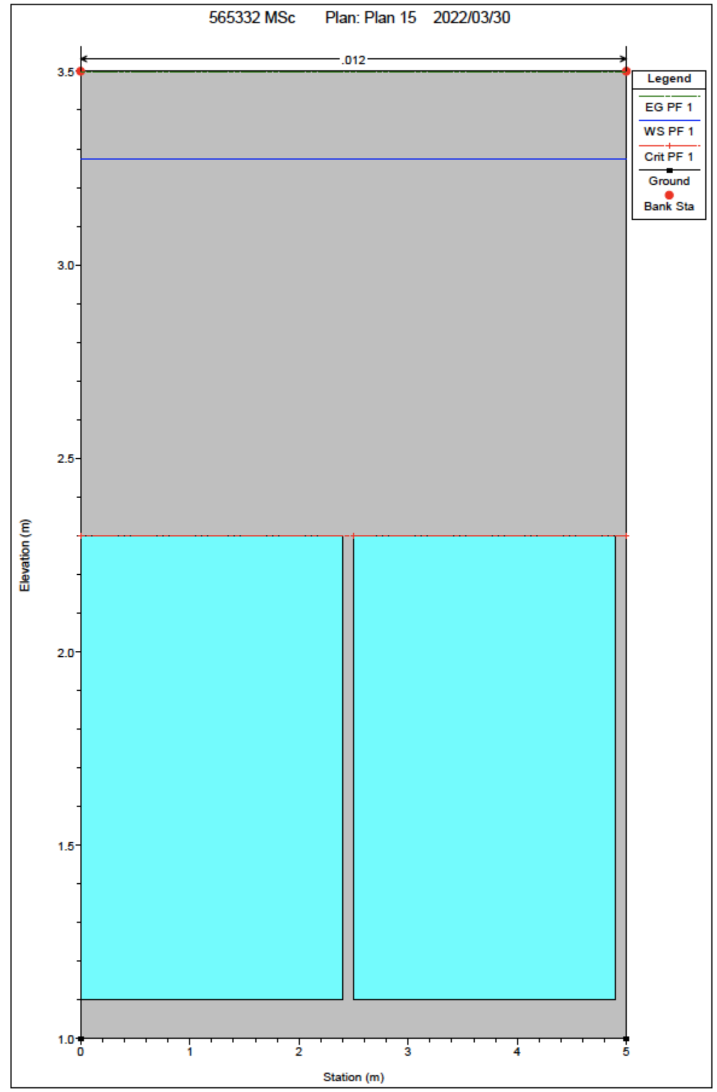


Figure 5.7: Hydraulic Analysis of the Drainage Structures Assuming 100mm Blockage (Pre-upgrades Left, Post Upgrades Right)

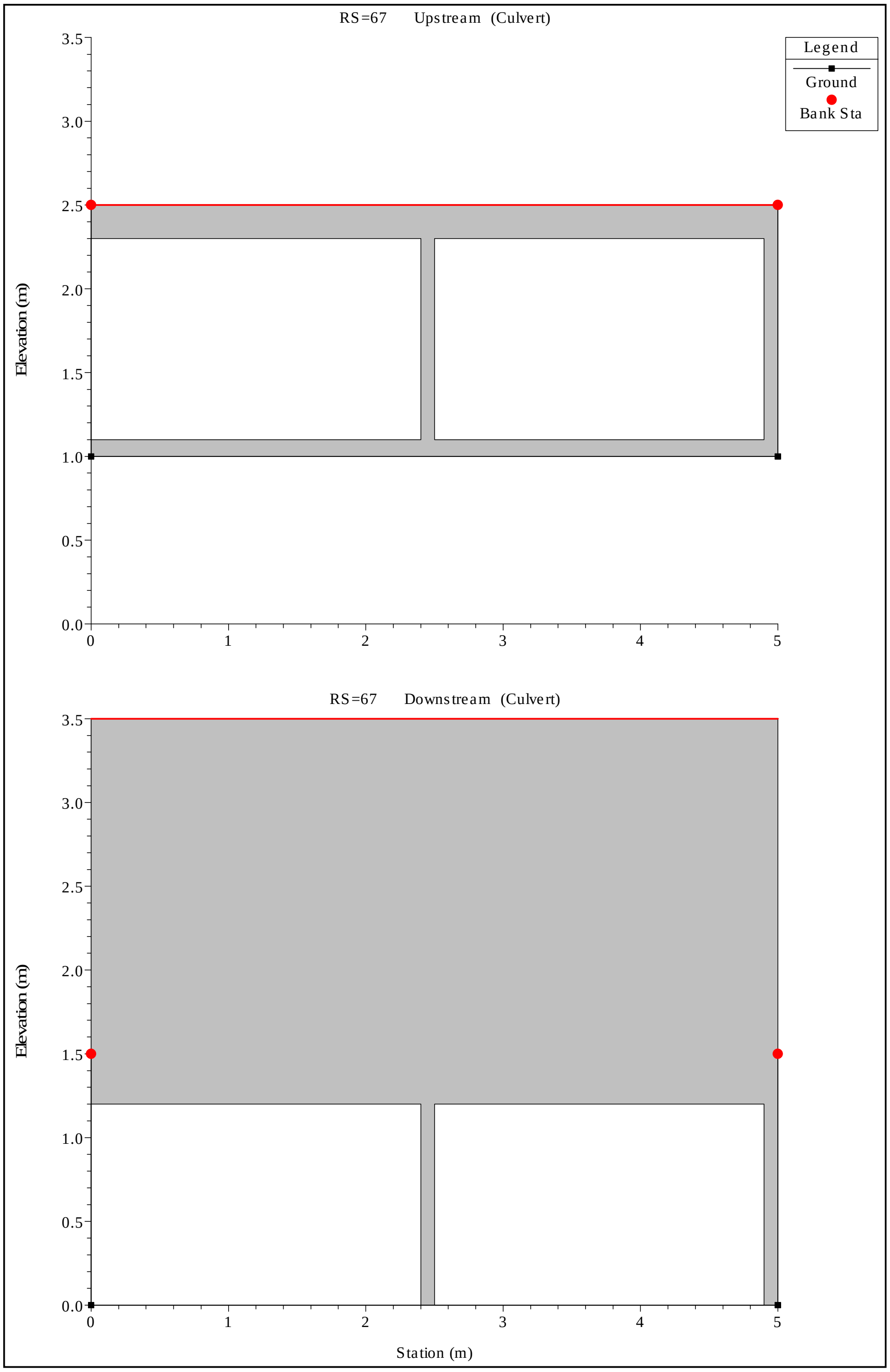


Figure 5.8: Cross-section of the Blocked Drainage structures (Shows the Existing Configuration with a Higher Headwall)

5.6 Future Scenarios

The drainage structure as it is currently configured will result in overtopping at or beyond the 2-year return period. One of the key findings of this investigation has been how flooding events that do not result in a total blockage of the travelled way, do not receive the same attention from the relevant authorities as the ones that do.

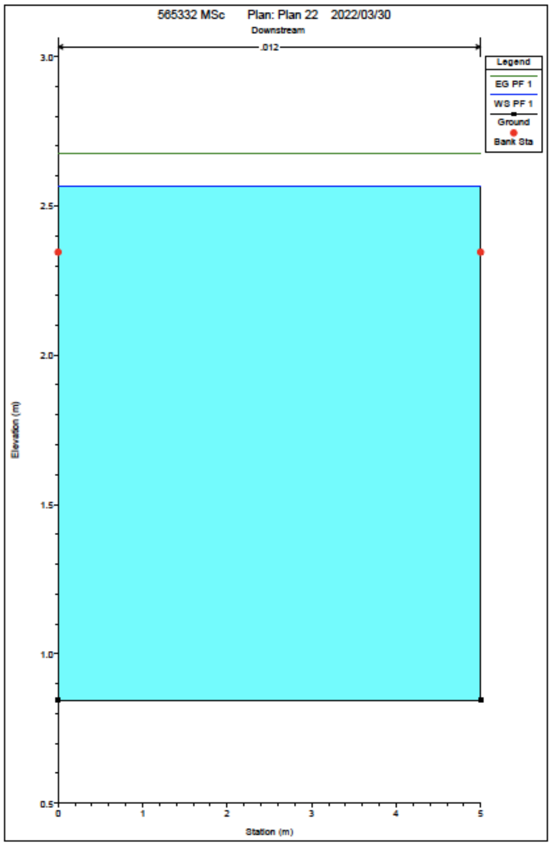
It is possible that there have been multiple scenarios in which the structure has been overtopped since it has been upgraded but as shown in Figure 5.9 below, the layout of the inlet in relation to the travelled way is such that even when overtopping occurs, these effects do not readily impact the travelled way. The cambered nature of the travelled way toward the inlet and the surrounding banks is responsible for this. This damming effect mimics an attenuation pond and for the less severe events with shorter return periods, the drainage structure performs in a seemingly adequate manner. As the analysis has shown, despite an under design of 144%, it took an extraordinary event to reveal these design deficiencies.

