Project Ladysmith

CIE4300
Multidisciplinary Project

Delft University of Technology

TU Delft

Project Ladysmith

CIE4300

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Preface

During the Civil Engineering master's program of the Delft University of Technology, students are given the opportunity to do a 10 ECTS multidisciplinary project. During this project, the knowledge of different disciplines is combined in an actual case. This report is the result of such a multidisciplinary project. The knowledge of six students, from the Hydraulic Engineering and Watermanagement disciplines, was combined to find the causes of the flood problem of Ladysmith, a town in South Africa.

The topic of this study is based on a project proposal by Royal HaskoningDHV. This proposal is written on the basis of their involvement in the Ladysmith Flood Protection Scheme. After the implementation of this flood scheme, Ladysmith has still suffered from flood events. The goal of the proposed project is to analyse the Ladysmith Flood Protection Scheme and to investigate the shortcomings in it. This report has been written for Royal HaskoningDHV. The floods in Ladysmith, since the implementation of the Ladysmith Flood Projection Scheme, will be analysed. Based on this analysis, a number of recommendations to Royal HaskoningDHV will be given about decreasing the probability of floods in Ladysmith.

For the reader, who has not been involved in the flood problem of Ladysmith before, the report starts with a description of the area and recent flood events. This is followed by an analysis of the possible causes of the floods. The analysis elaborates on the most probable causes, which are further analysed with two numerical models. The report ends with a conclusion and recommendations.

The project case, our stay in South-Africa and the pleasant working facilities at the Umhlanga office we owe to Dr. Ir. M. van Ledden and Ir. S. Zweers from Royal HaskoningDHV. We are very grateful for their help in finding a suitable project and the needed facilities for completing the project. Also during the project, they provided us with useful feedback. The same holds for our supervisors from the TU Delft: Prof. Dr. Ir. L.C. Rietveld, Dr. Ir. A. Blom and Dr. Ir. O.A.C. Hoes. Several times, they kept us sharp with numerous critical questions. Those questions helped us to keep an eye on the final goal of the project. Besides that, they helped us to solve the difficulties in the project.

We also like to thank our colleagues from the Royal HaskoningDHV Umhlanga office. They made us feel very welcome and at home in the office for the duration of our project. In addition, everybody was happy to contribute to the project. A special thanks to Dr. M. Nsibirwa, who accompanied us on the first field visit in and around Ladysmith and for his help with a lot of useful data.

Last but not least, we would very much like to thank our partners: Royal HaskoningDHV, TU Delft International Internship Fund, TU Delft | Global Initiative and the Hydraulic Engineering Department of the TU Delft, for their financial support. This support made it possible for us to go to South Africa and accommodate our stay.

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Executive summary

The town of Ladysmith is located in the uThukela Disctrict in the KwaZulu-Natal province, South Africa. In the past Ladysmith was subject to severe floods because of the Klip River. This river flows through the town and overtopped its banks regularly during the wet season. In order to prevent these floods from occurring, the Municipality of Ladysmith designed the Ladysmith Flood Protection Scheme (LFPS). The goal of this scheme is to protect the town of Ladysmith against a flood event with a return period of 100 years.

The most important aspect of the LFPS was the construction of the Qedusizi Dam, 5.5 kilometer upstream of Ladysmith. This dam was finished in 1998. Furthermore, levees were constructed around the Central Business District (CBD) of Ladysmith and new stormwater valves were installed on the outlets of the drainage system, that discharges into the Klip River. The stormwater valves were installed to make sure that no backflow would occur from the river into the drainage system, during high water level in the Klip River.

Despite the implemented measures, multiple floods occurred since the construction of the Qedusizi Dam. Although the severity of these floods has strongly reduced since the implementation of the LFPS, they do still cause a lot of nuisance and disrupt daily life in Ladysmith, mainly in the CBD. Therefore, the goal of this study is to investigate the causes of the recent flood events in the CBD and to assess its flood resilience to extreme events that might occur in the future.

This study puts its focus on the CBD of Ladysmith, as the town's business activity is centered in this area and the national route (N11) crosses the CBD. Furthermore, only the flood events after the construction of the dam have been investigated.

First, a broad analysis of the flood events and their possibles causes was carried out. From this analysis it was concluded that the recent flood events in the CBD had some common characteristics. They all occurred in the lower laying areas of the CBD and were accompanied by extreme daily rainfall in Ladysmith. Also, the water levels in the Klip River were high for an extended amount of time. However, the water levels and the discharge through the river were not extreme, the Klip River did not burst its banks.

Regarding the possible causes of the recent floods, it was concluded that they were not caused by overtopping of the river banks, but by a poorly functioning drainage system. It was concluded that the drainage system of the CBD is in a bad state and contains a lot of garbage and rubble. In addition, the stormwater valves prevent the drainage system from outflow in case of high water levels in the river. In the case of a higher pressure in the Klip River compared to the pressure in the drainage system the stormwater valves will close to prevent back flow.

In order to get more quantitative insight on the causes of the recent flood events, a HEC-RAS and SWMM model were applied. The HEC-RAS model is used to analyse the river system and the SWMM model is used to analyse the drainage system of the city. One of the output factors of HEC-RAS is the water level of the Klip River. This output factor is used as an input factor for SWMM. As there was no calibration data available, the models were used for a sensitivity analysis. The model results indicated that during the recent flood events, the garbage in the drainage system had a larger impact than the closure of the stormwater valves. However, the impact of the closure of the stormwater valves gets larger if the hourly rainfall gets more intense. Furthermore, by testing the system for different percentages of garbage, it can be concluded that the first 25% of garbage that is put into the system gives the strongest reduction in the capacity of the drainage system. Regarding the stormwater valves, it can be concluded that the drained area per stormwater valve differs a lot. The valves that drain the largest areas are also the ones that close first when the water level in the Klip River rises. This is caused by geographical characteristics of those areas, which have a smaller elevation compared to the other pipe inlets.

About the design of the drainage system no final conclusion could be drawn since the model was not calibrated. However, the model also showed floods in the system when a clean system with open stormwater valves was modelled.

Apart from the past flood events, potential future extreme events were tested with the models. This was done by combining the rainfall and discharge data related to different return periods, of up to a 1 in 100 year return period. For these extreme events, it can be concluded that the closure of the stormwater valves has a larger impact than the garbage in the system.

The HEC-RAS model that was used to model the water levels in the river, showed that for the 1 in 50 and 1 in 100 years events, the levees overtop at multiple locations along the river. Since the HEC-RAS model could not be calibrated as well, no final conclusions can be drawn from the results. The discharge that flows through Ladysmith with a return period of 100 years seems to be higher than was assumed for the design of the Qedusizi Dam. For the design of the Qedusizi Dam it was assumed that the 1 in 100 discharge through Ladysmith would be $450m^3/s$. Analysis of the data from two measurement stations upstream of Ladysmith indicated a discharge of almost $650m^3/s$ for the 1 in 100 year event. The analysis showed that this difference is due to an underestimation of the discharge of a tributary that joins the Klip River between the Qedusizi Dam and the CBD. The used Q-h relation to determine the the discharge of the Klip River is determined a long time ago. In a new research about this area, the Q-h relation has to be checked.

Contents

Acronyms

AMSL Above Mean Sea Level. 3

CBD Central Business District. 1, 2, 5, 7–9, 11–18, 22–24, 27–30, 32, 33, 39, 41–45, 55, 57–59, 64–66, 68, 76, 85, 88, 96

DWS Department of Water and Sanitation. 3, 48, 69

DWTP Drinking Water Treatment Plant. 5

EPA United States Environmental Protection Agency. 33

GIS Geographical Information System. 72, 73

GLS Global Land Survey. 72

HEC-RAS Hydrologic Engineering's Center's River Analysis System. 23–25, 27–29, 36, 39, 45, 92

KZN KwaZulu-Natal. 1, 2, 5, 8, 57, 74

LFPS Ladysmith Flood Protection Scheme. 4, 5, 7, 13, 14, 44, 46, 76

NASA National Aeronautics and Space administration. 72

NDVI Normalized Difference Vegetation Index. 72, 73

NIR Near Infrared. 73

SAWS South African Weather Service. 50

SSI Steward Scott Incorportated. 17, 52

SWMM Storm Water Management Model. 23, 24, 30–34, 36, 37, 39, 95–97

USGS United States Geological Survey. 72

VIS Visible Imaging System. 73

WWTP Waste Water Treatment Plant. 5

1
Introduction

"The situation is really bad," said Ladysmith deputy mayor Fana Madlala.
"We are trying to bring the situation to normality. There are two main roads in the city. One of the roads is close to the Klip River and is full of water," he said (4-1-2011). [64]

The town of Ladysmith, which is located in KwaZulu-Natal (KZN) South Africa, used to experience severe floods because the Klip River, that flows through the city, bursted its banks regularly during the wet season. After a number of severe floods in the nineties, a flood protection scheme was implemented in order to protect the town from a 1 : 100 year flood. This Ladysmith Flood Protection Scheme (LFPS) included the Qedusizi flood control dam, the levees and the flood control valves. Since the construction of the Qedusizi Dam (which is Zulu for 'End of Suffering') in 1997, the amount of floods and their severity have reduced significantly. However, despite the implemented flood protection scheme the Ladysmith's Central Business District (CBD) and other parts of the town are still coping with floods that cause inconveniences.

In January 2011 a well reported flood event occurred that caused problems to transport links and reduced business activity within the Ladysmith CBD. In September 2012 part of the CBD was flooded again. Currently, it is not known what exactly causes the floods.

The purpose of this study is therefore to *find the causes of the floods in the Central Business District of Ladysmith, that occurred after the implementation of the Ladysmith Flood Protection Scheme and to asses the flood resilience of the Central Business District to extreme events that might occur in the future.*

The focus of this study is on the CBD of Ladysmith, since the recent floods occurred often in this area and the CBD has the greatest economical value. Because the implementation of the LFPS changed the system significantly, only the floods after the construction are reviewed.

An analysis of the recent flood events and their possible causes was carried out first. In order to find the causes, all factors that can contribute to the occurrence of the floods were analysed. Next the probable causes identified from the analysis were tested quantitatively by models. Besides the past flood events, also more extreme scenarios were analysed in order to assess the flood resilience of the CBD.

This report starts with a description of the most important characteristics of the project area. Chapter two begins with a description of the entire catchment and subsequently zooms in on different aspects. In the third chapter, the floods events and their consequences are discussed. In the fourth chapter, the results of the detailed analysis of the river system, drainage system and past flood events are given. At the end of this chapter a conclusion on the possible causes is given. Whether these are the real causes is tested with two numerical models, HEC-RAS and SWMM. These models were not only used to test the causes of the past flood events, but also to assess the flood resilience of the CBD. The results of these models can be found in chapter 5. The reports ends with a conclusion on the findings of this research and recommendations based on these findings.

2

Area description of the Klip River catchment

As mentioned in Chapter 1, the purpose of the study is to find the causes of the floods of Ladysmith's CBD. The CBD is located along the banks of the Klip River. In this chapter, the characteristics of the Klip River and its catchment will be described. This will be followed up by a description of the actions taken by the municipality of Ladysmith to protect the area of severe floods and the water infrastructure in Ladysmith. The chapter ends with the description of the Ladysmith CBD, defined as the *focus area*.

2.1. Topography of Klip River catchment

The town of Ladysmith is located on the banks of the Klip River (*Stone River*) in the Alfred Duma Municipality. Together with the Okhahlamba Municipality and the Inkosi Langalibalele Municipality it forms the uThukela District, which is one of the eleven districts of the South African province KZN. With a population of 64855, counted in 2011, Ladysmith is the largest town of the uThukela District. See Figure 2.1 for the location of KZN in South Africa and the location of the uThukela District and Ladysmith in KZN.

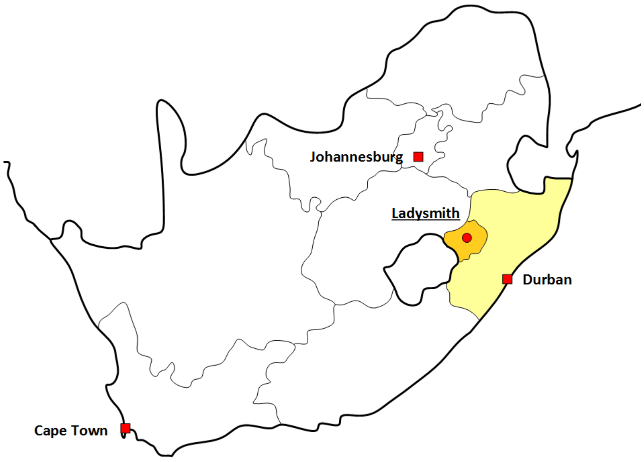


Figure 2.1: Location of KZN province, uThukela District and Ladysmith in South Africa

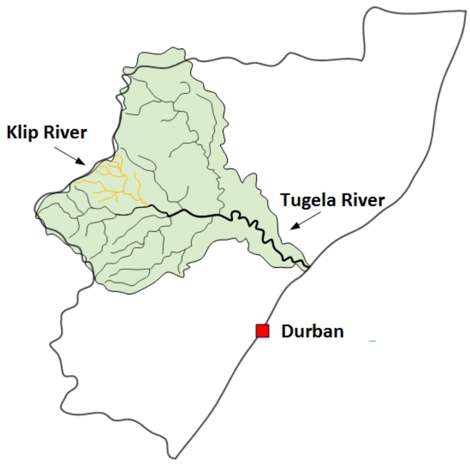


Figure 2.2: Tugela catchment within KZN and the location of the Klip River

The uThukela District is named after the main stem of the Klip River, the Tugela River. The Tugela River is the largest river of KZN and has its mouth at the Indian Ocean. Figure 2.2 shows the Tugela catchment and the location of the Klip River within. Like most tributaries of the Tugela River, the Klip River originates in the Drakensbergen, which border the uThukela District and KZN from Lesotho and the Free State province.

Being a very intermittent river, the Klip River barely has any flow for the majority of the year. In the summer period, from October to March, rainfall becomes a lot more frequent in the Drakensbergen and water levels rise tremendously. This results in an average discharge of the Klip River at Ladysmith of 11.4 m^3/s in the summer period, while in the winter period the average discharge is 1.7 m^3/s. It should be noted that these values are based on data measured in the Klip River after the construction of the Qedusizi Dam. Over its course the Klip River is joined by several tributaries. The main tributary, the Sand River joins 8 kilometer upstream of Ladysmith. The catchment of the Klip River has a surface of 2157 square kilometer and can be seen in Figure 2.3. Along with the stream order classification of the Klip River and its tributaries, it shows the location of the Windsor Dam, the Qedusizi Dam and Ladysmith along the Klip River. The dams will be discussed in Section 2.2. The stream order is based on the data of the South African Department of Water and Sanitation (DWS). These data are based on the Strahler Method [33].

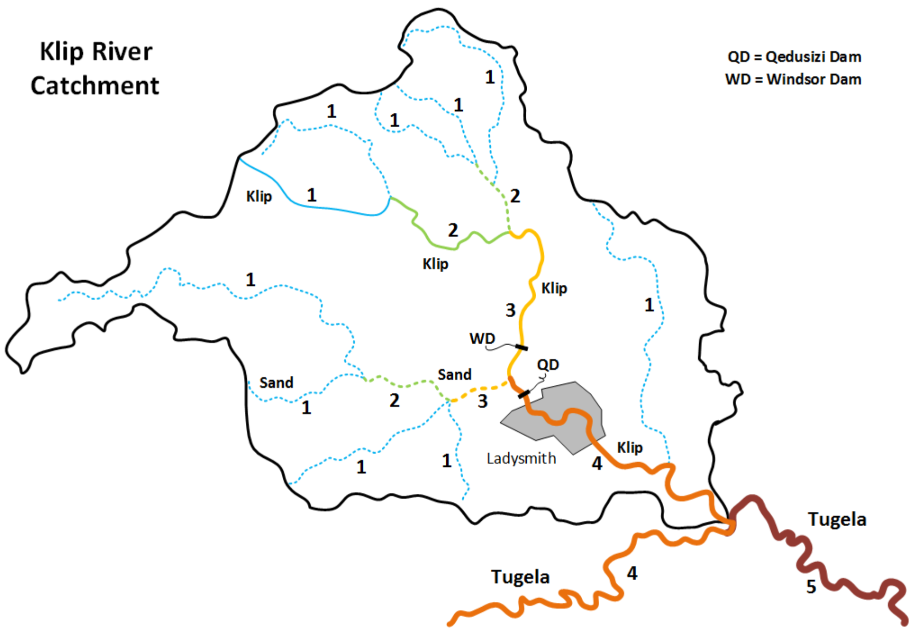


Figure 2.3: Klip River catchment

From its origin until its confluence with the Tugela River, the Klip River spans a total channel length of 140 kilometer. The straight line distance, or valley length, is 64 kilometer. The ratio of the valley length over the channel length, gives a Sinuosity Index of 2.19, which indicates a strongly meandering river. Satellite images and field observations confirmed this conclusion. Running from roughly 1800 meter Above Mean Sea Level (AMSL) in the Drakensbergen to just under 800 meter AMSL at the confluence with the Tugela River, an elevation difference of approximately 1000 meter is covered. This indicates an average bed slope of 0.71% for the Klip River. The longitudinal profile of the Klip River can be seen in Figure 2.4. To acquire this profile, first a so-called *path* was made of the Klip River in Google Earth™. The levels of elevation as given by Google Earth™ along this path were used to create the longitudinal profile in Excel®. More than half of the total elevation difference is covered in the Drakensbergen and another 200 meter just before the junction with the Tugela River. The majority of the Klip River has a milder slope of 0.24%. Zooming in on the Klip River's longitudinal and latitudinal profiles reveals larger variations between the river characteristics locally. In Section 2.4, a more detailed description is given of the Klip River's characteristics around the focus area.

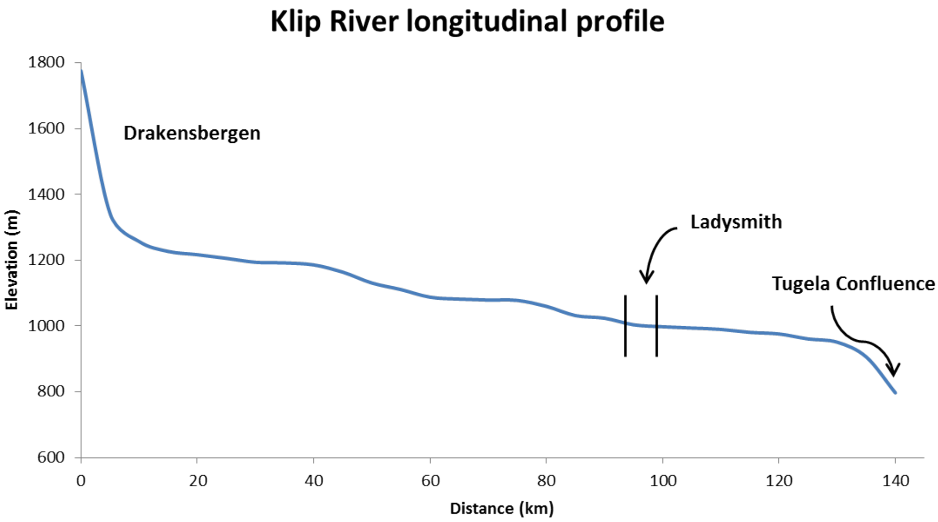


Figure 2.4: Klip River longitudinal profile

2.2. Ladysmith Flood Protection Scheme

In response to the flood history of Ladysmith the Ladysmith Flood Protection Scheme (LFPS) was initiated in 1998. The scheme consists of levees and stormwater valves along the Klip River and the Qedusizi dam, see Figure 2.5a. The first interference within the catchment of the Klip River was already done in 1949, with the construction of the Windsor Dam. However, due to severe sedimentation of the reservoir (reduction of approximately 50% within 20 years) the dam was not able to protect Ladysmith from high waters.

(a) The dam as seen from downstream

(b) Two uncontrolled bottom outlets

Figure 2.5: Qedusizi Dam

The Qedusizi Dam, with its flood attenuation function, is the main feature of the LFPS. The dam is located approximately 5.5 km upstream of Ladysmith and features two uncontrolled bottom outlets, see Figure 2.5b. The LFPS is designed such that it should protect Ladysmith up to a 1 in 100 year flood event. Included in the scheme is the maintenance, monitoring and operation of the different components along the Klip River and the river channel itself. Additionally a flood warning system is implemented, to alert residents for possible high waters. The stormwater valves are installed to prevent backflow of river water into the drainage system. At many points the invert levels of the outfalls, with the stormwater valves installed on them, are well below river flood levels with relatively small return periods. Also a significant part of the drained areas lies below the adjacent river bank or crest level.

2.3. Ladysmith's water services infrastructure

The town of Ladysmith has a Drinking Water Treatment Plant (DWTP), a Waste Water Treatment Plant (WWTP) and a separate sewerage and stormwater drainage system, see Figure 2.6). The DWTP uses water from the Spioenkop Dam and the Klip River as a source for the drinking water for the town. The present peak capacity is 36 Ml/day. Domestic sewage and industrial wastewater is treated at the WWTP of Ladysmith, which has a design capacity of 21 Ml/day. After treatment the water is discharged into the Klip River downstream of Ladysmith. Both treatment works are not always able to meet the legislation standards and are in need of maintenance.

The stormwater drainage system consists mostly of pipes and box culverts. As mentioned above, the drainage system discharges into the Klip River via stormwater valves [58]. Information about the current state of the sewerage system and the stormwater drainage system is not readily available. However, news reports and field observations suggest that both systems are in a bad state due to a lack of maintenance. [41] [44] [55] [49].

2.4. Ladysmith Central Business District

This paragraph elaborates on the section of the city that is believed to be most relevant to the flood problem of Ladysmith. The larger part of Ladysmith's CBD, as highlighted in Figure 2.6, is defined as the focus area of this study. With the definition of this system it is aimed to set a clear framework for the remainder of this study. This area has a smaller elevation compared to the environment and is so likely to flood.

Ladysmith's main business activities are located in the CBD. The economic damage or inconvenience caused by the floods is highest in this part of the city. In addition, the LFPS with its stormwater valves and levees is particularly designed for this area. The floods in this area are further elaborated in Chapter 3. The remaining urban areas adjacent to the Klip River are disregarded in this study, since the economic damage and nuisance caused by the floods is less in these areas.The northern boundary of the focus area is set along a natural ridge and a railway track, see Figure 3.1. The CBD of Ladysmith is crossed by an important traffic artery, the N11. This is a strategic link for freight transport between KZN and the northern Mpumalanga province. Also it is a key link to the N3 between Johannesburg and Durban. The N11 sees a high amount of traffic and these numbers are expected to increase. [51]

Ladysmith CBD lies within the natural floodplains of the Klip River, enclosed by a distinctive meander along a 4 kilometer reach. A knickpoint in the slope of the river, see Figure 2.7, is located in the crooked river bend, at the western border of the CBD. From the Qedusizi Dam, located 5.5 kilometer upstream of the CBD, until this knickpoint, the Klip River is relatively steep, has a narrow cross-section and shows a sequence of riffles and pools. After the knickpoint, when the river starts to flatten out, it consequently changes its physical characteristics. The riffles disappear and the river's floodplains widen. The somewhat abrupt change in slope implies a water level rise just in front of the CBD, illustrating why the area has been so sensitive to floods during high water in the river. Downstream of the railway bridge and CBD more room is available for the river. The floodplains have a more natural character in the shape of wetlands.

Along the Klip River, the water depth is measured by three measurement stations, as shown in Figure 2.6. The rainfall is measured by a station located at Ladysmith Aerodrome, south of the city. More information about the measurement stations and the validity of their data can be found in Appendix A.

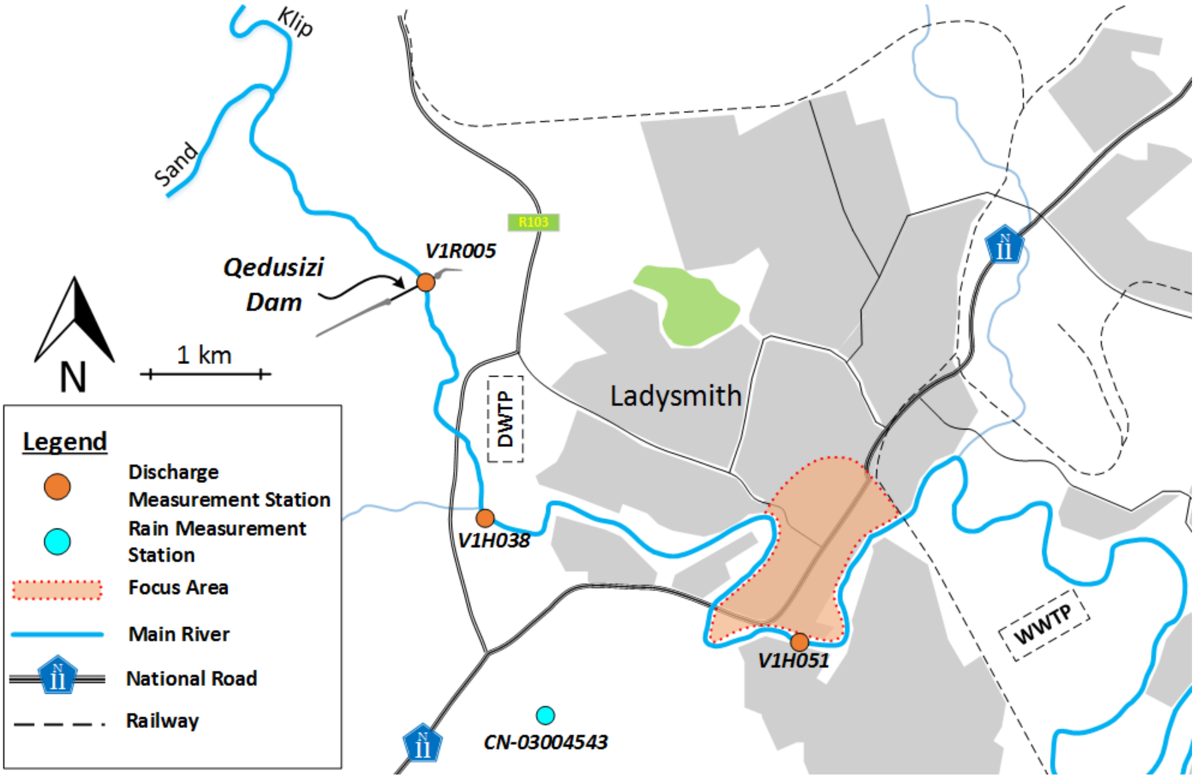


Figure 2.6: Ladysmith overview, showing the focus area, measurement stations and watertreatment works

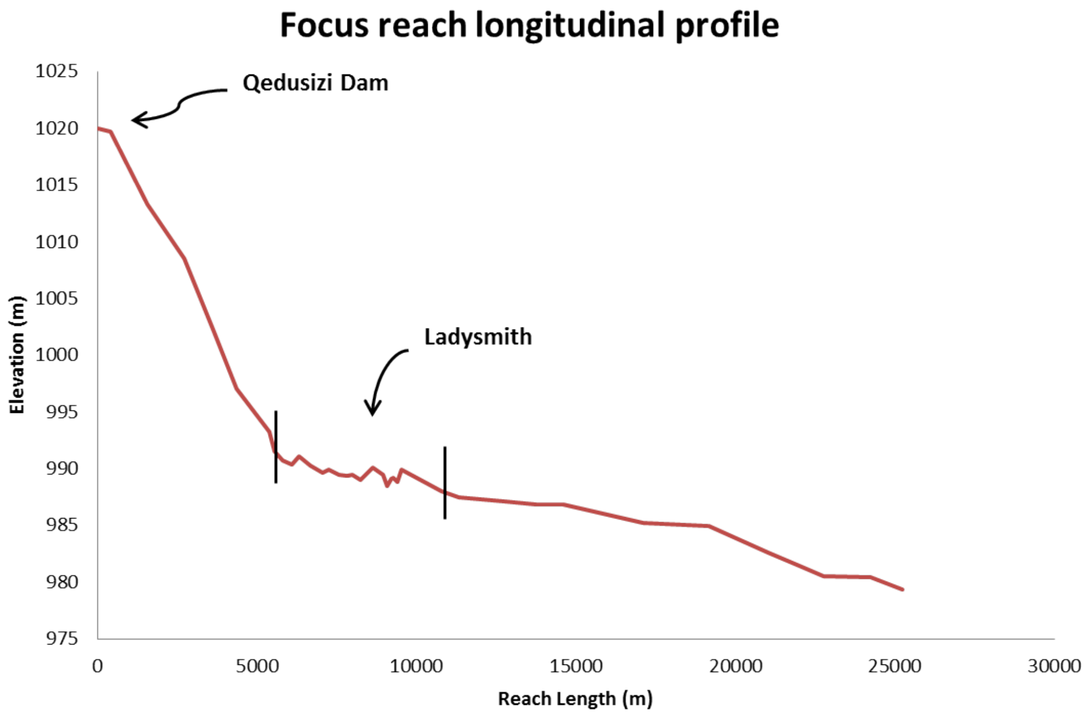


Figure 2.7: Longitudinal profile of the focus reach

3

Scale of the flood problem

The floods that occurred after the construction of the Qedusizi dam still cause problems to areas in and around Ladysmith. This chapter starts with listing the flood events after completion of the Qedusizi Dam that causes significant inconvenience within the CBD. Furthermore, it describes the consequences of these flood and informs the reader about the different opinions of Ladysmith's inhabitants.

3.1. Flood events after building the Qedusizi Dam

Despite the construction of the Qedusizi Dam in 1998 as part of the LFPS, a number of floods have occurred in Ladysmith. In Appendix C different sources are mentioned and an overview is given of all flood related events. As discussed in Section 2.4 only the flood events that affect the CBD, the focus area, are of interest for this study. These events are listed in Table 3.1. Unfortunately, the durations of the flood events could not be determined from news reports. The table therefore states time ranges in which the flood events took place.

Flood event	Date
1	3-5 January 2011
2	13-15 January 2012
3	6-8 September 2012
4	3-4 December 2015

Table 3.1: Floods events in focus area

The observed flood events differ in duration from a few hours to a couple of days and are often accompanied by rainfall and a high water level in the Klip River. A location that is frequently flooded according to news reports is the Lyell Street in the CBD, see Figure 3.1. In some reports specifically the crossings of the Princess Street, King Street, Queen Street and Alexander Street with the Lyell Street are mentioned. The Forbes Street is also reported, but less frequently. So far the floods after commissioning of the dam have been restricted to approximately half a meter water depth on the streets.

Figure 3.1: Central Business District of Ladysmith

3.2. Consequences of recent flood events

The floods that still occur in the CBD of Ladysmith affect the daily life of people in various ways. This section informs the reader about the different consequences of the flood event that occured after the construction of the Qedusizi Dam.

3.2.1. A threat to human life

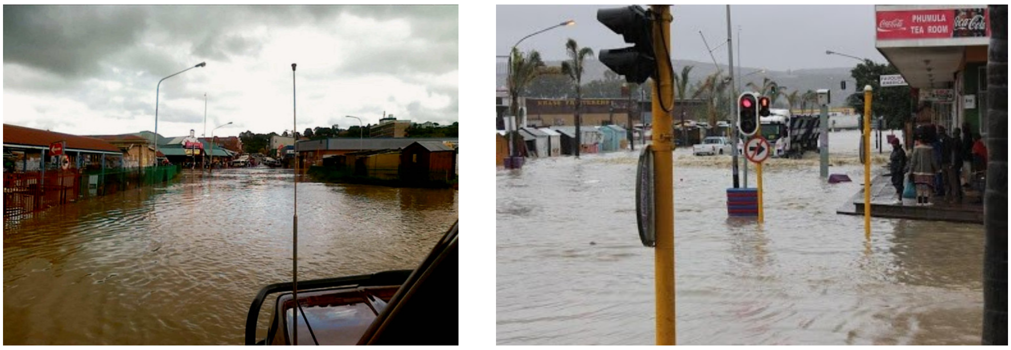
Although the severity of the floods has been significantly reduced by the presence of the Qedusizi Dam, the recent floods are still of potential risk to the inhabitants of Ladysmith. During the flood of January 2011 two persons in Ladysmith were killed when they tried to cross a stream [61]. Interviews with local shop owners have revealed that inhabitants are not always warned, and that the floods sometimes come by surprise. More information on these interviews can be found in Appendix D.

3.2.2. Disruption of infrastructure

The floods cause several systems, both on local as well as on regional level, to be disrupted for a period varying from hours to days.

On regional level the Lyell Street is part of the N11 National Route linking the KZN and Mpumalanga Province. This route can thus be seen as a major transport link for traffic in northern and southern direction. In case of a flood, high water levels in the CBD will stall the traffic on the N11 and immediately delay all transport activities between the provinces. An important junction is located 20 kilometer west of Ladysmith where the N11 crosses with the N3 National Route, which is the main road between Johannesburg and Durban. Additional trouble is caused to road users changing direction here as they will be affected by the traffic congestion on the N11 [3]. The disruption of infrastructure could be seen in Figure 3.2a and Figure 3.2b.