Figure 5.9: Inlet of the Drainage Structure at the Study Area.

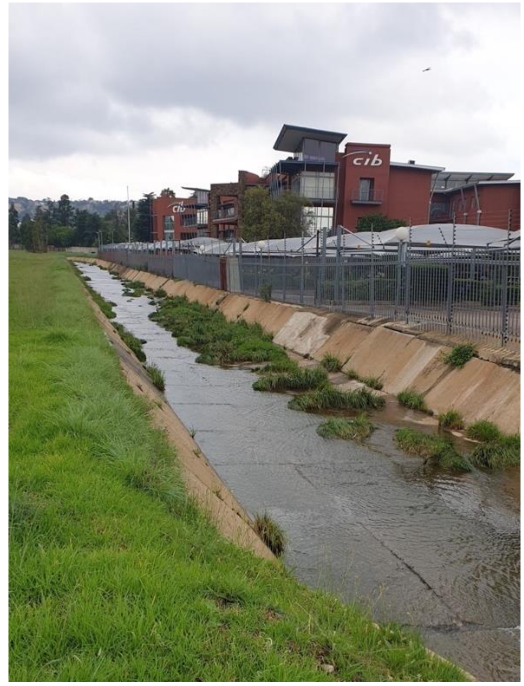
This phenomenon is more pronounced for the flooding event of the 9th of November 2016 where the jersey barrier acted as a headwall which allowed traffic flow in the Eastbound direction as shown in Figure 5.10.

Figure 5.10: Flooding at the Study Area (Bedfordview and Edenvale News, 2016)

In section 5.2 above, it was determined that for the pre-upgrade scenario, a maximum design flood of 12.7m^3/s would be accommodated prior to overtopping. However, this observation does not consider that the normal upstream depth exceeds the channel dimensions within the upper reaches as shown in Figure 5.11 below. This shows that the quoted flood of 12.7m^3/s with a narrow focus on the overtopping of the road fails to cater for the upstream effects. This could result in the flooding of the properties as shown in Figure 5.11 below.



The water surface profile exceeds the banks (red dots) of the pre-upgrade configuration at 1.5 from the surface bed at the upstream reach at this cross section.



Surrounding properties vulnerable to overtopping of the banks upstream of the drainage structure (see Figure 3.4 location A.)

Figure 5.11: Cross-section Exceeded by Water Surface Profile and Vulnerable Properties

This could explain why, in the interim, SANRAL raised the walls of the drainage structure to protect the properties southeast of the drainage structure. It also highlights why inadequacies in drainage structures may not always result in the flooding of the road travel way but possibly a complex response with many hidden risks. Therefore, the methodology for investigating flooding in this report which proposes the hydraulic modelling of the structures being investigated and the checking of adequacy is essential in adequately addressing flooding occurrences.

From this analysis, the behaviour of the flooding events analysed in the previous chapter may have been mitigated by the fact that overtopping would occur in a manner that impacts the function of the road once the banks of the stream were fully flooded. For as long as the road is not flooded to the point of being impassable, the action from the responsible authorities is not likely to follow. The parliamentary report (2020) further recommended that an additional four 3x1.8m culverts be added at the section including the addition of a retention/stilling pond upstream, linking the existing channel to the new culvert system. The addition of a stilling basin would serve to minimise the overtopping of the banks of the stream before the inlet. Various scenarios could be implemented to address the flooding however the solution proposed in this report excludes the use of a detention pond. This is informed by the reviewed literature which advised that detention ponds should be implemented after having done a catchment wide analysis to fully understand the downstream impact of the altered time to peak.

The methods used in this report (Rational and SDF) do not allow for a detailed analysis of the impact of the changes in land use identified in section 2.4.2.1 of this report. The land use changes constitute ~6% of the total catchment area which table 3.3 of the SANRAL Drainage Manual (2006) estimates will influence the peak flood by a factor of 1.24. Reviewed literature by Roodsaria & Chandler, (2017) has cautioned that the influence of the proximity of the imperviousness to the drainage point also influences the impact of impervious surfaces on the peak flow.

5.7 Influence of Climate Change on the Reliability of Drainage Structures

According to the United Nations (2022), climate change is defined as long-term shifts in temperatures and weather patterns. As discussed in section 2.4.1, the given definition challenges the assumption of stationarity which is dominant in water-based risk analysis.

Cheng & AghaKouchak (2014) showed, "*that a stationary climate assumption may lead to underestimation of extreme precipitation by as much as 60%."* This corresponds well with Sharma, et al., (2021) who indicated that applying factors of 1.4 to 1.7 to the pipe diameters from standard design guidelines increased the hydraulic reliability of structures at the 1:100-year return period. They then found that factors of 2.0-2.3 increased the hydraulic reliability of structures for the 1:500-year reliability.

Climate change remains a complex and dominant topic in flood modelling. The complexity of climate change is such that it impacts on topographic factors, developmental influences, and climatological variables and should accordingly be evaluated in an integrated manner. Climate change exacerbates the already difficult challenge of understanding fundamental hydrological processes as highlighted by Cordery and Pilgrim (2000). For example, changing rainfall patterns can alter settlement patterns causing either depopulation or overpopulation which affects the flooding risk profile. Separately or simultaneously, changing weather conditions can affect vegetation, influencing land cover and evapotranspiration rates and the runoff volume.

This complexity in identifying the drivers of hydrological inputs for most runoff determination methods such as rainfall and runoff coefficients and defining their relationships makes establishing the direct impact of climate change difficult. In literature, a lot of studies address climate change in the context of one variable i.e., rainfall. Piketh, et al., (2014) single out the rainfall variable when defining the influence of climate change when they say, "*climate impact as a result of increased rainfall intensity* was proportional to the size of the catchment."

This report has already shown that the influence of Climate Change on the entire hydrological process needs to be understood if the influence of climate change is to be adequately defined. For hydrological studies, the distinction between anthropogenic and natural climate change may not be necessary since both impact the reliability of drainage structures.

There is therefore not enough data, and it is beyond the scope of this report to adequately categorise the influence of climate change on the flooding experienced at the study area.

5.8 Summary

This chapter has discussed several aspects which are relevant to the reliability of drainage structures. These have been summarised in Table 5.2 below. The additional aspects (items 2 to 5) in Table 5.2 constitute an average of 45.2% increase in the calculated design peak flood.

This is significant enough to warrant the consideration of these aspects during design of drainage structures. Implementing agencies should require that designers of stormwater systems demonstrate that consideration is given to additional aspects listed in Table 5.2 after the initial peak flood has been determined.

Table 5.2: Summary of Factors with an Influence on the Reliability of Drainage Structures

Item	Description	Influence at the Study Area
1. Debris	It was shown that debris can lead to severe flooding using an example of the M1 double decker in Johannesburg. The impact of debris is influenced by the type of inlet and the influence of other activities within the catchment.	For the existing drainage structure, the influence of debris is modelled on the determined available capacity of the culverts i.e., 12.7m^3/s and 23.8m^3/s • For 100mm and 50mm thick blockages at the inlet, the pre-upgrade configuration results in overtopping of the travelled way for a 12.7m^3/ flood. • The post upgrade culvert configuration becomes outlet controlled for a modelled 100mm blockage for a 28.3m^3/s flood.
2. Freeboard	A freeboard requirement in the drainage manual requires that the drainage structure be able to handle a design	• The return period at the study area was determined at a 1:80-year return period and twice this is a 160-year return period.

Item	Description	Influence at the Study Area
	flood, twice the return period before overtopping occurs.	• The 1:160-year return period is associated with a peak flood of 149.2m^3/s for the study area. A 54.1% increase on the 80-year peak flood of 96.81m^3/s (see Table 4.3) using the SDF method (SDF method is used here since the rational method cannot be used to determine return periods beyond 1:100-yr). • This also suggests that this aspect was overlooked by SANRAL who recommended a 1:80-year design flood: "*The R24/N12 culvert system is inadequate to handle the recommended design flood (1:80).* (National Assembly, 2017),
3.Land Use	The impact of land use changes has been estimated to have an influence on the peak flow at the study area by a factor of 1.24	• A factor of 1.24 on the demand peak flood of 147.45m^3/s for the 80-year return period results in a peak flood of 182.84m^3/s an increase of 24.0%.
4. Climate Change	This report has shown climate change impacts to be complex and not readily understood	• A factor of 1.7 as proposed by Sharma, et al., (2021) has been applied to the demand peak flood of 147.45m^3/s for the 80-year return period resulting in a peak flood of 250.67m^3/s, an increase of 70.03%. • The estimated return period for the experienced rainfall event on the 9th

Item	Description	Influence at the Study Area
		of November 2016 was 197.2-years as estimated in Table 4.7 of this report. This return period translates to a peak flood of 155.41m^3/s using the SDF method. This suggests that the rainfall event of the 9th of November 2016 would have been handled by the structure if the factor of 1.7 to account for climate change had been applied.
5. Time of concentration	The time of concentration Tc estimates, such as those which consider only the main watercourse characteristics are underestimated on average by 50%. This in-turn leads to an overestimation of the peak discharge by between 30%-50%. (McCuen (2009), cited in; Smithers & Ockert, (2014))	• The Tc calculated for the Rational method of 0.9304hrs is increased to 1.39hrs resulting in a reduced peak flood of 98.3m^3/s. • The above represents a 33% change to the originally determined value of the peak flood using the rational method. • The adjusted design flood of the rational method (98.3m^3/s) brings it within 1.5% of the one determined using the SDF method of 96.81m3/s.

CHAPTER 6

Conclusions and Recommendations

6.1 Overview

This chapter concludes the research. The key findings are presented together with the limitations of the research. The conclusion also informs whether the research aims were achieved and makes recommendations for future research.

6.2 Conclusions

The research question sought to identify the causes of flooding at the study area by systematically reviewing all probable causes and assigning likelihood to them. The rainfall event on the 9th of November 2016 is largely responsible for the flooding on the day but the interactions between the event and the catchment is what significantly influenced the severity of the flooding.

It was established that the existing drainage structures did not meet the required capacity for the acceptable risk at the study area. Further, the identified changes in land use within the catchment likely had a negative impact on an already under designed structure. The pre-upgrade configuration of the drainage structure was found to be sensitive to debris and litter blockages at the inlet. However, the upgrades mitigate the sensitivity to blockages of up to 100mm at the drainage structure.

The cause of the flooding at the study area was largely a result of the inadequacy of the drainage structure which was amplified by changes in land use and maintenance lapses at the study area i.e., litter and blockages. This report highlights epistemic limitations in understanding the human and natural factors which combine in a complex process to cause flooding. These limitations make quantifying the influence of climate change particularly difficult. However, an estimate based on the reviewed literature showed that aspects such as climate change, land-use and other risk factors could influence the peak flow calculated from the drainage manual by up to 45% for the study area. This report does not carry out an in-depth analysis of all contributory factors, instead analysing what is currently the design norm, to understand current lapses at the study area within already established standards and procedures.

Other similar investigations may find compliance with all current design parameters and could be useful in exploring the vulnerability to other aspects not fully captured by current design methods nor the conditions of the selected study area. This includes depth-duration relationships, time of concentration, runoff parameters and the impacts of climate change. This report has found human factors in the design to be central to the flooding experienced at the study area. This includes the lack of coordinated catchment management, lapses in maintenance and the severely under-designed drainage structure which is contrary to design provisions. Some deterministic methods could benefit from improved data such as streamflow measurements. This could be used to harness the improved computing power of the modern era and to refine parameters which could improve the understanding of the land phase of the hydrological process.

6.3 Limitations of the Study

The following limitations, with an influence on the study outcomes were experienced.

- **Topographic data**: Detailed elevation data particularly at the drainage area would have ensured more accurate invert data for the hydraulic modelling at the drainage structure.
- **Drainage Masterplan**: Catchment interactions between the surface runoff and the piped stormwater networks could not be fully defined upstream of the drainage point. This report assumed that all piped networks would still drain at the study area justified by the fact that they are gravity systems. The effect of the piped networks on the runoff coefficient could not be captured but this was not too significant since the design flood was determined for the maximum probable flood where a coefficient of 1 was used. This report initially set out to also investigate the Gillooly's Interchange, which is 1,9km west of the study area and has more significance than the study area as it is a major interchange. The pipe networks which characterise the Gillooly's Interchange could not be determined, and this made it difficult to carry out a meaningful investigation at that location. Flood

attenuation has been discussed as one of the components requiring a catchment wide approach to avoid unintended simultaneous peaks at critical points downstream. The influence of detention on the resulting flooding could not be verified without an investigation into the current attenuation measures for the entire catchment which was beyond the scope of this report.

- **Existing Design Parameters and the Previous Design**: A portion of this investigation analyses the initial design at the study area. However, this analysis is done by proxy, looking at what is currently installed at the study area as the design outcome. This is because the original design and the assumed parameters therein were not available to the researcher. It is possible that the design was accurate and there could have been an error during the construction. Furthermore, should the design have been wrong, this report cannot identify where the original design got it wrong and use this as a lesson for future designs. Knowing the catchment area and parameters used could have also highlighted the influence of the findings in this study which relate to the changes in land use.
- **Town planning Records:** Town planning records which highlight catchment changes were not available. It was established that attenuation was a requirement for new developments in EMM. The basis upon which the EMM authorities approved the new developments and determined the attenuation needs was assumed to be based on the pre-development levels runoff levels. It was beyond the scope of this study to investigate this for each of the identified areas within the catchment that have undergone significant redevelopment as was shown through the aerial photos. However, not understanding the implemented attenuation measures at these redevelopment sites limited the influence of the analysis carried out on the changes in land use within the catchment area.
- **Flooding Reports:** Most flooding reports were obtained from the media. The responsible authorities did not have any specific information relating to the flooding and its extent. It is possible that flooding events with minor inundation of the roads were not reported in the media, which was the primary source on flooding events. These media reports did not include any relevant information regarding the precipitation event such as, the rainfall depth and the extent of the flooding.

Because the dates of the flooding were available in the reports, the corresponding event data was obtained from the SAWS. However, the extent of the flooding could not be readily established apart from the images available in the media reports. This report used hydraulic simulation to evaluate the water surface profile to quantify the extent of the flooding. This was also limited by the availability of topographic data which was inadequate to produce flood maps particularly for the pre-upgrade and post-upgrade scenarios.

The methodology used in this research could be used by road authorities to investigate flooding hotspots and to inform remedial actions. The limitations presented above can be readily overcome by the responsible catchment managers since all such information is either available or could be commissioned from other departments. This interdepartmental collaboration highlights the integrated approach required for effective catchment management.

6.4 Revisiting the Research Aims

This section analyses the extent to which the research aims have been achieved.

Aim 1: To determine the adequacy of the currently installed drainage structure and assess whether the adequacy of the drainage structure is consistent with the experienced flooding events. (Achieved- See Chapter 4)

Aim 2: To assess, based on the case study, the impact of various states of maintenance and catchment management practices on the reliability of flood drainage structures. (Achieved- See Chapters 2 and 3)

Aim 3: To contextualise the findings of the study within a wider geographic context and in consideration of the impacts of climate change. (Achieved see Chapter 5)

Most of the challenges in this research relate to the availability of adequate data and the inherent limitations associated with modelling the hydrologic process. Hydraulic modelling of the catchment is a data intensive process especially if one is to obtain high quality results which can minimise unnecessary costs from over-designed structures.