As mentioned before, multiple roads in the Ladysmith CBD can flood at the same time making it hard for residents to move or emergency services to operate. Furthermore, several low-lying bridges are known to be inaccessible as a consequence of the high water level in the river [46] [47].



(a) Flood on the junction of the Lyell Street and Queen Street in January 2011

(b) Flood on the junction of Lyell Street and Alexander Street in September 2012

Figure 3.2: Disruption of infrastructure

3.2.3. Loss of economic valuables

Floods that cause a stand-still of the CBD of Ladysmith will result in economic losses, since the area contains a significant part of the district's economic activity [3]. Shops in the CBD are no longer accessible and stocks are ruined if they are not moved to higher, safer grounds [62]. Additionally, damage to houses, cars, goods and infrastructure has to be compensated for.

3.3. Opinion of Ladysmith's inhabitants

There are different opinions on the floods that still occur in Ladysmith. Since people in South Africa often experience droughts, a common opinion is to be thankful for all the rainfall, even though it causes inconveniences. Others clearly speak out that the floods disrupt daily life and that a solution should be found. Additional information can be found in Appendix D. An opinion shared by the public is that the floods have become less severe since the construction of the Qedusizi Dam.

4

Analysis of flood events and their possible causes

In this chapter the possible causes of the flood problem in Ladysmith will be elaborated. While in previous chapters different subjects were introduced, here they are analysed in more detail. The analysis is done on basis of data, field observations, reports of Royal HaskoningDHV and news reports. In the first two sections the river system and drainage system are analysed. The section river system is structured in similar fashion as the Chapter 2, first analysing the large system and subsequently zooming in. Both sections discuss the possible causes briefly, for extensive calculation the reader is referred to Appendix F. In the third section of this chapter the flood events, as identified from news reports, are analysed in more detail. It is researched whether certain characteristics of the floods events can be explained and if the accompanied properties were unique. Finally, a conclusion on the possible causes will be given.

4.1. Causes related to the river system

In this section the possible causes within the river system are analysed by starting with the entire catchment and subsequently zooming in.

4.1.1. Causes related to the catchment area

In this subsection the precipitation, evaporation and land use within the catchment and Ladysmith are discussed.

Precipitation

The average annual rainfall within the catchment varies between 800-1000 mm and is approximately 700 mm in Ladysmith. These numbers are based on an analysis of the available rainfall data in the catchment and in Ladysmith. More details about this analysis can be found in Section F.1 in Appendix F. As mentioned before rain mainly falls in the summer period, from October to March. While precipitation within the catchment influences the discharge of the Klip River, the precipitation within Ladysmith influences the drainage system in the town.

The flood duration curve of the Klip River in Figure 4.1 shows a steep curve in the upper region. This steep curve indicates that this particular river is likely to be subjected to floods caused by precipitation that originates from a relatively small catchment [35].

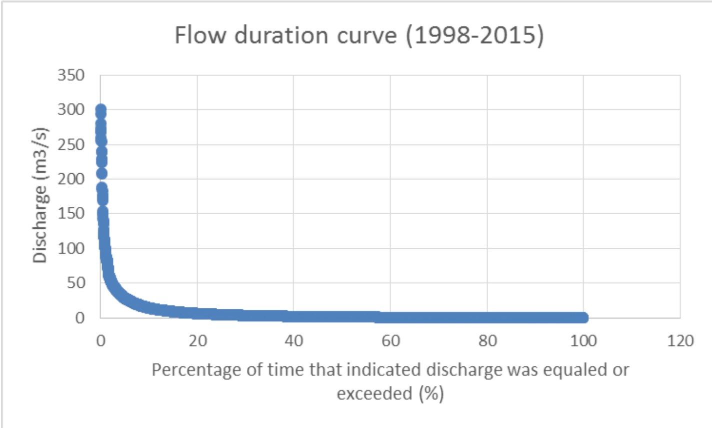


Figure 4.1: Flow duration curve (1998-2015), V1H038

The conclusion based on Figure 4.1 is confirmed by Figure 4.2. This Figure shows a clear correlation between precipitation and discharge for the Klip River. It can thus be concluded that the Klip River is sensitive to precipitation. Therefore the amount and the intensity of precipitation within the catchment and Ladysmith itself are important.

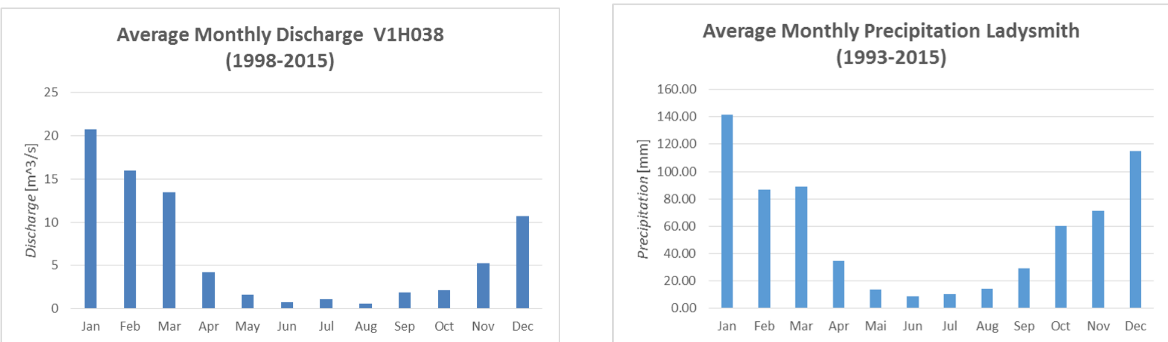


Figure 4.2: Average monthly discharge and precipitation at Ladysmith

Precipitation was analysed for both the entire catchment and Ladysmith. For the entire catchment daily precipitation data for the years 1965 -2014 were analysed. These data were obtained from several measurement stations in the catchment. For Ladysmith, hourly precipitation data for the years 1993 -2015 were analysed. These data were obtained from one measurement station, located three km from the CBD. Both analyses are explained in more detail in Appendix F. The precipitation is analysed for the entire catchment and for Ladysmith.

There have been no significant changes over time in the precipitation in the entire catchment. Therefore, the precipitation in the catchment could not have caused the flood events since the construction of the dam. The results of the analysis on rainfall data for the entire catchment show that there is no significant increase or decrease in the daily amount of precipitation. The change of the daily extreme values is also not significant. The 1 : 100 daily rainfall amount for the upstream catchment even slightly decreased.

Because of a lack of data, it could not be determined whether a change in precipitation in Ladysmith could have contributed to the occurrence of the recent flood events. In order to determine the difference in precipitation in Ladysmith before and after the construction of the dam, a trend analysis should be done. For the precipitation data of Ladysmith, no trend analysis could be done because the data record was not long enough.

Evaporation

The evaporation is another factor that could influence the discharge of the Klip River. The evaporation within the catchment and Ladysmith are analysed separately. There are different types of evaporation, for example open water evaporation, soil evaporation, transpiration and interception evaporation. For this analysis the most important one is evaporation from interception. All water that is not intercepted will run off in the direction of the river or will infiltrate into the soil. Soil moisture will also reach the river, however at a larger time scale. Thus, if the interception decreases, a larger part of the precipitation will flow into the river.

The interception has been analysed in Subsection F.1.1. From this analysis it can be concluded that there are no trends in the interception within the catchment or Ladysmith.

Land use

The land use within the catchment and Ladysmith influences the amount of water that will runoff to the river and the velocity of this runoff. In order to analyse changes in land use, the following three factors are of importance: vegetation, soil moisture and urbanization. All three have been analysed in Subsection F.1.2. In this section only the main conclusions are given.

The last couple of years the vegetation has changed significantly. Nowadays, there is less vegetation and less healthy vegetation compared to 1990. This means that less water will be captured by vegetation, so a larger part of the precipitation will runoff to the river. Data of the soil moisture have only been monitored after the construction of the dam. Therefore no conclusion on the difference in soil moisture before and after the construction of the dam can be drawn. The urbanization within the catchment and Ladysmith itself did not change significantly over the years.

4.1.2. Causes related to the Klip River

The general characteristics of the Klip River within the catchment have been discussed in Chapter 2. In this section the Klip River along the CBD is analysed in more detail. The river geometry and resistance along the Klip River are discussed.

River geometry

The geometry of the river has a large influence on the behavior of the Klip River along the CBD. An important characteristic is the significant change in slope of the Klip River, referred to as a knickpoint, at the beginning of the Ladysmith CBD. This can be seen in the longitudinal profile in Figure 2.7. As the discharge does not change, this will result in a larger water depth downstream of the knickpoint and a backwater curve. The knickpoint is a natural characteristic of this river system and it is assumed that it has not changed significantly since the establishment of Ladysmith in 1850. As was already described in Chapter 2, the settlement of the city in the natural floodplains of the Klip River and in the vicinity of the knickpoint explains why the area has always been prone to floods. The settlement of the city in the natural floodplains can be seen in Figure 4.3.

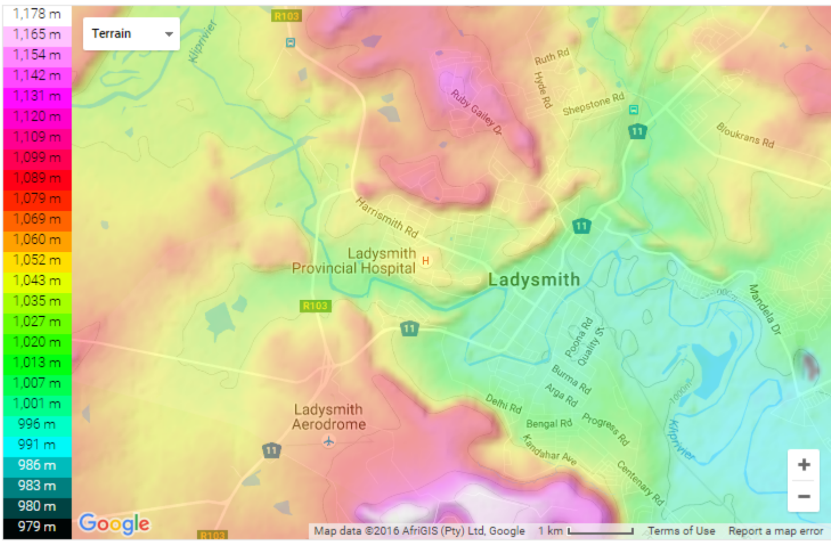


Figure 4.3: Elevation map of Ladysmith from Google Maps™

As part of the LFPS, 31 cross-sections along the Klip River are monitored every four years. The results of these surveys are summarized in the 2013 floodline report of Royal HaskoningDHV [29], see Section B.2. The report states that no general pattern of change in river geometry can be seen for the 31 cross-sections, since the baseline study in 1998. It is noted however, that changes were observed for four particular cross-sections, since the previous survey in 2009. Two of these were subject to sediment deposition and the other two were subject to erosion. A more detailed investigation was recommended for these cross-sections during the next survey.

Resistance

Several important hydraulic parameters in the Klip River are determined by the resistance exerted on the flow. This hydraulic roughness is commonly expressed by using Chézy's or Manning's friction coefficients. The exerted resistance on the flow is among other things dependent on soil and sediment characteristics, vegetation, channel obstructions and bedforms.

Water flow is influenced by the interaction between the sediment and water. The distinctive names of the two main tributaries, the Sand and Klip (Stone) River, give an indication of the sediment. The sediment characteristics of the Klip River along the CBD were mainly analysed from field observations. The sediment in the Klip River can be characterised as highly diverse, ranging from silt to large boulders. Upstream and downstream of the knickpoint a clear difference is observed, due to the change in slope. Upstream of the knickpoint the reach is characterised by a riffle-pool sequence of areas with large accumulated boulders alternated with fine sediment, see Figure 4.4a. The boulders found in the river become finer more downstream. Downstream of the knickpoint the riffles disappear and the floodplains widen and flatten, see Figure 4.4b. In Appendix D, Figure D.2, Figure D.3 and Figure D.4 give a clear overview of the differences between the reaches. Because of these differences, the Manning coefficients upstream of the knickpoint will probably be higher than the Manning coefficients downstream of the knickpoint. As mentioned before the changes of the monitored cross-sections reported by Royal HaskoningDHV [29] are limited since the baseline study in 1998. The influence of the Qedusizi dam on the sediment is probably minimal, as discussed inSubsection 4.1.3. Information on bedforms of the Klip River along the CBD is not available.

(a) Upstream of knickpoint

(b) Downstream of knickpoint

During the fieldwork it was observed that there are large differences in the type and amount of vegetation in the floodplains along different sections of the river. The types of vegetation mostly seen in the floodplains were high grass, rough bushes and trees. Along some parts of the river the vegetation was cut or burned down, see Subsection D.1.2.

A variety of structures is located in the riverbed and floodplains. These structures reduce the cross-sectional area of the Klip River. Structures can be identified as car bridges, pedestrian bridges, drainage outlets, pumps and one weir. In Subsection D.1.3 a detailed description is given of all elements that were observed during the field visit. These bridges and obstacles can contribute to higher water levels in the river. The blockage of a part of the river section will result in higher water levels upstream of the (partly) blocked location due to backwater effects. However, the overall contribution is expected to be small.

Royal HaskoningDHV made a model to determine the water levels in the Klip River. In this model, the roughness was indicated per cross-section with Manning's coefficients between 0.040 and 0.045 for the main channel and with Manning's coefficients between 0.050 and 0.065 for the flood plains [29]. Considering the large variety between the reaches upstream and downstream of the knickpoint, it seems that the range used for the Manning coefficients is quite small, especially for the main channel. Based on field observations, alternative values were determined for the Manning roughness coefficients. The Manning coefficients were determined for the reach upstream and downstream of the knickpoint. The final Manning coefficients are summarized in Table 4.1 and Table 4.2. Section F.2 elaborates on those values and the assumptions made for the determination of the Manning coefficients are explained. The estimated Manning coefficients show a strong contrast between the reach upstream and downstream of the knickpoint. In addition, the minimum Manning coefficients from Table 4.1 for the main channel, upstream of the knickpoint, are higher than the highest value 0.045 used in the 2013 flood line report. The impact of the use of different Manning's coefficients for separate reaches is to be evaluated.

Manning	Main river channel			Floodplains		
Factor	**Min.**	**Med.**	**Max.**	**Min.**	**Med.**	**Max.**
n	0.0494	0.0839	0.1248	0.036	0.056	0.115

Table 4.1: Manning values, upstream of knickpoint

Manning	Main river channel			Floodplains		
Factor	**Min.**	**Med.**	**Max.**	**Min.**	**Med.**	**Max.**
n	0.0299	0.0585	0.0923	0.036	0.056	0.115

Table 4.2: Manning values, downstream of knickpoint

4.1.3. Causes related to the LFPS

In this section the Ladysmith Flood Protection Scheme, which is mainly constructed around the focus area, is analysed in more detail. The Qeduzi dam, the levees and the stormwater valves along the Klip River are discussed.

Qedusizi Dam

The Qedusizi dam is the main feature of the LFPS. This dam has a flood attenuation function and is located approximately 5.5 km upstream of Ladysmith. In terms of the Dam Safety Regulations the dam has been classified as a Category III dam with a high hazard potential. In order to function properly it is important that the design, operation and maintenance of the dam is correct. It is difficult to check the entire design of the Qedusizi Dam, because not all the assumptions and methods on which the design was based are available. Therefore, the analysis of the dam was focused on the most important design assumptions and features, such as the height of the dam and the discharge through the outlets. Additionally, the maintenance of the dam and the sedimentation of the reservoir behind the dam is discussed.

The design of the dam was based on the assumption that the river bed around the CBD of Ladysmith has a maximum capacity of 450 m^3/s. This was found in several reports regarding the design of the Qedusizi dam [19]. However, the ratio between the contribution of the discharge through the outlets of the Qedusizi dam and the discharge of the intermediate catchment between the dam and Ladysmith differs in reports. The Flagstone Spruit is the main contribution in the intermediate catchment, see Figure 4.5. In the Dam Safety Inspection (DSI) of the dam in 2010, the outlet design capacity is said to be 400 m^3/s and the attribution of the intermediate catchment is said to be 50 m^3/s [19]. Another source claims a ratio of 386 m^3/s and 64 m^3/s [15].