Research Aim 2 was addressed using rephotographs which quantified the extent of significant land use changes within the catchment by area. However, the lack of information on the developments limited the significance of these findings in the context of the impact on the runoff within the catchment.

This report can be useful for local authorities involved in the management of the catchment. Urban catchments can be hydraulically modelled, and such models can be used as a tool to investigate flooding, approve new developments or to understand the impact of changing rainfall patterns. Such a model can also inform the areas in need of additional data for complex urban catchments. Without adequate data on the effectiveness of current methods, particularly in the context of current climatic and physical changes, it is difficult to improve current design methods.

No design guide or stormwater masterplan is currently available for Ekurhuleni. This complicates the design response particularly for remedial work such as that required at the study area. It also hinders collaboration between different implementing agencies as is required at the study area where SANRAL would need to design for a catchment managed by Ekurhuleni. Stormwater by-laws are typically informed by the stormwater masterplan, and they were also unavailable on the Ekurhuleni Municipality's website and in the reviewed literature.

6.5 Recommendations for Future Studies

Areas which future studies may focus on are listed below:

- This is a desktop study utilising a single case study. Further studies could expand on the general applicability of the findings of this research. Studies may also be carried out in partnership with local catchment management agencies to offer more significance to the findings.
- The factors used in design such as the area reduction factors, runoff coefficients, time of concentration and other catchment parameters need to be better understood particularly for the urban environment. More studies investigating the interaction between rainfall with the catchment would improve existing flood

estimation methods. This report has also discussed the challenge posed by rainfall variability which is at odds with methods that ascribe homogeneity to various parameters i.e., the Rainfall basins of the SDF method and the IDF-curve and the Mean Annual Precipitation of the Rational Method. It should be the goal of future studies to intensify physical data collection to better capture complex hydrological processes and improve modelling accuracy.

- Road agencies use a lot of resources monitoring the traffic along routes for information on the pavement performance and life. For the drainage structures, it is also necessary to investigate the advantages of strategic monitoring of stream flows in urban areas to be able to reflect catchment changes for the attention of the responsible authorities. This could be used to understand the rainfall-runoff processes better and thereby undertake flood modelling with more confidence and less uncertainty. For example, they could be used to develop a more robust correlation between a forecasted rainfall depth with anticipated hazards in urban catchments. The viability of these measurements could be explored in future studies which may elaborate on the cost benefits of such measurements.
- The SANRAL drainage manual, though seemingly adequate for existing conditions, needs to be revised to include climate resiliency guidelines as well as other water sensitive urban design aspects. For example, when compared to the U.S. Department of Transportation Federal Highway Administration's Hydraulic Design of Highway Culverts, (2012)**,** the SANRAL drainage manual does not seem to cater to the Aquatic Organisms Passage (AOP). This may be inconsequential in some urban settings but crucial in certain (rural) catchments.
- Future studies could explore the viability of dedicated flood estimation methods focussing on ungauged urban catchments. This could also extend to new risk models that appropriately map the flooding risks to the design or peak floods. For example, certain major catchments within the Metropolitan cities and other urban centres could be subjected to a different design regime which caters to the representative risks in such areas.
- More studies are required to verify the increased variability of short duration high-intensity rainfall events, and this is evident in the literature. However, resilience

should be built upon sound designs as this can later inform future design guidelines.

- The impact of fragmented catchment management needs to be understood better to establish improved catchment management processes. Even when a catchment is under the management of a single authority, the activities of and management by various departments contribute to the ultimate runoff factor and runoff volume. The use of live catchment models can be explored as a way of understanding the complex interactions between the precipitation event and the catchment characteristics.
- A detailed study with information on the stormwater masterplan within the catchment could be carried out to verify the impact of the changes in land use on the parameters of the SANRAL Drainage Manual highlighted in this report.

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APPENDIX 1: PEAK FLOOD CALCULATION

Data Extracted from Daily Rainfall Estimate Database File

Design Rainfall in South Africa: Ver 3 (July 2012)

User selection has the following criteria:
Coordinates: Latitude: 26 degrees 10 minutes; Longitude: 28 degreess 10 minutes
Durations requested: 5 m, 10 m, 15 m, 30 m, 45 m, 1 h, 1.5 h, 2 h, 4 h, 6 h, 8 h, 10 h, 12 h, 16 h, 20 h, 24 h, 1 d, 2 d, 3 d, 4 d, 5 d, 6 d, 7 d
Return Periods requested: 2 yr, 5 yr, 10 yr, 20 yr, 50 yr, 100 yr, 200 yr
Block Size requested: 0 minutes

Data extracted from Daily Rainfall Estimate Database File
The six closest stations are listed