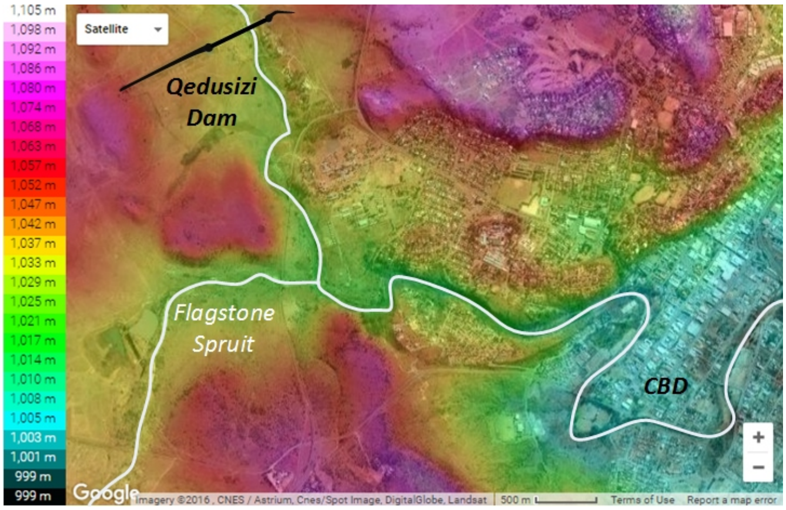


Figure 4.5: The intermediate catchment between the dam and Ladysmith CBD, from Google Maps™

It is difficult to determine whether these assumptions are correct. The discharge capacity of the river bed around the CBD of Ladysmith cannot be easily checked. A simple calculation was done in order to get an indication of the water depth around the CBD of Ladysmith for a discharge of 450 m^3/s. This calculation can be found in Subsection F.3.1. This calculation shows that the water level for this discharge is probably quite high. Since this calculation was carried out for a simplified system, more research should be carried out to determine a better estimate of the water level around the CBD of Ladysmith.

In order to check the assumption that between the dam and the CBD 50 m^3/s - 64 m^3/s is attributed to the Klip River, some simple calculations were carried out. These calculations are based on the data provided by measurement stations V1R005 and V1H038a and are described in more detail in Appendix F. The results of these calculations show that the assumption of 50 m^3/s - 64 m^3/s is quite low. Also, a matlab script was used to compare the discharge of measuring stations V1R005 and V1H038 during periods of high water levels in the Klip River. The results of this analysis showed that the attribution of the Flagstone Spruit to the discharge in Klip River often exceeds 50 m^3/s. An example is shown in Figure 4.6. The results also showed that a big difference in discharge between these two measuring stations was often accompanied by heavy hourly rainfall in Ladysmith. This suggests that the catchment of the Flagstone Spruit is very sensitive to rainfall and has a very direct response.

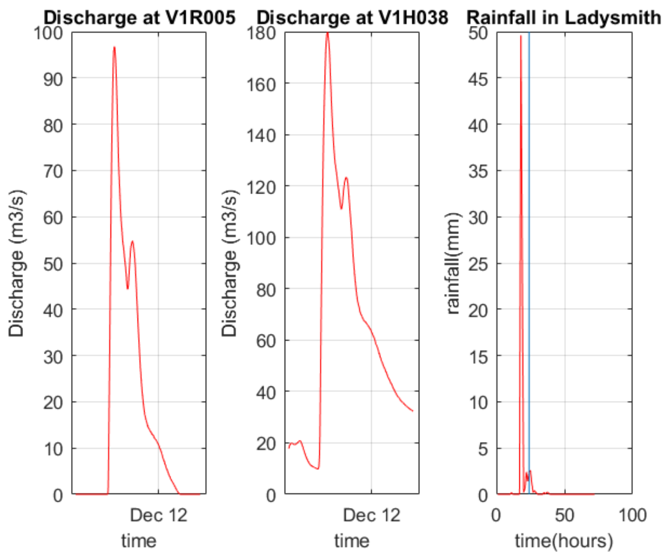


Figure 4.6: Discharge at V1R005 and V1H038 in December 2012

The most important design aspects in the context of this research are the height of the dam and the amount of water that flows through it. The height of the dam is 20.5 m. If the water level behind the dam is equal to this maximum level, the flow through the dam is expected to be 400 m^3/s. The water level just behind the Qedusizi Dam and the accompanying discharge through the dam have been monitored since the construction of the dam in 1998. These data were used to analyse the design features in more detail. The analyses are described in more detail in Subsection F.3.3 and Subsection F.3.4.

Both design aspects of the dam seem to be correct. Using an extreme value analysis on the water levels measured by measurement station V1R005, the water level behind the dam for a return period of 100 years was calculated. The results of this analysis are summarized in Table 4.3. The calculated water level with a return period of 100 years is lower than 20.5 m. Therefore the dam will not be topped over for the 1 in 100 year event. Also, the accompanying discharge of 386 m^3/s is correct according to the calculations made.

Return period [years]	Water level (Gumbel) [m]	Water level (Pearson) [m]
1:10	11.9	11.1
1:50	16.9	15.9
1:100	19.1	18.0

Table 4.3: The water level behind the dam with associated return period according to the Gumbel and Pearson distribution

Dam safety is legislated under the National Water Act of 1998. This Water Act requires that a registered dam is evaluated every five years. The most recent Dam Safety Inspection of the Qedusizi Dam was performed in March 2010 [19]. In the report it is stated that the dam is being well maintained. However, no additional information on the maintenance is specified.

An additional factor that might influence the performance of the dam is the sedimentation of the dam reservoir. Because the dam has two uncontrolled bottom outlets, water and sediment are able to move freely through the outlets. Therefore, the sedimentation in front of the dam is expected to be low. Due to the bottom outlets, the dam does not have a storage function. This choice was made deliberately during the design of the dam since sedimentation in front of the dam, and thus a decrease in capacity, was expected if a storage function was implemented [60]. The water level upstream of the dam exceeds the top of the outlets only during a few days a year. During these periods the dam attenuates the high water levels and sedimentation rates increase. The duration and amount of these periods differ per year, but are not considered long enough to contribute significantly to sedimentation in the reservoir. In the Operation and Maintanance of the Qedusizi dam of 1998 [37], it is mentioned that allowance has been made for the accumulation of 20.0 million m^3 after a period of 50 years. So, sedimentation of the reservoir is included in the design.

Levees

The levees surrounding some parts of the Ladysmith CBD have been constructed in order to prevent the Klip River from bursting its banks during high water levels. The locations of the levees are mapped during different cross-section surveys commissioned by Royal HaskoningDHV, see Figure B.4 and Appendix B. From the results of survey it can be concluded the height of the levees differs between the cross-sections. Also, the height differs between the two river sides within one cross-section. This was confirmed during field observations.

In the floodline report, the results of the analysis of the freeboards along the different sections of the Klip river are described. These freeboards are defined as the distance between the flood water levels for different return periods (as determined by Royal HaskoningDHV) and the highest crest levels at either side of the river (as determined in the surveys commissioned by Royal HaskoningDHV). In the report it is concluded that at some locations the freeboard is not sufficient for a 1 in 100 year flood event. However, these results should be treated with care since the model that was used to calculate the water levels was not calibrated.

Stormwater valves

The drainage system of the CBD discharges the stormwater into the Klip River. At some locations, the outfalls are located at a low level compared to the water levels in the Klip River. For many years, stormvalves have been used to prevent backflow from the Klip River into the drainage system. Different types were used, such as sluice and flap gates. Examples can be found in Figure 4.7a and Figure 4.7b. The sluice and flap gates did not perform adequately and required a lot of maintenance for basic operation. Also the sluice gates require manual operation which is inconvenient during nighttime or in case of flash floods. In 1997, the predecessor of Royal HaskoningDHV, Steward Scott Incorportated (SSI) did a study on the use of Tideflex® valves as an alternative to the sluice and flap gates installed on the stormwater drainage outlets at that time.

Tideflex® valves from the Red Valve Company, shown in Figure 4.7c, are basically a rubber construction with a curved bill that is normally closed. It is stated that they are sensitive enough to open with as little as 2.54 cm of pressure difference. They are supposed to be self-draining, need little to no maintenance, seal around small debris and have a lifespan of 35 years [38][36]. SSI proposed three different options to replace the storm water valves with Tideflex® valves [36]. From the 2011 report made by SSI on an assessment of the storm valves, done in October 2010, it is found that at that time not all the outfalls had been equipped with Tideflex® valves. This indicates that the municipality chose to execute one of the low-cost options proposed by SSI in 1997, see Table B.1. A comparable report made by Royal HaskoningDHV in 2014 shows that by then all drainage outlets were equipped with Tideflex® valves. However, that same report of 2014, shows that some of the Tideflex® valves have gapping valve bills "that could allow enough backflow at high river levels to adversely impact upstream roads and developments" [28]. This is often the result of incorrect installment or the accumulation of sand and debris in or around the valves, decreasing the functionality of the system (see Figure B.2). From field observations the same conclusion is drawn, see Figure D.9a for an example.

(a) Sluice gate [32] (b) Flap gate [11] (c) Tideflex® valve [7]

Figure 4.7: Pictures of valve types

4.2. Causes related to the drainage system

For the analysis of the drainage system a detailed map of the stormwater drainage [12] was used. The drainage system of the Ladysmith CBD consists of open and closed drainage pipes. The drainage map shows the different dimensions of the used pipes. All the pipes are made of concrete. For these concrete pipes, a Manning coefficient of 0.012 was assumed [31].

Based on the analysis of the drainage map, it seems that the design of the drainage system is not sufficient. Namely, the manholes are disproportionally divided over the area. Therefore, some manholes have to drain a much larger catchment than others. Furthermore, sometimes the successive pipe diameters do not seem to have a logical order. For example, the diameter of the pipe after the junction of two other pipes is sometimes small compared to sum of the separate pipe diameters before. However, the quality of the design cannot be tested because of a lack of data.

The maximum capacity of the drainage system depends on several factors. First, it depends on the surface runoff. The surface runoff is defined as the amount of water that flows into a manhole per unit of time. This runoff depends on the rainfall intensity and the surface properties. An example of these properties is the ratio of pervious to impervious area. Furthermore, the capacity depends on the volume of the drainage system, the area of the catchment that is drained by the system and the flow velocity in the system. Both the surface runoff and the velocity in the system are hard to determine by hand. Thus, a model is needed to calculate the capacity of the drainage system.

From field observation and news reports, see Appendix C and Appendix D, it can be concluded that the maintenance of the drainage system is inadequate. Based on the observation of the visible part of the drainage systems during the field trip, the blockage of the pipes due to garbage is estimated to be around 40%. The drainage pipe of Figure 4.8 is representative of the general state of the drainage system as observed during the field trip. Not only the inlets are blocked by the garbage, a comparable amount of garbage was observed in the drainage pipes. As can be seen in Figure 4.8 the inlet of the pipe is almost half blocked.

Another factor that might influence the drainage system is the sewerage system of Ladysmith. At some points the sewerage system is blocked and in a bad state due due to a lack of maintenance [41] [44] [55] [49]. Sewage is then able to leak into open channels or the storm water drainage system, which discharges untreated to the Klip River.

Figure 4.8: Blocked drainage inlet CBD Ladysmith

4.3. Characteristics of flood events

In this section the flood events within the focus area, as described in Chapter 3, will be analysed in more detail. The similarities and characteristics of the floods are described in order to get a better insight in the causes of the problem. First, the location of the flood events within the Ladysmith CBD will be explained. Subsequently the precipitation, water level and discharge during the flood events will be analysed to determine whether extremes in precipitation and discharge occurred during these events.

4.3.1. Location

From different sources it can be concluded that within the focus area the Lyell Street and its surroundings are most prone to floods. Especially the intersections of the Lyell Street with the King Street, Queen Street and Alexandra Street are mentioned in numerous news reports. The Forbes street, which runs parallel to the Lyell Street, is mentioned in some reports but significantly less than the Lyell Street. This seems strange, since Figure 4.9 shows that these two streets have the same elevation. An explanation could be the fact that the Lyell street has a higher economic value.

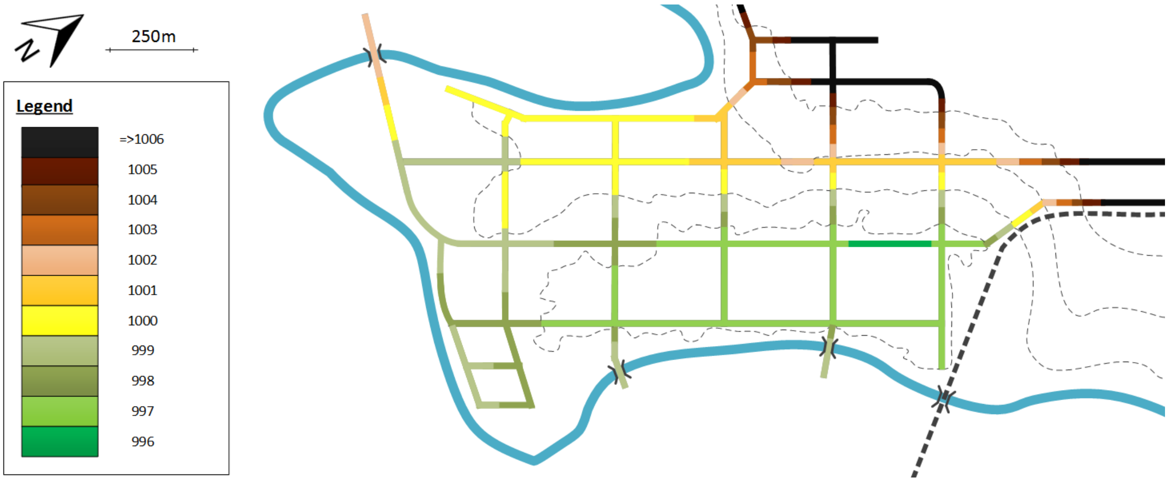


Figure 4.9: Elevation of the roads in the Ladymith CBD

From Figure 4.9 it can be seen that the streets prone to floods are located in the lowest part of the CBD of Ladysmith.

4.3.2. Precipitation

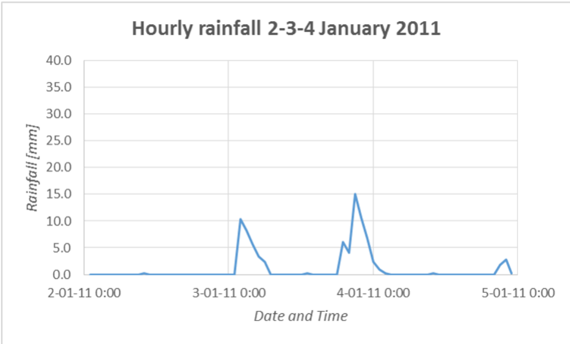
A large amount of the news reports analysed in Appendix D mention heavy rainfall during flood events. An extreme value distribution of the precipitation within Ladysmith was conducted in Section F.1. The results of this distribution are used in the analysis of the precipitation during flood events, see Table 4.4.

	Daily rainfall		**Hourly rainfall**	
Flood event	Amount [mm/day]	Return period [year]	Maximum [mm/hour]	Return period [year]
January 2011	75.0	8	15.0	<1
January 2012	76.0	9	37.6	4
September 2012	91.0	22	14.8	<1
December 2015	44.8	1	25.0	1

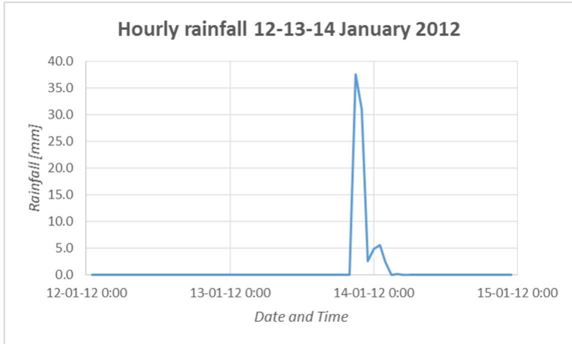
Table 4.4: Daily and hourly rainfall with return periods during flood events

From Table 4.4 it can be seen that during some flood events extreme daily rainfall occurred. The extreme daily rainfall events that occurred since 1993 were ranked from maximum to minimum values. Number 1,4 and 5 of this list occurred during the flood event of September 2012, January 2012 and January 2011 respectively. Number 2 of this list was a hail storm that occurred in December 2012, see Appendix C. Number 3 occurred on 25 January 1996. This was before the construction of the dam and is therefore outside of the scope of this research. It can be concluded that the daily rainfall amounts during the flood events were extreme and probably contributed to the occurrence of the floods.

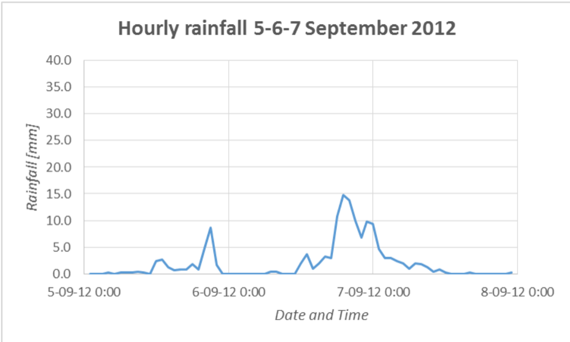
Figure 4.10 shows the hourly rainfall during the days surrounding the flood events. It can be seen that during the events of January 2011 and September 2012 the rainfall was constant and had a low intensity. The rainfall during the events of January 2012 and December 2015 on the other hand were short and had a high intensity. From Table 4.4 it can be seen that the hourly rainfall intensity was not extreme. The maximum return period of the hourly rainfall during the flood events was 3.56 year. This rainfall event occurred in January 2012.



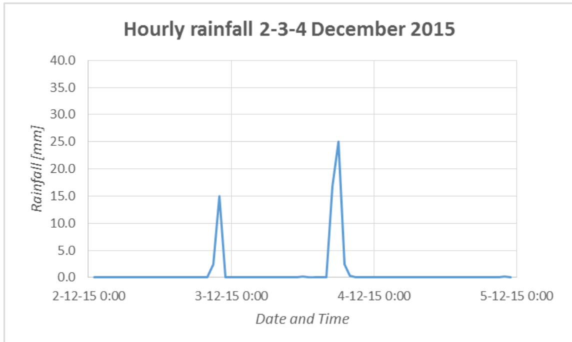
(a) January 2011



(b) January 2012



(c) September 2012



(d) December 2015

Figure 4.10: Precipitation of multiple days surrounding the flood events

4.3.3. Discharge and water level

The water level behind the Qedusizi dam and the discharge in the Klip River are important characteristics of the flood events. An extreme value analysis of the water level behind the dam was conducted and is described in Subsection F.3.3. The results of this analysis are described in this section.

	V1R005			**V1H038**	
Flood event	**Max. water level [m]**	**Max. discharge [m^3/s]**	**Return period [year]**	**Max. discharge [m^3/s]**	**Return period [year]**
January 2011	10.4	270	8	332	13
January 2012	6.0	204	6	241	5
September 2012	10.4	270	8	313	11
December 2015	0.6	14	<1	125	2

Table 4.5: Water level and discharge of measurements station V1R005 and V1H038 during flood events with return period

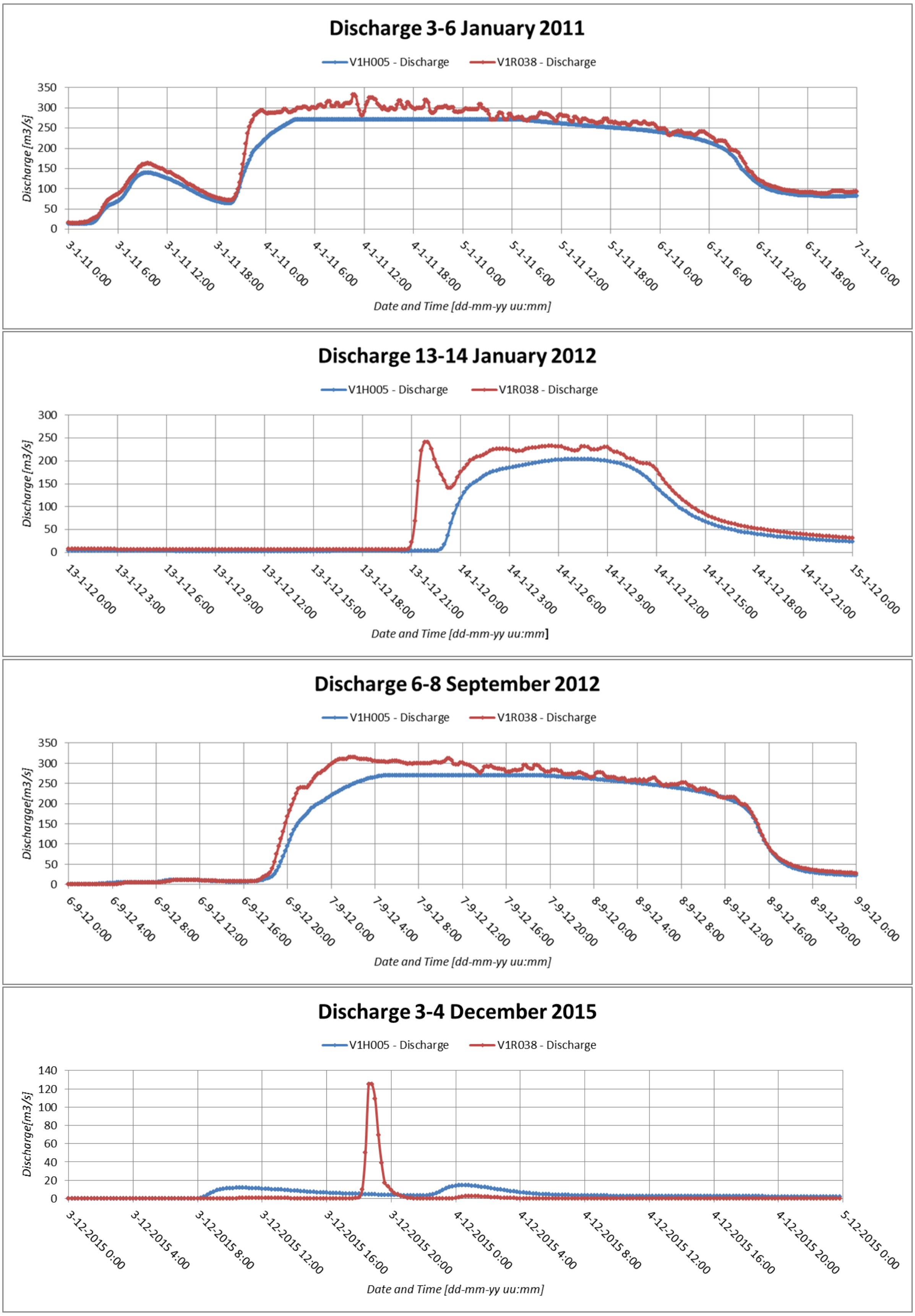


Figure 4.11: Discharge measurements in the Klip River during flood events

From Figure 4.11 it can be seen that the discharge during the flood events of January 2011 and September 2012 is high for a relatively long period: 2.5 and 1.5 days respectively. Table 4.5 shows that the discharge at the Qedusizi Dam and the discharge in the Klip River were roughly the same for the flood events of January 2011, January 2012 and September 2012. During the flood event of December 2015, the discharge in the Klip River was significantly higher than the discharge at the dam. This indicates heavy rainfall in Ladysmith during this flood event. Table 4.5 also shows the return period for all events. These return periods are not extreme.

The high discharge during the flood events of January 2012 and December 2015 are of shorter duration. The discharge peak in January 2012 lasts approximately 16 hours, with a maximum between 200 and 250 m^3/s. The discharge peak in December 2015 lasts approximately two hours.

4.4. Conclusion

The causes related to the river system have been analysed in Section 4.1. The following can be concluded: looking at the entire catchment of the Klip River, no significant changes occurred in precipitation, evaporation and land use since the construction of the dam. The monitoring of the cross-sections done by Royal HaskoningDHV indicates that these show no adaption to the new hydraulic conditions imposed by the dam. Furthermore, the levees along the CBD differ in height. Calculations indicated that the Manning coefficients in the floodline monitoring done by Royal HaskoningDHV are probably quite low. The design and functioning of the Qedusizi Dam appears correct, however the contribution of the intermediate catchment between the dam and town seems underestimated. An important conclusion about the current stormwater valves is that they may not work properly due to wrong installation and a lack of maintenance. Additionally, during the flood events in 2011 and 2012 not all outlets were equipped with Tideflex® valves.

Regarding the drainage system, it can be concluded that the design may not be sufficient. Also, due to the bad state and high amounts of garbage in the system, the drainage system may not function optimally.

Three characteristics of the flood events have been identified. First of all, floods occured in the lower-laying parts of the Ladysmith CBD. Secondly, the flood events were accompanied by extreme daily rainfall events. For the flood events of January 2012 and December 2015, also the hourly rainfall was extreme. This extreme rainfall lasted only for a short period. Thirdly, during flood events water levels in the Klip River are high, however not necessarily extreme. Furthermore, it should be noted that the water levels in the Klip River were high for multiple days in January 2011 and September 2012, while the other events experienced high waters for less then 24 hours. According to the news reports, the flood events of January 2011 and September 2012 were more severe compared to the other two.

During the analyses described in this chapter, the possible causes were investigated qualitatively or with simple calculations. It has to be investigated in a more quantitative manner whether the identified possible causes really contributed to the occurrence of the flood events.

5

Modelling of the river and drainage system

The recent flood events in Ladysmith are most probably caused by a combination of garbage in the drainage system and stormwater valves that either do not function properly or are closed as a consequence of high water levels in the Klip River. Various factors influence these causes. In this chapter two models, Hydrologic Engineering's Center's River Analysis System (HEC-RAS) and Storm Water Management Model (SWMM) are used to test this hypothesis. A sensitivity analysis will give a (direction to) quantify the impact of each of the causes. However, these past flood events were not accompanied by extreme conditions. Therefore, future flood events were modelled, where the precipitation and discharge are extreme. The most probable causes of these events were also investigated. This chapter will start with explaining the choice for these models. Secondly, the models and the main assumptions are described. Finally, it can be concluded what causes the flood events, including the sensitivity of each of the causes.

5.1. Choice of models

The hypothesis that the recent flood events of the CBD are caused by a combination of the current state of the drainage system and the functioning of the stormwater valves had to be tested. In order to test whether the water level of the Klip River was high enough to close the stormwater valves, HEC-RAS is used. HEC-RAS is also used to test what will happen in case of extreme discharges. To investigate the sensitivity of the drainage system on garbage and the closure of the stormwater valves, SWMM is used. Both the past flood events and the extreme precipitation scenarios are used for testing. The two models are both needed in order to test whether broken or closed stormwater valves have been a cause of the past flood events or could be in the future.

HEC-RAS can be used to model the hydraulics of water flow through natural rivers. The HEC-RAS model of the Klip River is made by Royal HaskoningDHV and is updated yearly. The latest version dates from October 2016 and is used as starting point of this research. The October 2016 version is based on surveys carried out in September 2016. To test whether the water level of the Klip River was too high during the past flood events, the discharge during these floods was taken as input for the model. The HEC-RAS model gives the water levels of the Klip River during the recent flood events as output. In the analysis of the past flood events it is already noted that the discharge at the moment of the past floods was not extreme and the water level remained lower than the crest level. However, in the case of extreme discharges the water level might be higher than the crest level, causing a major flood. Therefore, extreme discharges are also tested with the HEC-RAS model of the Klip River. In the analysis it was concluded that the factors garbage in the river, the amount of vegetation in the river, the roughness of the vegetation and the roughness of the sediment contribute to the Manning coefficient and thereby influence the water level. The sensitivity of the river for these factors is tested with the HEC-RAS model by varying the Manning coefficient.

SWMM is a simulation model used for planning, analysis and design of different aspects regarding drainage in urban areas [1]. The goal of using a SWMM model was to test the sensitivity of the drainage system on garbage and the closure of (certain) stormwater valves. The model is built by using the drainage map of the Ladysmith CBD [12]. The input for the model is the precipitation during flood events, the amount of garbage, and the assumptions on the closure of the stormwater valves. The assumptions on the closure of the stormwater valves are based on the information of the functioning of the stormwater valves and the output of HEC-RAS.

Insufficient data is available to model the quantity of backflow at the location of broken or non-functioning stormwater valves. Therefore, this scenario is not tested. The influence of Manning's coefficient of the pipes on the occurrence of floods is also tested, since the Manning coefficient is different for a clean and a partly blocked system. The output of the SWMM model is the total volume that does not directly drain of via the drainage system and the location of the floods.

5.2. HEC-RAS

A HEC-RAS model was used in this project for two reasons. Firstly, the model was used to give an estimation of the water levels around Ladysmith during the past flood events. These water levels need to be compared to the height of the drained areas. If the water level of the Klip River is higher compared to the water level in the drainage system, the system is assumed to be closed. For all stormwater valves the water level in the Klip River have been determined. Secondly, the model was used to estimate the water levels around the CBD of Ladysmith for events with a return period of 10, 50 and 100 years. This was done in order to evaluate if the river bed would be able to cope with these discharges, or whether the levees would be overtopped during these events.

Model
The HEC-RAS model is used to analyse the water levels in the Klip River. Therefor the details of the the Klip River are needed, this characteristics are described in this paragraph. Furthermore, the input factors used for the model are described.

River system
The model that was used, was created by Royal HaskoningDHV. The general layout of the model can be seen in Figure 5.1. In the model, the river section of 23.5 km downstream of the Qedusizi Dam was modelled.

There are 31 cross-sections implemented in the model, most of which are located around the CBD of Ladysmith, see Figure 5.2. These cross-sections are the result of the surveys commissioned by Royal HaskoningDHV. These surveys are performed approximately every three years and for this model the results of the most recent survey in October 2016 were used. More information about the reports of the monitoring and cross-sections can be seen in Section B.2.

The river section that was modelled in this HEC-RAS model contains multiple bridges. In the model, only the bridges around the CBD of Ladysmith are implemented. There are five bridges in total. The other bridges, most of which are located in the river section between the Qedusizi Dam and the CBD, are not accounted for in the model. An overview of all the bridges in the river can be seen in Figure 5.3. The bridges that were implemented in the model are marked green, the ones that are not implemented are marked red.

During earlier applications of this model, a subcritical flow regime in the river was assumed. In order for the model to be able to run for this subcritical flow regime, a boundary condition at the downstream end of the model needs to be given. During earlier applications of the model by Royal HaskoningDHV, critical depth was assumed at the downstream end of the model. However, for this project normal flow depth (equilibrium depth) was assumed here. This assumption is explained in more detail in Subsection 5.2.1. Influences of a possible backwater curve from tributaries downstream on the lower boundary condition are disregarded. The first tributary that can influence the computational domain is a small stream located more than ten kilometers downstream of the lower boundary condition.

Another important input for the model are the Manning coefficients. The model requires a minimum of three Manning coefficients per cross-section: one for the river bed and two for the floodplains left and right of the riverbed. The determination of these Manning coefficients was also part of the surveys commissioned by Royal HaskoningDHV. During each survey, photos of the different cross-sections were made. The determination of the Manning coefficients was based on these photos. The Manning coefficients for the Klip River are likely to vary strongly over the year due to the large differences in vegetation between the dry and wet period. The surveys since the construction of the dam were done at different times in the year and did therefore not always lead to the correct Manning coefficients. The 2009 survey reflects the seasonal high vegetation growth since it was performed during the wet season [29]. Since the 2013 and 2016 surveys were completed during the dry season, for these years the Manning coefficients of the 2009 survey were used.

During the analysis phase of this project it was concluded that these Manning coefficients may not be realistic and seem a bit low. Using the guidelines of Chow [6], three different scenarios (low, medium and high) for the Manning coefficients were established. This is described in Appendix F. These three scenarios and the values from Royal HaskoningDHV were all used in the HEC-RAS model.

Input used for HEC-RAS model
The HEC-RAS model needs the discharge at every cross-section in order to be able to calculate the water levels in the river at these cross-sections. The discharge varies over the section of the river, since there are three tributaries that join the Klip River in this section. This can be seen in Figure G.2. The way in which these discharges were determined is explained in more detail in Subsection 5.2.1, where the model assumptions are described.