Station Name	SAWS Number	Distance (km)	Record (Years)	Latitude (∞)	(')	Longitude (∞)	(')	MAP (mm)	Altitude (m)	Duration (m/h/d)	Return Period (years) 2	2L	2U	5	5L	5U	10	10L	10U	20	20L	20U	50		200	200L	200U
RIETFONTEIN (GM)	0476309_W	2.5	44	26	9	28	11	718	1705	1 d	52.0	51.7	52.2	71.7	71.4	72.1	86.7	86.3	87.1	102.6	101.8	103.4	125.9	12	166.8	162.9	170.5
										2 d	64.2	63.7	64.6	89.0	88.4	89.4	108.2	106.7	109.2	129.0	126.0	131.4	159.9	15	216.2	201.1	230.0
										3 d	69.9	69.6	70.2	96.3	96.0	96.8	116.7	115.8	117.2	138.6	137.0	140.0	171.1	16	229.5	220.7	237.2
										4 d	78.3	78.0	78.7	106.9	106.4	107.4	128.7	127.7	129.3	152.1	150.4	153.5	186.3	18	247.6	238.1	255.3
										5 d	83.6	83.2	84.1	113.7	113.2	114.2	136.3	135.2	136.9	160.0	158.5	161.5	194.5	19	254.6	246.0	262.1
										6 d	88.4	88.0	89.1	120.1	119.5	120.6	143.5	141.4	144.9	168.0	164.3	171.1	203.1	19	263.6	246.4	279.0
										7 d	93.2	92.8	93.6	126.2	125.6	126.8	150.5	148.8	152.0	175.9	172.7	178.4	211.9	20	273.3	257.8	286.8
WIT DEEP (GM)	0476342_W	5.1	41	26	12	28	12	747	1645	1 d	60.5	60.2	60.8	83.5	83.1	83.9	101.0	100.4	101.4	119.5	118.5	120.4	146.5	14	194.2	189.7	198.5
										2 d	75.0	74.5	75.5	104.0	103.4	104.5	126.4	124.8	127.7	150.8	147.3	153.6	187.0	18	252.8	235.2	268.9
										3 d	82.3	81.9	82.6	113.3	112.9	113.9	137.3	136.3	137.9	163.0	161.1	164.7	201.3	19	270.0	259.7	279.0
										4 d	87.2	86.8	87.6	119.0	118.4	119.6	143.3	142.2	144.0	169.3	167.5	170.9	207.5	20	275.7	265.1	284.3
										5 d	91.8	91.4	92.3	124.9	124.3	125.4	149.6	148.5	150.3	175.8	174.1	177.4	213.6	20	279.6	270.2	287.9
										6 d	100.5	100.1	101.3	136.6	135.9	137.2	163.2	160.9	164.9	191.1	186.9	194.6	231.0	22	299.9	280.3	317.4
										7 d	106.2	105.7	106.6	143.7	143.0	144.4	171.4	169.4	173.0	200.3	196.7	203.2	241.3	23	311.2	293.6	326.6
GERMISTON-FJ PAYNE PARK	0476283_W	5.4	91	26	13	28	10	729	1663	1 d	59.1	58.8	59.4	81.6	81.2	82.0	98.7	98.2	99.2	116.8	115.8	117.7	143.2	14	189.8	185.4	194.0
										2 d	72.9	72.4	73.3	101.0	100.4	101.5	122.8	121.2	124.0	146.4	143.1	149.2	181.6	17	245.5	228.4	261.2
										3 d	81.7	81.3	82.0	112.5	112.1	113.1	136.3	135.3	137.0	161.9	160.0	163.6	199.9	19	268.1	257.9	277.1
										4 d	87.9	87.5	88.3	120.0	119.4	120.6	144.5	143.4	145.2	170.7	168.9	172.3	209.2	20	278.0	267.3	286.6
										5 d	95.7	95.3	96.3	130.3	129.6	130.8	156.0	154.9	156.8	183.3	181.5	185.0	222.7	21	291.5	281.8	300.2
										6 d	100.8	100.4	101.6	136.9	136.2	137.6	163.6	161.3	165.3	191.6	187.4	195.1	231.6	22	300.6	281.0	318.2
										7 d	106.2	105.7	106.6	143.7	143.0	144.4	171.4	169.4	173.0	200.2	196.7	203.1	241.3	23	311.2	293.5	326.6
GERMISTON-SIMMER PAN(PO W)	0476283AW	5.7	83	26	13	28	9	738	1663	1 d	53.4	53.1	53.6	73.7	73.3	74.0	89.1	88.6	89.5	105.4	104.5	106.2	129.3	12	171.3	167.3	175.1
										2 d	68.0	67.5	68.4	94.3	93.7	94.7	114.6	113.1	115.7	136.6	133.5	139.2	169.4	16	229.1	213.1	243.7
										3 d	77.3	77.0	77.7	106.5	106.1	107.0	129.0	128.1	129.6	153.3	151.5	154.8	189.2	18	253.8	244.1	262.3
										4 d	83.3	82.9	83.6	113.6	113.1	114.2	136.8	135.8	137.4	161.6	159.9	163.1	198.1	19	263.2	253.1	271.4
										5 d	90.4	90.0	90.8	122.9	122.3	123.4	147.2	146.2	147.9	173.0	171.3	174.5	210.2	20	275.1	265.9	283.3
										6 d	95.6	95.2	96.3	129.8	129.2	130.5	155.2	153.0	156.7	181.7	177.7	185.0	219.7	21	285.1	266.5	301.8
										7 d	99.9	99.5	100.4	135.3	134.6	136.0	161.4	159.5	162.9	188.5	185.2	191.3	227.2	21	293.0	276.4	307.5
GERMINSTON-SIMMER JACK- GM	0476252_W	6.5	61	26	13	28	8	761	1645	1 d	55.8	55.5	56.0	77.0	76.7	77.4	93.1	92.6	93.6	110.2	109.3	111.1	135.2	13	179.1	175.0	183.1
										2 d	70.1	69.7	70.6	97.2	96.6	97.7	118.2	116.6	119.4	140.9	137.7	143.6	174.8	16	236.3	219.8	251.4
										3 d	77.1	76.8	77.5	106.3	105.8	106.8	128.7	127.8	129.3	152.9	151.1	154.4	188.7	18	253.1	243.4	261.6
										4 d	82.7	82.3	83.0	112.8	112.3	113.4	135.9	134.9	136.5	160.5	158.8	162.0	196.7	19	261.4	251.3	269.5
										5 d	90.1	89.7	90.6	122.6	122.0	123.1	146.9	145.8	147.6	172.6	170.9	174.1	209.7	20	274.5	265.3	282.6
										6 d	95.4	95.0	96.1	129.6	128.9	130.2	154.9	152.7	156.4	181.3	177.3	184.6	219.2	21	284.5	265.9	301.1
										7 d	101.4	100.9	101.8	137.2	136.5	137.9	163.7	161.8	165.2	191.2	187.8	194.0	230.4	22	297.1	280.3	311.8
JHB-ROYAL GOLF CLUB	0476160_W	7.4	64	26	9	28	6	762	1637	1 d	51.1	50.9	51.4	70.6	70.3	70.9	85.4	84.9	85.8	101.0	100.2	101.8	123.9	12	164.2	160.4	167.8
										2 d	66.9	66.4	67.3	92.7	92.2	93.2	112.7	111.2	113.8	134.4	131.4	137.0	166.7	16	225.4	209.7	239.8
										3 d	74.3	73.9	74.6	102.3	101.9	102.8	123.9	123.0	124.5	147.2	145.5	148.7	181.7	17	243.7	234.4	251.9
										4 d	83.0	82.6	83.3	113.2	112.7	113.8	136.3	135.3	136.9	161.1	159.3	162.5	197.4	19	262.3	252.2	270.4
										5 d	88.8	88.4	89.3	120.8	120.3	121.3	144.8	143.7	145.4	170.0	168.4	171.6	206.6	20	270.5	261.4	278.5
										6 d	94.8	94.4	95.5	128.8	128.2	129.4	153.9	151.7	155.5	180.2	176.2	183.5	217.9	20	282.8	264.3	299.3
										7 d	100.6	100.1	101.0	136.2	135.5	136.8	162.4	160.5	163.9	189.7	186.3	192.5	228.6	22			

1. Gridded Values of All points Within Specified Block

Gridded values of all points within the specified block

Latitude		Longitude		MAP	Altitude	Duration	Return Period (years)																				
(∞)	(')	(∞)	(')	(mm)	(m)	(m/h/d)	2	2L	2U	5	5L	5U	10	10L	10U	20	20L	20U	50	50L	50U	100	100L	100U	200	200L	200U
26	10	28	10	730	1660	5 m	8.9	7.2	10.6	12.3	9.9	14.7	14.9	12.0	17.8	17.6	14.1	21.1	21.6	17.2	26.0	24.9	19.8	30.2	28.6	22.6	34.8
						10 m	12.8	10.1	15.5	17.7	14.0	21.4	21.4	16.9	25.8	25.3	20.0	30.7	31.0	24.3	37.8	35.8	28.0	43.8	41.1	32.0	50.5
						15 m	15.8	12.4	19.3	21.8	17.1	26.6	26.4	20.7	32.1	31.3	24.4	38.2	38.3	29.8	47.0	44.3	34.2	54.5	50.8	39.1	62.9
						30 m	20.3	16.2	24.4	28.0	22.4	33.7	33.9	27.0	40.8	40.1	31.9	48.4	49.2	38.9	59.6	56.8	44.7	69.2	65.2	51.1	79.8
						45 m	23.5	18.9	28.1	32.5	26.2	38.8	39.2	31.6	46.9	46.4	37.3	55.7	56.9	45.5	68.5	65.8	52.3	79.6	75.5	59.7	91.7
						1 h	26.1	21.2	31.0	36.0	29.2	42.8	43.5	35.3	51.8	51.5	41.7	61.5	63.2	50.8	75.6	72.9	58.4	87.8	83.7	66.7	101.3
						1.5 h	30.2	24.7	35.6	41.7	34.2	49.2	50.4	41.3	59.5	59.6	48.7	70.7	73.1	59.4	86.9	84.4	68.3	101.0	96.9	78.0	116.4
						2 h	33.5	27.6	39.3	46.2	38.2	54.3	55.9	46.1	65.7	66.1	54.4	78.0	81.1	66.4	96.0	93.7	76.3	111.4	107.5	87.1	128.5
						4 h	40.3	34.1	46.4	55.6	47.1	64.1	67.2	57.0	77.4	79.6	67.2	91.9	97.5	81.9	113.1	112.6	94.2	131.4	129.3	107.6	151.5
						6 h	44.9	38.6	51.1	61.9	53.3	70.5	74.9	64.4	85.3	88.6	76.0	101.2	108.7	92.7	124.6	125.5	106.5	144.7	144.0	121.7	166.8
						8 h	48.4	42.1	54.7	66.9	58.2	75.5	80.8	70.3	91.3	95.7	82.9	108.4	117.3	101.1	133.4	135.5	116.2	154.9	155.5	132.8	178.6
						10 h	51.4	45.1	57.7	71.0	62.3	79.6	85.8	75.2	96.3	101.5	88.8	114.3	124.5	108.2	140.7	143.8	124.4	163.3	165.0	142.1	188.3
						12 h	54.0	47.6	60.2	74.5	65.8	83.2	90.1	79.5	100.5	106.6	93.8	119.4	130.7	114.4	146.9	150.9	131.5	170.5	173.2	150.2	196.7
						16 h	58.2	52.0	64.5	80.4	71.8	89.0	97.2	86.8	107.6	115.1	102.4	127.8	141.1	124.8	157.3	163.0	143.5	182.6	187.0	163.9	210.6
						20 h	61.8	55.6	68.0	85.3	76.8	93.9	103.2	92.9	113.5	122.1	109.6	134.7	149.8	133.6	165.8	172.9	153.6	192.5	198.4	175.4	222.0
						24 h	64.9	58.8	71.0	89.6	81.2	98.0	108.3	98.2	118.5	128.2	115.8	140.7	157.2	141.2	173.2	181.5	162.3	201.1	208.3	185.4	231.9
						1 d	56.2	50.9	61.5	77.6	70.4	84.9	93.8	85.0	102.7	111.1	100.3	121.9	136.2	122.3	150.0	157.3	140.6	174.2	180.5	160.6	200.9
						2 d	69.3	64.3	74.2	95.6	88.9	102.5	115.6	107.4	123.9	136.9	126.7	147.1	167.8	154.5	181.0	193.8	177.6	210.1	222.4	202.8	242.3
						3 d	78.3	73.7	82.8	108.1	101.8	114.3	130.7	123.1	138.2	154.6	145.2	164.1	189.6	177.0	202.0	219.0	203.5	234.5	251.3	232.5	270.4
						4 d	85.3	79.7	90.9	117.8	110.1	125.6	142.5	133.1	151.8	168.6	157.0	180.3	206.8	191.4	221.8	238.8	220.1	257.6	274.0	251.4	297.0
						5 d	91.3	84.7	97.8	126.0	117.0	135.1	152.4	141.4	163.3	180.3	166.8	193.9	221.1	203.4	238.6	255.3	233.8	277.0	293.0	267.1	319.5
						6 d	96.4	89.0	103.8	133.1	123.0	143.4	161.0	148.6	173.3	190.5	175.3	205.8	233.6	213.7	253.2	269.7	245.7	294.0	309.5	280.7	339.1
						7 d	101.0	92.8	109.2	139.4	128.2	150.7	168.6	155.0	182.2	199.5	182.8	216.4	244.7	222.9	266.3	282.5	256.2	309.2	324.2	292.7	356.6