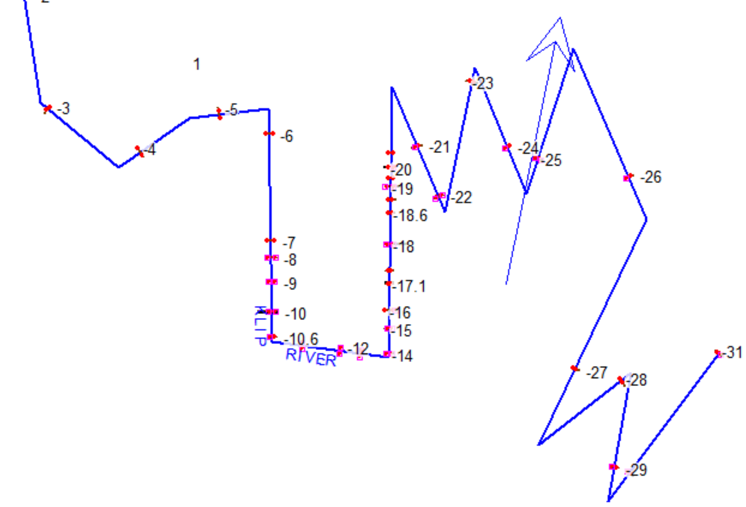


Figure 5.1: Layout of the HEC-RAS model

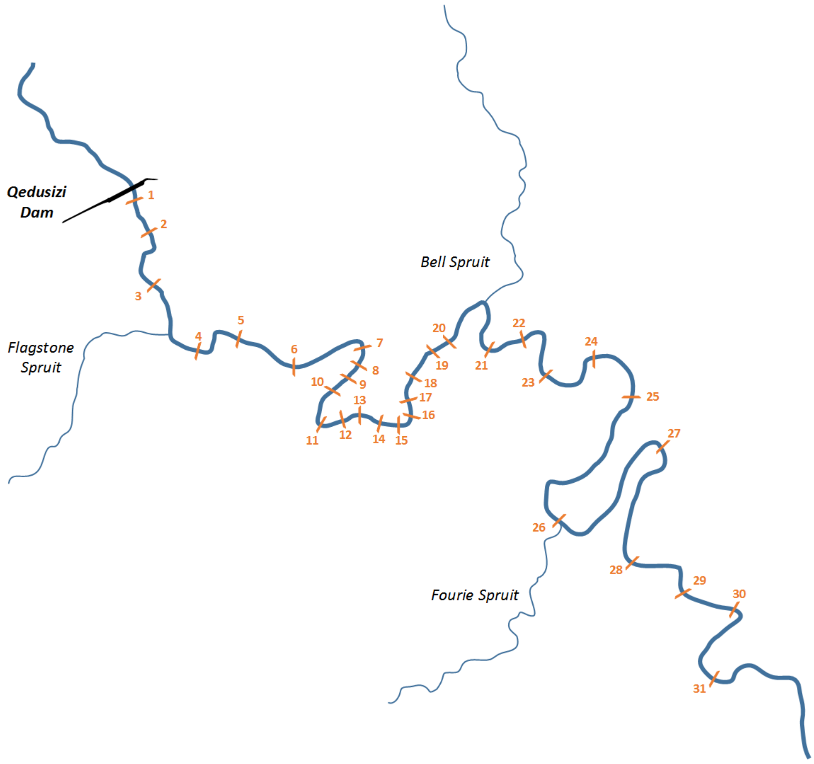


Figure 5.2: Cross-sections of the floodline monitoring by Royal HaskoningDHV

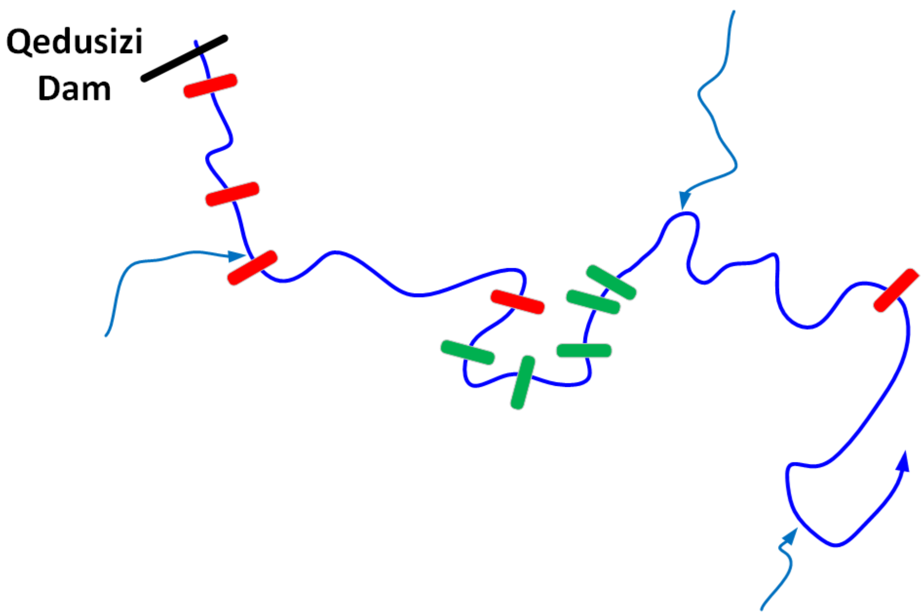


Figure 5.3: Bridges defined in the HEC-RAS model

5.2.1. Assumptions

Like with all models, the results are influenced by the assumptions that were made. A number of assumptions were already implicit in the program that was used, namely the HEC-RAS 1D steady flow model program. Subsequently, several assumptions were made when the river was modelled in this program. All these assumptions will be covered in this section.

Assumptions implicit in the used model program
The assumptions that are implicit in the HEC-RAS 1D steady flow model are a result of the computation procedure of the model. More information on this computation procedure can be found in Appendix G.

The first assumption that is implicit in the used program, is the assumption of steady flow. As was already described in Subsection 4.3.3, the period in which the discharge was high during the flood events of January 2011 and September 2012 lasted approximately two days. During the flood event of January 2012 the discharge remained high during a period of approximately 16 hours. For these three flood events, the assumption of steady flow seems justified for the period of high water. The flood event of December 2015 was not modelled in HEC-RAS, because the assumption of steady flow does not seem justified in this case.

The second assumption implicit in the HEC-RAS 1D steady flow model is that the flow in the river is gradually varied. This means that the water level changes only gradually over the course of the river section. For simplicity, over the majority of the Klip River gradually varying flow is assumed. During fieldwork no abrupt changes, that could cause rapid varying flow, were observed in the modelled river section.

Another assumption implicit in the used HEC-RAS model is that the flow is one dimensional. This reduces the accuracy of the model outcomes, because the river contains a lot of bends in which 2D-effects are expected to occur. However, since the model was only used as an indication for the water level, one dimensional flow was assumed.

The last implicit assumption in the HEC-RAS model is that the river channels have slopes smaller than a slope of 1:10. As was described in Section 2.4, the river section that was modelled can roughly be divided in two parts: the parts before and after the knickpoint in the riverbed. These parts have different slopes. The part before the knickpoint has an average slope of 1:200, which is much smaller than a slope of 1:10. The slope downstream of the knickpoint is even less steep ($i_b = 1:4000$).

Assumptions in building the model
When running the HEC-RAS model, a subcritical flow regime was assumed. Whether a flow is sub- or supercritical can be determined by calculating the Froude number. When using values for the parameters that seem realistic in this situation, a subcritical flow regime is found.

Since a subcritical flow regime was assumed, a downstream boundary needed to be implemented in the model. HEC-RAS has four possible types of boundary conditions: a fixed water level, critical depth, normal depth or a rating curve. Since the rating curve and the water level were unknown at the downstream boundary of the model, the choice was limited to two. Normal depth was chosen as a boundary condition, since this is more realistic in a river with a subcritical flow regime.

As was stated in Section 5.2 about the HEC-RAS model, the model contains 31 cross-sections. Most of these cross-sections are located around the CBD of Ladysmith, as can be seen on Figure 5.2. The computation procedure of the HEC-RAS model starts at the downstream end of the model. The first computation step is done by balancing this cross-section with the first cross-section upstream of it, to determine the water level in the upstream cross-section. Subsequently, this determined water level is used to calculate the water level in the next upstream cross-section. Since the river section downstream of the CBD of Ladysmith contains a small number of cross-sections, the water level that is calculated just downstream of the CBD is fairly rough. This influences the accuracy of the calculation of the water levels around the CBD of Ladysmith.

Furthermore, all cross-sections that were used in the model have been measured during a survey commissioned by Royal HaskoningDHV in 2016. These cross-sections were also used for the modelling of the past flood events, because the cross-sections during these events are unknown. This introduces an error in the modelling results, since the cross-sections were probably a bit different during these flood events. However, as can be seen from the cross-sectional survey plots in Appendix B, the cross-sections do not seem to change a lot over the years. The error is therefore expected to be small.

As was already explained in Section F.2, the determination of the Manning coefficients by Royal HaskoningDHV might not be entirely correct. As part of the analysis three scenarios were determined for the Manning coefficients: low, medium and high values. Because the determination of the Manning coefficients is difficult and subject to large uncertainties, all Manning scenarios were modelled in the HEC-RAS model. The exact values of the Manning coefficients for the different scenarios can be found in Section F.2.

As was already explained in Section 5.2, not all bridges in the river section are implemented in the model. As can be seen in Figure 5.3, most bridges around the CBD of Ladysmith are incorporated in the model. The bridges between the Qedusizi Dam and the CBD were not incorporated in the model. However, these bridges are not expected to have a large impact on the water level in downstream direction. Therefore, the error introduced by not taking these bridges into account in the model is expected to be small.

Input used for the model

As was explained before, the input used for the model consists of the discharge at every cross-section. In order to determine these discharges, the measuring stations V1R005 and V1H038 were used. Measuring station V1R005 is located just upstream of the dam and therefore the discharge data from this station are used as the inflow at the upstream boundary of the model. More information on the measuring stations can be found in Appendix A.

Along the modelled river sections there are three small tributaries that join the Klip River. The locations of these tributaries can be seen in Figure 5.2. Since there are no data on these tributaries, the discharge of these tributaries into the Klip River is not directly known. Measuring station V1H038 is located just downstream of the first tributary. For the first tributary, the discharge is assumed to be equal to the difference in discharge between measuring stations V1R005 and V1H038. Using Google Earth, the catchment area of each of the three tributaries was determined. The results are summarized in Table 5.1. The discharge of the other two tributaries was determined by scaling the discharge of the first tributary using the ratio of the catchment areas. This procedure is schematized in Figure 5.4. In this procedure, the catchment properties of all three tributaries are assumed to be the same. Also, the rainfall in the three catchments is assumed to be the same.

Tributary	**Surface area** (km^2)
Flagstone Spruit	50.7
Bell Spruit	38.0
Fourie Spruit	27.4

Table 5.1: Surface areas of the three tributaries

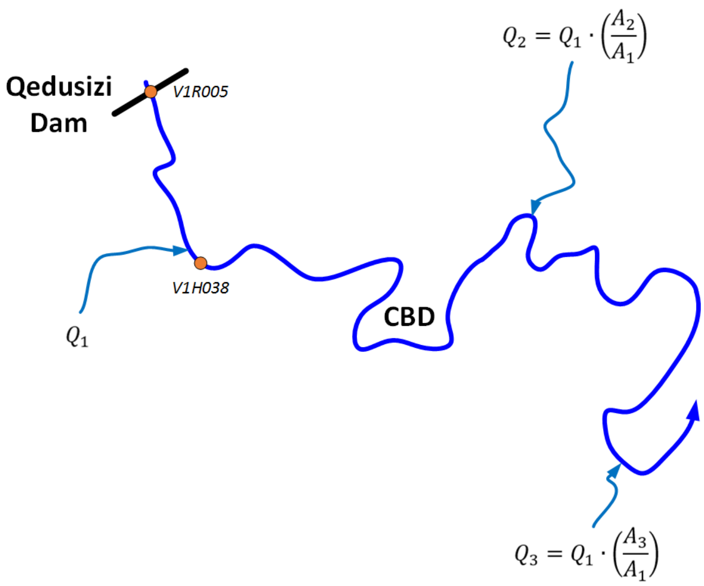


Figure 5.4: Adjoining tributaries in the HEC-RAS model

For the determination of the discharge during the past flood events, the maximum discharge at station V1H038 during the flood event was selected. As was described in Subsection 5.2.1, only the flood events of December 2011, January 2012 and September 2012 were modelled in HEC-RAS. The flood event of 2015 was not modelled in HEC-RAS, because the steady flow assumption does not seem to hold for this event. The discharge at station V1R005 was taken about 30 minutes before. The difference between these discharges was then used to determine the discharges in the tributaries. The results are summarized in Table 5.2.

For the determination of the discharge during the future flood events, extreme value analyses were performed on the water levels of both measuring stations. The extreme flood events that were modelled in HEC-RAS have a return period of 10, 50 and 100 years. For both stations, the discharges for different return periods were determined using the Gumbel distribution. The discharges of the three tributaries were determined by the procedure that was described above. The discharge in the first tributary was defined as the difference between the discharges in measuring station V1H038 and V1R005, for the same return period. More details about these analyses can be found in Appendix F. The results are summarized in Table 5.3.

Flood event	**Qedusizi Dam** $[m^3/s]$	**Flagstone Spruit** $[m^3/s]$	**Bell Spruit** $[m^3/s]$	**Fourie Spruit** $[m^3/s]$
January 2011	175	187	196	203
January 2012	270	332	378	412
September 2012	3	241	420	548
December 2015	238	315	372	414

Table 5.2: Discharges for the past flood events

Return period	**Qedusizi Dam** $[m^3/s]$	**Flagstone Spruit** $[m^3/s]$	**Bell Spruit** $[m^3/s]$	**Fourie Spruit** $[m^3/s]$
1:10	289	363	419	460
1:50	348	549	700	808
1:100	370	646	853	1003

Table 5.3: Discharges for the future flood events

5.2.2. Calibration

The used HEC-RAS model was not calibrated due to the absence of calibration data. There used to be a measuring station in the river section south of the CBD of Ladysmith. Unfortunately, this measuring station closed down a few years before the construction of the dam. These data would be necessary to calibrate the model. The results of the HEC-RAS model could therefore not be verified and should be used with care.

5.2.3. Model scenarios

The HEC-RAS model was run for different scenarios, that are linked to different values for the Manning coefficients. Scenario 1 uses the Manning coefficients as determined during the surveys by Royal HaskoningDHV. Scenario 2, 3 and 4 use the low, medium and high Manning coefficients as determined in Section F.2. All past and future flood events were modelled for these four different scenarios.

5.2.4. Results of sensitivity analysis of the Klip River

The model results can be subdivided in the results related to the modelling of the past flood events and of the future events. All events were compared to the crest levels, as determined during the survey in 2016 commissioned by Royal HaskoningDHV, and the water level in the drainage system. A comparison with the water level in the drainage system is necessary in order to determine if the drainage system in the CBD is able to discharge the stormwater into the Klip River. This will be further explained in Subsection 5.3.3.

Past flood events

Each past flood event was modelled four times, with different Manning coefficients for the riverbed and the floodplains. For the different scenarios, the water levels were compared with the crest levels along both sides of the river. The comparison with the left crest level (when looking in the downstream direction) is of particular interest in this study, since the CBD of Ladysmith is located at this side of the Klip River. The results of this analysis can be found in Appendix G.

From these results it can be concluded that for all flood events, the water level does not exceed the crest levels in scenario 1,2 and 3 (the Manning coefficients from Royal HaskoningDHV, the low and the medium values). Only for cross-section 16 the water level exceeds the left crest level in all scenarios. The location of this cross-section can be seen in Figure 5.2. For scenario 4, with high Manning values, the water level in the river exceeds the left crest level at multiple locations for all flood events.

As was described above, the water levels in the river were also compared to the water level in the drainage system. However, the water level in the drainage system is not measured or cannot be observed. Therefore, the following local streetlevel is taken: the topside of the first upstream manhole connected to a stormwater valve. If this local streetlevel is reached, the manhole is not able to cope with the amount of water in the system and the streets will flood. This level will be referred to as the local streetlevel, the term is explained in Figure 5.5. The complete overview of the results of the analysis can be found in Appendix G. The results which have to be used in SWMM are summarized in Table 5.4, the location of all stormwater valves can be found in Figure B.4.

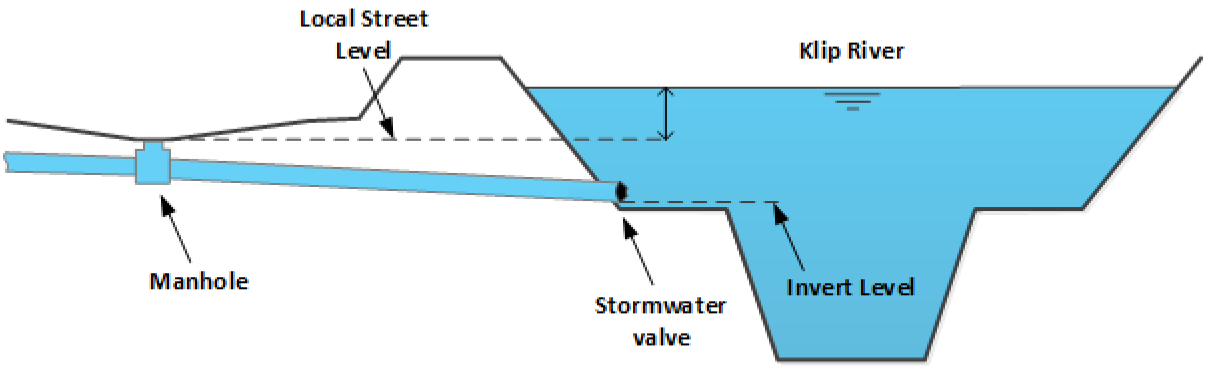


Figure 5.5: Explanation of the term local streetlevel

Flood event	Scenario 1: Manning Royal HaskoningDHV	Scenario 2: low Manning	Scenario 3: medium Manning	Scenario 4: high Manning
January 2011	water level in Klip River **lower** compared to all local streetlevels as defined in Figure 5.5	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C13C, C12, C10, C9, C8 and C7 closed	water level in Klip River **higher** compared to all local streetlevels
January 2011	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C10, C9, C8 and C7 closed	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C20, C16, C15, C14, C13A, C13C, C12A, C12, C11A, C10, C9, C8 and C7
September 2012	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C13C, C10, C9, C8 and C7 closed	water level in Klip River **higher** compared to all local streetlevels

Table 5.4: HEC-RAS results past flood events

Extreme flood events
As was done for the past flood events, all extreme flood events were modelled four times, each time with different Manning's coefficients for the riverbed and the floodplains. For all the different scenarios, the water levels were compared with the crest levels at both sides of the river. The results of this analysis can be found in Appendix G.

From these results it can be concluded that during the extreme flood with a return period of 10 years, the water level in the river does not exceed the crest levels at both sides of the river for scenarios 1, 2 and 3 (the Manning coefficients from Royal HaskoningDHV, the low and the medium Manning coefficients). Again, only the left crest level at cross-section 16 is overtopped. Scenario 4, with the high Manning coefficients, does show overtopping of the left crest levels at multiple locations. During the extreme floods with return periods of 50 and 100 years the left crest levels are overtopped at multiple locations in scenario 1, 3 and 4 (the Manning coefficients from Royal HaskoningDHV, the medium and high Manning coefficients). The amount of locations increases for increasing Manning coefficients. Also, the amount of locations is higher for the flood event with a return period of 100 years than for the flood event with a return period of 50 years.

As described above, the water levels in the Klip River were also compared to the local streetlevel. The results of this analysis can be found in Appendix G. The results which have to be used in SWMM are summarized in Table 5.5, the location of all stormwater valves can be found in Figure B.4.

Retun period	**Scenario 1: Manning Royal HaskoningDHV**	**Scenario 2: low Manning**	**Scenario 3: medium Manning**	**Scenario 4: high Manning**
1:10	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **lower** compared to all local streetlevels	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C12, C11A, C10, C9, C8 and C7	water level in Klip River **higher** compared to all local streetlevels
1:50	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C20, C16, C15, C14, C13A, C13C, C12A, C12, C11A, C10, C9, C8 and C7	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C13C, C12, C10, C9, C8 and C7	water level in Klip River **higher** compared to all reference	water level in Klip River **higher** compared to all reference
1:100	water level in Klip River **higher** compared to all reference	water level in Klip River **higher** compared to the local streetlevels of the stormwater valves C16, C15, C13A, C13C, C12A, C12, C11A, C10, C9, C8 and C7	water level in Klip River **higher** compared to all reference	water level in Klip River **higher** compared to all reference

Table 5.5: HEC-RAS results extreme events

5.3. SWMM

The goal of using SWMM is to test the sensitivity of the drainage system to the amount of garbage in the system and the closure of the stormwater valves. Based on literature research and fieldwork Ladysmith, see Appendix D, it is concluded that the drainage system contains a lot of garbage. This could be a reason for the not optimal functioning of the system. The low position and possibly even the non-functioning of the Tideflex® valves could also be a reason. Since the availability of data is limited, only the response of the drainage system on the factors can be tested.

5.3.1. Model and assumptions

To build a model in SWMM that can give this result, details about the drainage system and catchments are required. Subcatchments need to be defined, that each drain to either a manhole or another subcatchment. All subcatchments that drain to one outfall together form a catchment. For this model, the information needed is based on the drainage map of the CBD, delivered by Stewart Scott Consulting Engineering [12]. It is assumed that the map is still correct and did not change during the investigated period. The map gives an overview of the locations of drainage pipes and manholes including their dimensions. Some pipe diameters are missing, so they were measured during fieldwork Ladysmith, see Appendix D. During fieldwork it was observed that the manholes are not significantly larger in size that the pipes. Since the pipes are located to the sides of the streets an are only just below street level, a manhole is often a hole in the pipe. In the model the open drainage receives inflow from a subcatchment at only one point, just as a closed pipe, and is also only able to flood at that point. Flow of stormwater over the streets can also not be modelled. In the model the stormwater will only flow through pipes, after it has runoff from the subcatchment. If the pipes are full it will form a water column on the manhole that floods. It will not flow over the streets, but stay there until it can flow away through the manhole. In reality, of course, the water will divide itself over the street, and in the case of a slope, flow to the lowest point. If it comes past a manhole that is not flooding, the water will enter the drainage system at that point.

For the simplicity of the model, manholes are modelled as normal 'nodes' with a certain invert level. Crossings of pipes are modelled in the same way. This will influence both the accuracy of the location of the floods and the size of the floods, since in reality crossings of pipes will not flood. This assumption will make the results of the location of the floods less accurate. Besides the characteristics of the drainage system, the map also includes contour lines. Using these contour lines, the subcatchments could be determined. The characteristics (area, width, slope, Manning and D-store) of these subcatchments determine the rainfall-runoff and the velocity of the runoff to the drainage system. In this study, Google Earth^{TM} is used to provide information about the characteristics of the subcatchments. The visible aspects are combined with characteristic values of those visible aspects. The characteristic values are provided by United States Environmental Protection Agency (EPA) in their user guide of SWMM [31]. An extended description of these characteristics can be found in Appendix H.

The contour lines on the map are also used to determine the invert levels of the pipes, which SWMM uses to calculate the slope. Since no information is available on the depth of the drainage system, it is assumed that the manholes and pipes are at street level. This assumption is valid, because observations during the fieldwork, see Appendix D, showed that the pipes are often just below street level. Since the accuracy of the map is 0.5 m in height, many manholes are at the same level. SWMM is only able to determine flow if there is a hydraulic gradient. Thus, the invert levels increased by 5 cm starting from the node closest to the outlet, whenever the elevation was the same.

Since the rainfall data is provided by only one measurement station, the rainfall is assumed to be uniform for the entire area. The measurement station is located near the Ladysmith Aerodrome Airfield, approximately 3 kilometres south-west of the CBD. For more information the reader referred to Appendix A. This distance and the lack of more stations makes the data less reliable. Rainfall can be very local, which influences the results significantly. Evaporation is not taken into account, since there is no trend in the evaporation through the years, see Chapter 4. Besides this, only the relative effect of the precipitation on the drainage system is important for this study. The evaporation was probably slightly different for each flood event. However, taking the evaporation into account in detail is not possible since there is no data available on the precise local weather conditions during a flood. Additionally, there is no evaporation measurement station in Ladysmith.

Model overview

The interface of SWMM is shown below in Figure 5.6. The model summary is also shown below, see Table 5.6. For more information on the choices made in this model, see Appendix H. Table 5.6 shows that the used flow routing type is the Kinematic wave. The smallest possible timestep for this type 60 s, so this timestep is used.

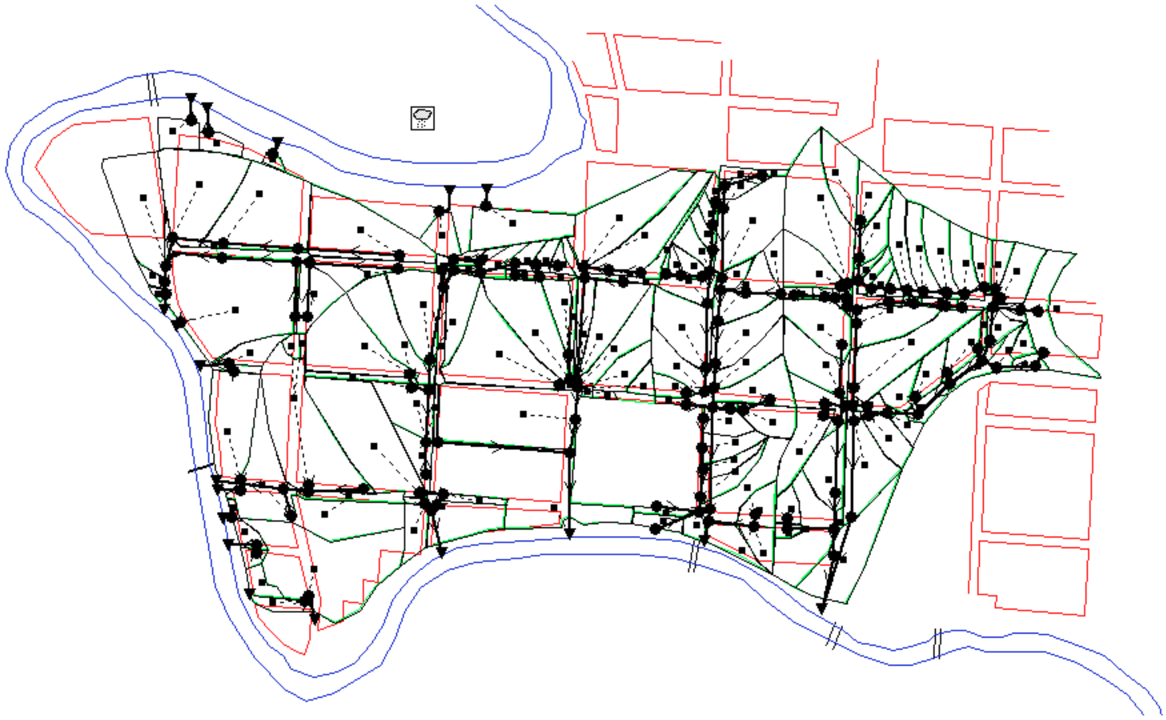


Figure 5.6: Interface SWMM model of the drainage system in the CBD

Raingauges	1	Flow Units	CMS
Subcatchments	159	Flow Routing	KINWAVE
Aquifers	0	Control Rules	0
Snowpacks	0	Land Uses	0
RDII Hydrographs	0	Time Series Inflows	0
Infiltration	HORTON	Dry Weather Inflows	0
Junction Nodes	161	Groundwater Inflows	0
Outfall Nodes	18	RDII Inflows	0
Flow Divider Nodes	0	Treatment Units	0
Storage Unit Nodes	0	Pollutants	0
Conduit Links	161	Outlet Links	0
Pump Links	0	Weir Links	0
Orifice Links	0		

Table 5.6: SWMM model summary

5.3.2. Calibration

Calibration of the SWMM model is not possible, since the data needed is not available. In order to calibrate the model data of the exact location of the floods and the flood volume at each location is needed. Data of more than four flood events is needed in order to get a more accurate calibration. With this uncalibrated model it is not possible to give accurate absolute values of the volume of the modelled floods. Therefore, all modelled factors will be reported relatively to a design scenario, which is explained further below.

5.3.3. Scenarios

Based on the analysis of the drainage system in Chapter 4, three influential factors are defines. Based on these three factors, eight scenarios could be defined. An overview of the influential factors can be seen in Figure 5.7.

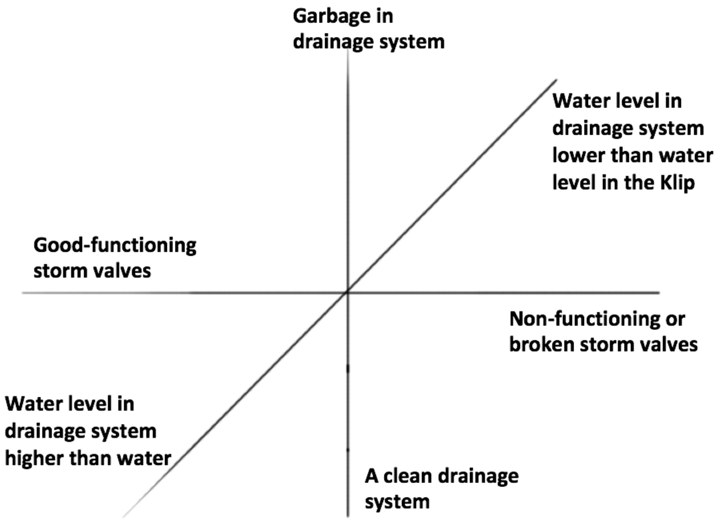


Figure 5.7: Overview of influential factors

Because of model limitations, it is not possible to model inflow from the Klip River into the drainage system. Therefore, only the scenarios including well functioning stormwater valves can be tested. Because the influence of each of the factors has to be determined, only the scenarios that change one factor at the time are relevant. Thus three scenarios are tested:

1. Design system: Clean, $h_{drainage} > h_{Klip}$, functioning valves
2. Garbage in the drainage system: Garbage, $h_{drainage} > h_{Klip}$, functioning stormwater valves
3. Closure stormwater valves: Clean, $h_{drainage} < h_{Klip}$, functioning stormwater valves

Scenario: Design system
First, the design situation was modelled, in which no garbage is present. The goal of modelling this scenario is to get an overview of the most flood prone nodes. The results of this scenario are also used as reference data for the other scenarios. The precipitation data from the start of the rainfall event surrounding a flood event until the end of the rainfall event where modelled, in order to allow the system to start up. The amount of flood volume was reported only during the flood event.

Scenario: Garbage in the drainage system
In the analysis in Chapter 4 it was observed that the amount of garbage in the drainage system is approximately 40%. Since it was hard to make an exact estimation, more run, which varying amount of garbage, were done. The amount of blockage is modelled as a percentage (%) decrease of the cross-sectional area. Of course, the estimated value was modelled. The modelled lower boundary was 25%, while the modelled upper boundary was 55%. Since only the relative effect of garbage is investigated, the modelled percentages are accurate enough for the goal. However, it is still important to note the following assumptions. The same percentage is modelled for all pipes and no difference is made between streets with different activities (business or residential areas). In reality, the amount of garbage in streets with many shops is largest. It is assumed that due to the blockage of garbage the pipe diameter is decreased. However, in reality the pipe diameter will not decrease, but garbage will accumulate on the bottom of the pipe resulting in a smaller cross-sectional area. The influence of garbage transported by the flow is not taken into account. Another assumption is that the type of garbage is the same everywhere, so its characteristics such as ability to float are not taken into account.

To get a clear insight into the influence of garbage on the drainage system, first the roughness of concrete (0.012) was taken [31]. However, garbage in the system will always be accompanied by a higher roughness, since the pipes with garbage are less smooth. According to the user manual of SWMM [31], the Manning coefficient of an old, non-smooth pipe is between 0.015 and 0.030. Since the goal is to find the effect of the roughness and it is not possible to find the exact value, in the model, the Manning coefficient is doubled to 0.024. The same Manning coefficient is taken for all the pipes, because of the made assumptions on the type and location of the garbage.

Scenario: Closure stormwater valves
In this scenario, all valves were closed to test if there is an influence of the closure on the capacity of the drainage system. Second, the valves that were probably closed during the flood events were closed in SWMM, based on the results of HEC-RAS. Since information about the control of the sluicegates and the precise functioning of the flapgates is not available, it is assumed that all valves are Tideflex® valves. This assumption is made since precise data of the replacement of the valves is not available, and the functioning of the valves is comparable. So, in combination with the information on the functioning of the Tideflex® valves, the output of HEC-RAS (a list of data and water levels in the Klip River) is used to determine which valves were closed during which floods. Namely, there is no drainage to the Klip River possible if the water level of the drainage system is less than 2.54 cm higher than the water level of the Klip River: $h_{drainage} + 2.54\ cm \leq h_{Klip}$, see Chapter 4. Similar to the HEC-RAS modelling, the event of December 2015 is not modelled, since during this event there was no steady flow.

HEC-RAS produces different water levels for different Manning's coefficients of the Klip River. According to Shallon Pachkowdie and supported by interviews with locals, see Section E.3 and Appendix D, it is most probable that in 2011 backflow of river water into the drainage system occurred. Since the water level in the medium Manning scenario and high Manning scenario is high enough to allow for backflow these values are more realistic. Therefore, the medium Manning scenario with its accompanying water levels are used. The 2.54 cm difference in height is discarded from this point on because it is irrelevant compared to the uncertainties in the water level of the Klip River. Thus, the valves are closed if $h_{Klip} \geq h_{drainage}$. Another assumption that is made is that the flow velocity in the last pipe, between the stormwater valve and last manhole, is zero at the moment that the stormwater valves close. It can be observed from precipitation data and the water level of the Klip River that during the past flood event the water level was high at the moment the rainfall event started. Therefore it can be concluded that the valves close before the rainfall starts. For simplicity, we assume that this also the case for the extreme flood events.

In order to determine whether a valve is closed or open, a local streetlevel to which to compare the water level of the Klip River is defined. This local streetlevel is defined per sub catchment and is the first manhole from the outlet, see Figure 5.5. In case the $h_{Klip} \geq h_{manhole}$ the stormwater valves closes. It opens again when $h_{Klip} < h_{manhole}$. This statement can be elaborated when $h_{drainage}$ is said to be equal to $h_{manhole}$, this can be done under the assumption that water flowing from upstream pipes to the manhole does not add additional water pressure. The extra water will flow up on the streets through the first manhole and flow away to the lowest level on the streets and not stands on the manhole.