APPENDIX 2: DESIGN FLOOD CALCULATIONS

Appendix 2.1: Rational Method

In this subsection the steps towards determining the peak flood using the rational method are presented and the assumptions thereof, the steps followed in Table A2.1 below are according to the SANRAL Drainage Manual.

Table A2.1: Determination of the Peak Flow Using the Rational Method

Steps	**Calculation/Parameters**
Step 1: Catchment Area	**5.389km^2**
Step 2: Length of Longest Watercourse	**4.09km**
Step 3: Determine the Average Slope of Longest Watercourse S_{av}=Average slope m/m $H_{0.1L}$= Elevation height at 10% of the length of watercourse (m) $H_{0.85L}$= Elevation height at 85% of the length of the watercourse (m) L= Length of watercourse $S_{av}=\frac{H_{0.85L}-H_{0.1L}}{(1000)(0.75L)}$	**0.036552m/m**
Step 4: Calculate Time of concentration T_c The channel leading to the culvert under review does not span the entire channel. Its length was calculated at 949m. As such the time of concentration for overland flow and that of the defined watercourse will be summed. Overland Flow: Tc $=0.604(\frac{rL}{S^{0.5}})^{0.467}$=0.228 plus Channel Flow Components $T_c= (\frac{0,87L^2}{1000S_{av}})^{0,385}$=0.7015	T_c=0.2289+0.7015= **0.9304 Hours** Check average flow velocity(v=L/T_c) = **1.94m/s**

Figure A2.1: Defined Watercourse for Catchment A not to be Confused with Stream 1 which is the Hydraulic Length of the Catchment

Step 5: Determine the Mean Annual Precipitation The literature shows that the most conservative MAP is 740mm. However, Figure 3.7 of the drainage manual shows an average annual rainfall of between 600-800mm. The more conservative choice would be to use 800mm	**800mm**
Step 6: Determine the Point Rainfall Values (P_T) This is obtained using Figure 3.7 of the SANRAL Drainage Manual which is derived from HRU (1978), starting with the Time of concentration calculated above as the storm duration.	**85mm** for P_{50} or 50-year return period **118.18mm** for P_{100} or 100-year return period
Step 7: Calculate Point Intensity This is equal to the point rainfall obtained in step 6 above divided by the time of concentration in step 4.	P_{i2}=**26mm/h** P_{i5}=**36mm/h**

	P_{i20}=**62mm/h** P_{i50}=**91.36mm/h** P_{i64}=**95mm/h** P_{i80}=**100mm/h** P_{i100}=**127.02mm/h**
Step 8: Area Reduction Factors See Figure 3.2 of the SANRAL Drainage Manual.	ARF_2=**99%** ARF_5=**99%** ARF_{20}=**99%** ARF_{50}=**98.75%** ARF_{50}=**98.6%** ARF_{80}=**98.5%** ARF_{100}=**98.3%**
Step 9: Determine the Effective Catchment Precipitation This is the application of the factors in step 8 to the point intensities obtained in step 7 of this table.	I_2= **27.76mm/h** I_5= **38.3mm/h** I_{20}= **65.972mm/h** I_{50}= **90.218mm/h** I_{64}= **93.67mm/h** I_{80}= **98.5mm/h** I_{100}= **124.90mm/h**
Step 10: Determine the Runoff Coefficient The area under analysis is urban and hence C_2 factor from the drainage manual will apply and it will be adjusted for the maximum flood, as such, $C_{2=}1$	**C_{50} and C_{100}= 1** (For the Probable Maximum Flood)
Step 11: Determine Peak Flow for Each of the Required Return Periods $$Q_t = \frac{I_T C_T A}{3,6}$$	Q_2=**41.5m³/s** Q_5=**57.3m³/s** Q_{20}=**98.76m³/s** Q_{50}=**135m³/s** Q_{50}=**140.2m³/s** Q_{80}=**147.45m³/s** Q_{100}=**187m³/s**

Appendix 2.2: SDF Method

Table A2.2: SDF Method

Step 1: Identify SDF Drainage Basin	**Basin No 7** (-26.168672°, 28.132696°). See SDF Basin Regions (Gericke & DuPlessis, 2012)
Step 2: Catchment Area	**5.389km^2**
Step 3: Length of Main Channel	**4.09km**
Step 4: Determine the 1085-slope S_{av}=Average slope m/m $H_{0.1L}$= Elevation height at 10% of the length of watercourse (m), $H_{0.85L}$= Elevation height at 85% of the length of the watercourse (m), L= Length of watercourse. $S_{av}=\frac{H_{0.85L}-H_{0.1L}}{(1000)(0.75L)}$	**0.036552m/m**
Step 5: Determine time of concentration $T_c=\left[\frac{0.87L^2}{1000Sav}\right]^{0.385}$ Where: T_c= time of concentration (hours) L= watercourse length (km) S_{av}= average slope	$\left(\frac{0.87(4,09)^2}{1000(0.036552)}\right)^{0.385}$ =**0.7014 hours** Therefore: $T_{c=}$0.7014hrs**/42.09minutes**
Step 6: Determine the point precipitation $P_{t,\ T.}$ If T_c is more than 24 hours use interpolation of the values of the reference rainfall station from TR102, otherwise use Hershfield equation	$P_{t,\ T}$=1.13(0.41+0.64lnT) (0.11+0.27lnt) (0.79$M^{0.69}R^{0.20}$) M= Mean annual daily maxima (49mm for basin 7) R= average number of days per year on which thunder was heard (39 days for basin 7) T= Return Period, 1 year.

	Therefore: $\mathbf{P_{t,\,T}=10.04mm}$ T= Return Period, 2 years. Therefore: $\mathbf{P_{t,\,T}=20.92mm}$ T= Return Period, 5 years. Therefore: $\mathbf{P_{t,\,T}=35.29mm}$ T= Return Period, 20 years. Therefore: $\mathbf{P_{t,\,T}=57.03mm}$ T= Return Period, 50 years. Therefore: $\mathbf{P_{t,\,T}=71.4mm}$ T= Return Period, 64 years. Therefore: $\mathbf{P_{t,\,T}=75.28mm}$ T= Return Period, 80 years. Therefore: $\mathbf{P_{t,\,T}=78.77mm}$ T= Return Period, 100 years. Therefore: $\mathbf{P_{t,\,T}=82.27mm}$ T= Return Period, 160 years. Therefore: $\mathbf{P_{t,\,T}=111.56mm}$ T= Return Period, 197.2 years. Therefore: $\mathbf{P_{t,\,T}=115.65mm}$
Step 7: Determine average rainfall over the catchment (P_t x ARF%) for the required return period T (years). Where ARF%= $(9000\text{-}12800\ln A+9830\ln t)^{0.4}$ Determine the rainfall intensity by dividing the average rainfall with the time of concentration.	ARF=$(90000\text{-}12800\ln A+9830\ln t)^{0.4}$ ARF= =102.05% Area Factors are above 100% thus the existing areas are maintained. Rainfall Intensity: (1-year Return Period) =10.04/0.7040= **14.26mm/h** (2-year Return Period) =20.92/0.7040= **29.72mm/h**

	(20-year Return Period) =35.29/0.7040= **50.13mm/h** (20-year Return Period) =57.03/0.7040= **81.31mm/h** (50-year Return Period) =71,4/0.7040= **101.420mm/h** (64-year Return Period) =71,4/0.7040= **106.92mm/h** (80-year Return Period) =71,4/0.7040= **111.89mm/h** Rainfall Intensity (100-year Return Period) =82.27/0.7040= **116.86mm/h** Rainfall Intensity (160-year Return Period) =111.56/0.7040= **158.46mm/h** Rainfall Intensity (160-year Return Period) =115.65/0.7040= **164.27mm/h**
Step 8: Determine the run-off coefficients C_{100} and C_2 from Table 3B.1 of the SANRAL Drainage Manual. Determine the return period factor Y_T. $C_T=\frac{C2}{100}+\left(\frac{Yt}{2.33}\right)\left(\frac{C100}{100}-\frac{C2}{100}\right)$	C_2=15% C_{100}=60% Y_{20}=1.64, Y_{50}=2.05 and Y_{100}=2.33 A weighted mean is applied to determine Y_{80}=2.218 A weighted mean is applied to determine Y_{64}=2.1284

	Therefore C_1=0,15 C_2=0,15 C_5=0,31 C_{20}=0,467 C_{50}=0,546 C_{64}=0,561 C_{80}=0,578 C_{100}=0,6 C_{160}=0,629 C_{197}=0,6328
Step 9: The flood peak Q_T (m^3/s) for the required return period $Q_T=\frac{C_T I_T A}{3.6}$	$Q_1=\frac{0.15(14.26)(5,389)}{3.6}$=**3.2m³/s** $Q_2=\frac{0.15(29.72)(5,389)}{3.6}$=**6.67m³/s** $Q_5=\frac{0.31(50.13)(5,389)}{3.6}$=**23.26m³/s** $Q_{20}=\frac{0.467(81,31)(5,389)}{3.6}$=**56.84m³/s** $Q_{50}=\frac{0.546(101,4)(5,389)}{3.6}$=**82.87m³/s** $Q_{64}=\frac{0.561(106,92)(5,389)}{3.6}$=**89.79m³/s** $Q_{80}=\frac{0.578(111,89)(5,389)}{3.6}$=**96.81m³/s** $Q_{100}=\frac{0.6(116.86)(5,389)}{3.6}$=**104.95m³/s** $Q_{160}=\frac{0.629(158.46)(5,389)}{3.6}$=**149.2m³/s** $Q_{160}=\frac{0.632(164.27)(5,389)}{3.6}$=**155.4m³/s**

APPENDIX 3: FLOOD WARNINGS

Links to Flood Warnings

- **Flood warning 1**: https://www.iol.co.za/news/south-africa/gauteng/ekurhuleni-warns-of-heavy-rains-that-may-damage-property-cause-death-0309e673-aa40-42d1-815a-57d4ccc70883
- **Flood warning 2**: https://www.ekurhuleni.gov.za/component/content/article/30-press-releases/community-safety/5682-warning-severe-thunderstorm.html?Itemid=101
- **Flood Warning 3**: https://www.timeslive.co.za/news/south-africa/2021-02-26-severe-storm-warning-as-hail-rain-pummel-parts-of-gauteng/

APPENDIX 4: RAW RAINFALL DEPTH DATA PROVIDED BY SAWS

- **21 January 2016- Johannesburg INT WO**

STATION	RAINFALL Depth	Time
JOHANNESBURG INT WO	0,4	21-Jan-2016 14:30
JOHANNESBURG INT WO	0,2	21-Jan-2016 14:35
JOHANNESBURG INT WO	0,0	21-Jan-2016 14:40
JOHANNESBURG INT WO	2,0	21-Jan-2016 14:45
JOHANNESBURG INT WO	2,2	21-Jan-2016 14:50
JOHANNESBURG INT WO	0,6	21-Jan-2016 14:55
JOHANNESBURG INT WO	1,2	21-Jan-2016 15:00
JOHANNESBURG INT WO	1,0	21-Jan-2016 15:05
JOHANNESBURG INT WO	0,6	21-Jan-2016 15:10
JOHANNESBURG INT WO	0,2	21-Jan-2016 15:15
JOHANNESBURG INT WO	0,2	21-Jan-2016 15:20
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:25
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:30
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:35
JOHANNESBURG INT WO	0,2	21-Jan-2016 15:40
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:45
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:50
JOHANNESBURG INT WO	0,0	21-Jan-2016 15:55
JOHANNESBURG INT WO	0,0	21-Jan-2016 16:00

- **21 January 2016- JHB BOT TUINE**

STATION	RAINFALL Depth	Time
JHB BOT TUINE	0,0	21-Jan-2016 16:50
JHB BOT TUINE	0,0	21-Jan-2016 16:55
JHB BOT TUINE	0,2	21-Jan-2016 17:00
JHB BOT TUINE	0,2	21-Jan-2016 17:05
JHB BOT TUINE	0,0	21-Jan-2016 17:10
JHB BOT TUINE	0,0	21-Jan-2016 17:15
JHB BOT TUINE	0,6	21-Jan-2016 17:20
JHB BOT TUINE	0,6	21-Jan-2016 17:25
JHB BOT TUINE	0,8	21-Jan-2016 17:30
JHB BOT TUINE	0,2	21-Jan-2016 17:35

STATION	RAINFALL Depth	Time
JHB BOT TUINE	0,0	21-Jan-2016 17:40
JHB BOT TUINE	0,2	21-Jan-2016 17:45
JHB BOT TUINE	0,2	21-Jan-2016 17:50
JHB BOT TUINE	0,0	21-Jan-2016 17:55
JHB BOT TUINE	0,0	21-Jan-2016 18:00
JHB BOT TUINE	0,0	21-Jan-2016 18:05
JHB BOT TUINE	0,0	21-Jan-2016 18:10
JHB BOT TUINE	0,0	21-Jan-2016 18:15
JHB BOT TUINE	0,0	21-Jan-2016 18:20
JHB BOT TUINE	0,0	21-Jan-2016 18:25
JHB BOT TUINE	0,0	21-Jan-2016 18:30
JHB BOT TUINE	0,0	21-Jan-2016 18:35
JHB BOT TUINE	0,2	21-Jan-2016 18:40
JHB BOT TUINE	0,6	21-Jan-2016 18:45
JHB BOT TUINE	2,2	21-Jan-2016 18:50
JHB BOT TUINE	0,2	21-Jan-2016 18:55
JHB BOT TUINE	0,2	21-Jan-2016 19:00
JHB BOT TUINE	0,0	21-Jan-2016 19:05

- **09 November 2016- JHB BOT TUINE**

STATION	RAINFALL Depth	Time
JHB BOT TUINE	0,4	09-Nov-2016 16:50
JHB BOT TUINE	2,2	09-Nov-2016 16:55
JHB BOT TUINE	0,2	09-Nov-2016 17:00
JHB BOT TUINE	0,0	09-Nov-2016 17:05
JHB BOT TUINE	0,2	09-Nov-2016 17:10
JHB BOT TUINE	0,0	09-Nov-2016 17:15
JHB BOT TUINE	0,0	09-Nov-2016 17:20
JHB BOT TUINE	1,2	09-Nov-2016 17:25
JHB BOT TUINE	1,0	09-Nov-2016 17:30
JHB BOT TUINE	0,0	09-Nov-2016 17:35
JHB BOT TUINE	0,0	09-Nov-2016 17:40
JHB BOT TUINE	0,6	09-Nov-2016 17:45
JHB BOT TUINE	0,4	09-Nov-2016 17:50
JHB BOT TUINE	0,0	09-Nov-2016 17:55
JHB BOT TUINE	0,0	09-Nov-2016 18:00

JHB BOT TUINE	0,0	09-Nov-2016 18:05
JHB BOT TUINE	0,0	09-Nov-2016 18:10
JHB BOT TUINE	0,0	09-Nov-2016 18:15
JHB BOT TUINE	0,0	09-Nov-2016 18:20
JHB BOT TUINE	0,0	09-Nov-2016 18:25
JHB BOT TUINE	0,0	09-Nov-2016 18:30
JHB BOT TUINE	0,4	09-Nov-2016 18:35
JHB BOT TUINE	0,2	09-Nov-2016 18:40

- **09 November 2016- Johannesburg INT WO**

STATION	RAINFALL Depth	Time
JOHANNESBURG INT WO	0,2	09-Nov-2016 16:55
JOHANNESBURG INT WO	0,0	09-Nov-2016 17:00
JOHANNESBURG INT WO	0,0	09-Nov-2016 17:05
JOHANNESBURG INT WO	0,2	09-Nov-2016 17:10
JOHANNESBURG INT WO	4,0	09-Nov-2016 17:15
JOHANNESBURG INT WO	7,8	09-Nov-2016 17:20
JOHANNESBURG INT WO	10,8	09-Nov-2016 17:25
JOHANNESBURG INT WO	9,4	09-Nov-2016 17:30
JOHANNESBURG INT WO	17,2	09-Nov-2016 17:35
JOHANNESBURG INT WO	8,2	09-Nov-2016 17:40
JOHANNESBURG INT WO	3,2	09-Nov-2016 17:45
JOHANNESBURG INT WO	5,6	09-Nov-2016 17:50
JOHANNESBURG INT WO	9,0	09-Nov-2016 17:55
JOHANNESBURG INT WO	4,0	09-Nov-2016 18:00
JOHANNESBURG INT WO	2,6	09-Nov-2016 18:05
JOHANNESBURG INT WO	1,6	09-Nov-2016 18:10
JOHANNESBURG INT WO	0,8	09-Nov-2016 18:15
JOHANNESBURG INT WO	0,4	09-Nov-2016 18:20
JOHANNESBURG INT WO	0,2	09-Nov-2016 18:25
JOHANNESBURG INT WO	0,2	09-Nov-2016 18:30
JOHANNESBURG INT WO	0,0	09-Nov-2016 18:35
JOHANNESBURG INT WO	0,2	09-Nov-2016 18:40
JOHANNESBURG INT WO	0,4	09-Nov-2016 18:45
JOHANNESBURG INT WO	0,4	09-Nov-2016 18:50
JOHANNESBURG INT WO	0,2	09-Nov-2016 18:55

- **02 March 2017- Zurbano AWS**

STATION	RAINFALL Depth	Time
ZUURBEKOM AWS	0,2	02-Mar-2017 15:35
ZUURBEKOM AWS	0,2	02-Mar-2017 15:40
ZUURBEKOM AWS	0,0	02-Mar-2017 15:45
ZUURBEKOM AWS	0,0	02-Mar-2017 15:50
ZUURBEKOM AWS	2,4	02-Mar-2017 15:55
ZUURBEKOM AWS	1,8	02-Mar-2017 16:00
ZUURBEKOM AWS	0,8	02-Mar-2017 16:05
ZUURBEKOM AWS	0,4	02-Mar-2017 16:10
ZUURBEKOM AWS	0,0	02-Mar-2017 16:15
ZUURBEKOM AWS	0,2	02-Mar-2017 16:20
ZUURBEKOM AWS	0,4	02-Mar-2017 16:25
ZUURBEKOM AWS	1,0	02-Mar-2017 16:30
ZUURBEKOM AWS	0,6	02-Mar-2017 16:35
ZUURBEKOM AWS	0,8	02-Mar-2017 16:40
ZUURBEKOM AWS	0,0	02-Mar-2017 16:45
ZUURBEKOM AWS	0,0	02-Mar-2017 16:50
ZUURBEKOM AWS	0,2	02-Mar-2017 16:55
ZUURBEKOM AWS	0,0	02-Mar-2017 17:00
ZUURBEKOM AWS	0,0	02-Mar-2017 17:05
ZUURBEKOM AWS	0,0	02-Mar-2017 17:10
ZUURBEKOM AWS	0,0	02-Mar-2017 17:15
ZUURBEKOM AWS	0,0	02-Mar-2017 17:20

- **02 March 2017- Johannesburg INT WO**

STATION	RAINFALL Depth	Time
JOHANNESBURG INT WO	0,2	02-Mar-2017 15:55
JOHANNESBURG INT WO	0,2	02-Mar-2017 16:00
JOHANNESBURG INT WO	0,2	02-Mar-2017 16:05
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:10
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:15
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:20
JOHANNESBURG INT WO	0,2	02-Mar-2017 16:25
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:30
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:35
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:40
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:45
JOHANNESBURG INT WO	0,2	02-Mar-2017 16:50
JOHANNESBURG INT WO	0,0	02-Mar-2017 16:55

STATION	RAINFALL Depth	Time
JOHANNESBURG INT WO	0,2	02-Mar-2017 17:00
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:05
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:10
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:15
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:20
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:25
JOHANNESBURG INT WO	0,2	02-Mar-2017 17:30
JOHANNESBURG INT WO	0,0	02-Mar-2017 17:35

- **02 March 2017- JHB BOT TUINE**

STATION	RAINFALL Depth	Time
JHB BOT TUINE	0,0	02-Mar-2017 15:35
JHB BOT TUINE	0,8	02-Mar-2017 15:40
JHB BOT TUINE	5,6	02-Mar-2017 15:45
JHB BOT TUINE	6,4	02-Mar-2017 15:50
JHB BOT TUINE	3,6	02-Mar-2017 15:55
JHB BOT TUINE	1,4	02-Mar-2017 16:00
JHB BOT TUINE	0,4	02-Mar-2017 16:05
JHB BOT TUINE	0,4	02-Mar-2017 16:10
JHB BOT TUINE	1,0	02-Mar-2017 16:15
JHB BOT TUINE	0,4	02-Mar-2017 16:20
JHB BOT TUINE	0,2	02-Mar-2017 16:25
JHB BOT TUINE	0,2	02-Mar-2017 16:30
JHB BOT TUINE	0,2	02-Mar-2017 16:35
JHB BOT TUINE	0,0	02-Mar-2017 16:40
JHB BOT TUINE	0,0	02-Mar-2017 16:45
JHB BOT TUINE	0,0	02-Mar-2017 16:50
JHB BOT TUINE	0,0	02-Mar-2017 16:55
JHB BOT TUINE	0,0	02-Mar-2017 17:00
JHB BOT TUINE	0,0	02-Mar-2017 17:05
JHB BOT TUINE	0,2	02-Mar-2017 17:10
JHB BOT TUINE	0,0	02-Mar-2017 17:15
JHB BOT TUINE	0,2	02-Mar-2017 17:20
JHB BOT TUINE	0,4	02-Mar-2017 17:25
JHB BOT TUINE	0,0	02-Mar-2017 17:30
JHB BOT TUINE	0,0	02-Mar-2017 17:35
JHB BOT TUINE	0,0	02-Mar-2017 17:40