The result of the comparison of data was that the following valves close during the flood events:

- January 2011: C13C, C12, C10, C9, C8, C7
- January 2012: C10, C9, C8, C7
- September 2012: C10, C9, C8, C7, C13C

The pipes leading to these valves were closed during the whole modelled period in SWMM.

Future events
Analysis of the precipitation data during the recent flood events showed that the return period of these rainfall events is small. Therefore, it remains interesting to analyse the sensitivity of the drainage system to the factors garbage and closure of the valves during extreme events. For this cause, the hourly 1 : 10, 1 : 50 and 1 : 100 year rainfall events were modelled. Since most probably the stormwater valves will close in the case of the 1 : 50 and 1 : 100 precipitation events, these events were only modelled with closed stormwater valves. The 1 : 10 was also modelled with open valves. There is a large probability that both factors, garbage and closure of the stormwater valves, influence the functioning of the drainage system. Hence, both factors are modelled at the same time. Thus, the goal of modelling the future events is to get an idea of whether the system will response to extreme precipitation events, and to give an indication of the influence of the two factors together on the drainage system during intense precipitation.

5.3.4. Results of sensitivity analysis of the drainage system

For the sensitivity of all scenarios the percentage of internal outflow is compared. Internal outflow is the flow that leaves the system through outflow at non-outfall nodes, in this case the manholes [31]. All the simulated models have a different continuity error, that will result in small differences in total outflow. Due to this small difference in total flow, it is not possible to look at the internal outflow volume. A continuity error is the difference in inflow and outflow of the model. All the errors are between -0.5 % and -3.2 % , so small enough to have a look to the results [31].

Location of floods in SWMM
The main flood events in SWMM, after a rainfall event, are located in the Lyell Street, Murchison Street and the Forbes Street. The cyan blue, green, yellow and red dots in Figure 5.8 indicate the locations where the drainage system is not able to drain the amount of stormwater. The location of the manholes that cannot handle the amount of water does not have to be the location of the floods, since water will flow over the streets from a high elevation level to a low elevation level. In Subsection 4.3.1, Lyell Street and Forbes Street are already mentioned as locations prone to floods. The Murchison Street is not mentioned as a location like this, but this street has a higher elevation level compared to the Lyell Street. Most probably, the water on the Murchison Street will flow towards the Lyell Street.

Figure 5.8 gives an overview of the flood locations during a simulation of the first scenario, the design system. The maximum hourly rainfall in the simulated rainfall event has a low return period of 3.46 *year*, see Subsection 4.3.2. Because of the low return period of the hourly precipitation and the amount of floods in the area, it can be concluded that the design of the drainage system is not sufficient enough. The insufficient design could be caused by to small pipe diameters at inconvenient locations. For example, the water of two drainage pipes are combined in one with a smaller cross-section compared to the two separate ones. Another aspect of the insufficient design is about the location of the manholes. The catchment for some manholes is large, so a large amount of water will flow into the direction of the manhole. This could cause a flooded manhole.

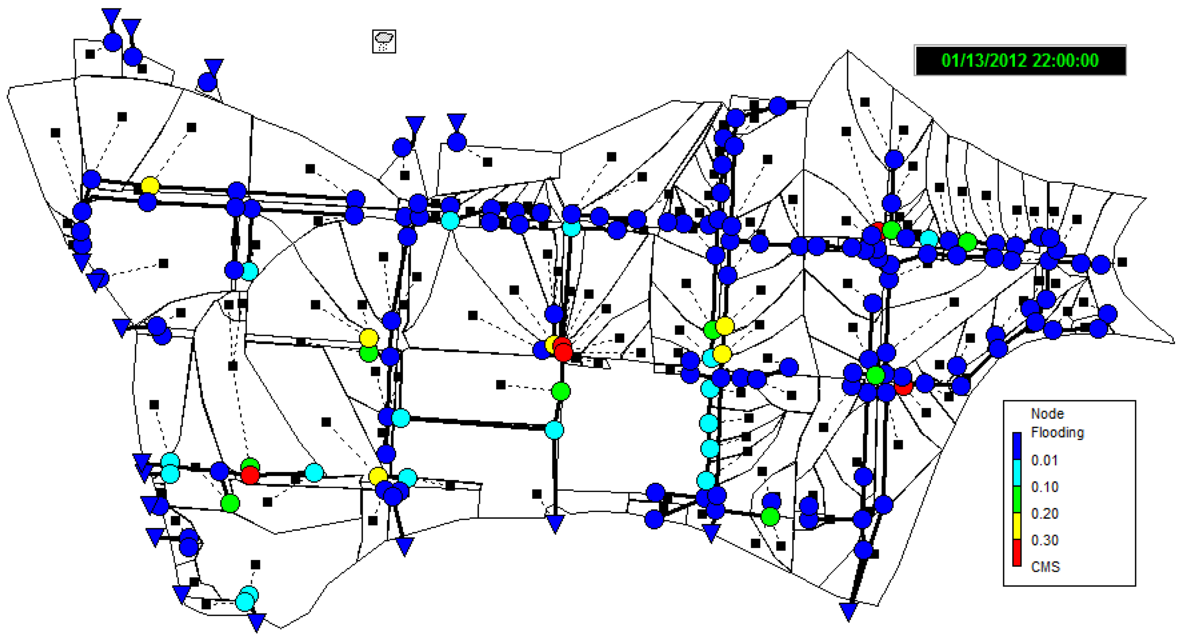


Figure 5.8: Flood locations in SWMM of the flood event January 2012 (scenario 1: design system)

Garbage in drainage system
The sensitivity of garbage in the drainage system is tested by adapting the diameters and the roughness of the drainage pipes. These two factors will be described separately in this paragraph.

The results of decreasing the diameters of the drainage pipes, due to the garbage in it, will increase the flood volume. An overview of the increase in flood volume per flood event and per percentage of blockage is given in Table 5.7. In average compared to the design system, up to a blockage of 25%, per extra 1% blockage the flood volume will increase 1%. Since 25%, 1% extra blockage will increase the flood volume 0.5%. A notable observation, based on the simulations, is about the flood levels per node.

By increasing the blockage of the drainage pipe, almost no increase in flood level per node is observed. In this situation, there is an increase in number of flooded nodes. So, due to blockage of the system, less pipes are able to handle the incoming flow.

Flood event	**Percentage internal outflow in clean system**	**Percentage internal outflow in a 25% blocked system**	**Percentage internal outflow in a 40% blocked system**	**Percentage internal outflow in a 55% blocked system**
January 2011	11	29	36	39
January 2012	37	56	59	64
September 2012	15	34	41	44
December 2015	28	49	53	58

Table 5.7: Results of internal outflow compared to total flow in blocked systems

Doubling the roughness of the pipes due to the garbage in the drainage system will increase the flood volume by 61%. So, the sensitivity of the roughness of pipes is higher compared to decreasing the diameters of the pipes. In real, only a combination of both will appear.

Table 5.8 shows the percentage of internal outflow by combining the blockages and increased roughness. If the roughness of the pipes is doubled due to the garbage in the drainage system, the flood volume will have an extra increase of approximately 7-11%. If there is more garbage in the system, the effects of a change in roughness are smaller compared to a clean system. This effect can be explained by the formula of Darcy-Weisbach.

Not totally full drainage pipe:

$$S = f_D \frac{1}{2g} \frac{V^2}{D} \tag{5.1}$$

A full drainage pipe:

$$S = f_D \frac{8}{\pi^2 g} \frac{Q^2}{D^5} \tag{5.2}$$

S	= head loss	$[m]$
f_d	= Darcy's friction coefficient	$[-]$
g	= gravitational acceleration	$[m/s^2]$
V	= mean flow velocity	$[m/s]$
Q	= discharge	$[m^3/s]$
D	= diameter of the draiange pipes	$[m]$

If there is more garbage in the system, the diameter of the pipes will be smaller. The change of a filled pipe will be higher in this situation. In this case, the diameter in the formula is of the fifth order. The smaller the diameter, the higher the number of filled pipes.

Flood event	**Percentage internal outflow in a 25% blocked system**	**Percentage internal outflow in a 40% blocked system**	**Percentage internal outflow in a 55% blocked system**
January 2011	43	47	51
January 2012	68	69	74
September 2012	48	51	56
December 2015	63	64	69

Table 5.8: Results of internal outflow compared to total flow by an increased roughness

The blockage of the drainage pipes, in the model, is equally divided over the CBD. During the field trip in Ladysmith is observed that the amount of garbage is higher surround the shops compared to the other areas. In the current models, the floods are already in the vicinity of the shops. By taking into account the deviation in amount of garbage, the effects of garbage in the drainage system will be higher.

Closed stormwater valves
Two sensitivity aspects of closing the stormwater valves are tested in SWMM. First all the stormwater valves were closed to test the effect on the drainage system. Then specific stormwater valves were closed based on the results of the HEC-RAS model. The results of internal outflow in both situations is can be found in Table 5.9.

If all stormwater valves are closed, the internal outflow volume is almost tripled. Based on the water levels of the Klip River, the stormwater valves C13C, C10, C9, C8 and C7 will close firstly. The location of these stormwater valves is visible in Appendix B. Those stormwater valves were closed during the flood event of September 2012. If only these stormwater valves are closed, the internal flow volume will only decrease by 15%. In conclusion, the stormwater valves, which have the greatest effect on the flood volume, will close firstly.

Flood event	Percentage internal outflow in design system	Percentage internal outflow in a fully closed system	Percentage internal outflow in a partly closed system
January 2011	11	33	27
January 2012	37	96	72
September 2012	15	39	33

Table 5.9: Results of internal outflow compared to total flow by a closed and partly closed system

Based on fieldwork, Appendix D, it is likely that the stormwater valves do not close properly. This is caused by garbage in the stormwater valves. During a situation in which the stormwater valve has to be closed, the pressure in the Klip River is higher compared to the pressure in the drainage system. Because of the difference in pressure, the water of the Klip River will flow into the drainage system. This will increase internal outflow volume.

Future events
The return period of the rainfall events during the previous floods is small. In the future, there will be more intense rainfall events. The hourly 1 : 10, 1 : 50 and 1 : 100 year rainfall events are used to analyse the sensitivity of the drainage system for extreme events. Table 5.10 shows the percentage of internal outflow during the different extreme events, in the current system.

Current state	Percentage internal flow 1:10	Percentage internal flow 1:50	Percentage internal flow 1:100
25% garbage, open system	65	-	-
25% garbage, closed system	82	97	97
40% garbage, open system	66	-	-
40% garbage, closed system	84	97	97
55% blocked, open system	72	-	-
55% blocked, closed system	86	97	97

Table 5.10: Percentage internal flow compared to total flow during extreme rainfall events

Regarding to the factors of the sensitivity analysis, the drainage system could be improved by cleaning the pipes or moving the stormwater valves. The results of improving the system for extreme situation are visible in Table 5.11. The percentage of decrease means the difference in percentage internal outflow between the current situation and the improved situation. The effects of cleaning the system are smaller compared to relocating the stormwater valves. The stormwater valves have to be relocated in such a way that the pressure in the drainage system will be higher compared to the pressure in the Klip River.

Improvement of system	**Percentage decrease in internal flow 1:10**	**Percentage decrease in internal flow 1:50**	**Percentage decrease in internal flow 1:100**
Clean system compared to a blockage of 25%	10	0	0
Clean system compared to a blockage of 40%	21	0	0
Clean system compared to a blockage of 55%	17	0	0
Open system with a blockage of 25%	21	27	26
Open system with a blockage of 40%	21	26	24
Open system with a blockage of 55%	17	21	20
Open and clean system compared to a blockage of 25%	44	45	42
Open and clean system compared to a blockage of 40%	45	45	42
Open and clean system compared to a blockage of 55%	47	45	42

Table 5.11: Options for improvement of the drainage system

6 Conclusion

The purpose of this study was to find the causes of the recent floods that occured in the Central Business District (CBD) of Ladysmith after the implementation of the Ladysmith Flood Protection Scheme (LFPS) and to asses the flood resilience of the Central Business District.

The causes of the past flood events
The four floods that were analysed are the floods of January 2011, January 2012, September 2012 and December 2015. The floods of January 2012 and December 2015 are characterised by short intense rainfall, whereas the floods of January 2011 and September 2012 are characterised by a high water level of Klip River at the time of the flood. These last two floods were also the most severe floods in terms of amount of water on the streets, damage and obstruction of daily life.

The research done in this study shows that the recent flood events have different causes, mostly related to the drainage system of the CBD. The drainage system consists of both open and closed pipes, and drains to the Klip River. The outfalls are located at a level that is quite often below the water level of the Klip River. In the past, different types of stormwater valves have been used in order to prevent backflow of river water into the drainage system. At the moment Tideflex® valves are used.

The main cause of the recent floods is the state of the drainage system. The drainage system contains a lot of garbage and is broken at several locations. The garbage in the drainage system has a large influence on the emerge of floods. Especially the first 25% of garbage has a large influence.

Another cause is that due to the location and invert level of the outfalls, the drainage system is not always able to drain. It is unknown what type of stormwater valves were used during which flood event and whether the stormwater valves prevented backflow. Assuming that the stormwater valves functioned well, the stormwater valves C13C, C10, C9, C8 and C7 were closed during the flood events of January 2011, January 2012 and September 2012. These are the stormwater valves that drain the largest areas of the CBD. Therefore, their closure has the largest impact on the occurrence of a flood. During the flood event of January 2011 and September 2012 even more stormwater valves were closed. The effect of the closure of stormwater valves increases with an increasing hourly rainfall intensity. However, even with these valves closed, the modelled size of the flood of January 2011 compared to the other floods is smaller than expected. Together with the information obtained from the engineer responsible for the stormwater valves, it can be concluded that backflow occurred during this flood event. Whether backflow did occur during the events in January 2012 and September 2012 remains unknown.

The research showed that the design of the drainage system is probably not able to deal with extreme precipitation. The unequal distribution of manholes over the CBD and the used pipe diameters at certain locations seem to negatively influence the ability of the system to drain and thus could be a cause of the floods. However, whether the quality of the design is a real cause could not be tested by the used model due to a lack of available calibration data.

The water levels in the Klip River have not reached the crest levels. Therefore the water level in the Klip River is not a direct cause of the past flood events.

Conclusion on the flood resilience of the CBD of Ladysmith
The return period of the discharge and precipitation during the previous flood events is small. Since the construction of the Qedusizi Dam, the precipitation and discharges have not exceeded a ten year return period.

Garbage in the drainage system will still be the main cause of a flood that occurs as a result of a 1 : 10 year precipitation and discharge. In this case most stormwater valves will be closed, which also contributes to the occurrence and size of the flood. The water level of the river might be higher than the crest level at a number of locations. However, the model was not calibrated and thus the results should be handled with care.

During extreme events, the most important cause is no longer garbage. During the 1 : 50 and 1 : 100 discharges almost all stormwater valves will be closed. The effect of closing the system is in these cases larger than the effect of garbage. More specifically, in the model there is no difference between a clean system and a system with 55% garbage observed, in the case of 1 : 50 and 1 : 100 precipitation.

According to the results of the used model, the water levels in the Klip River will be higher than the crest levels at multiple locations in the case of 1 : 50 and 1 : 100 year discharges. This implies that the 1 : 100 year flood safety level does not hold. The discharge that flows through Ladysmith with a return period of 100 years seems to be higher than was assumed for the design of the Qedusizi dam. For the design of the Qedusizi dam it was assumed that the 1 in 100 discharge through Ladysmith would be 450 m^3/s. Analysis of the data from two measurement stations upstream of Ladysmith indicated a discharge of almost 650 m^3/s for the 1 in 100 year event. The analysis showed that this difference is due to an underestimation of the discharge of a tributary that joins the Klip River between the Qedusizi Dam and the CBD. Since these extreme discharges are based on a fairly small data record of 18 years, these results should be handled with care.

In summary, the more extreme the precipitation and discharge, the more important the effects of the location and elevation of the outfalls, the functioning of the stormwater valves and the height of the levees are.

At the moment all stormwater valves are Tideflex® valves. Research showed that the current state of the Tideflex® valves may prevent the valves from functioning correctly. If the state of the stormwater valves is not improved, backwater flow can occur again in the future, increasing the probability and the severity of floods.

7 Recommendations

To prevent the CBD of Ladysmith for flood events like the ones in the past years, and to reduce the effects of extreme events, it is advisable to do adaptions and more research. Adaptions have to be done to the river system, drainage system and the methods of measuring. Furthermore, extra research and measurements have to be done to provide better insight on the floods. This chapter will elaborate on the advisable actions that should be taken.

7.1. Maintenance of drainage system

The floods of recent years are mainly caused by inadequate maintenance of the drainage system. Specific actions have to be taken to ensure the functioning of the drainage system. The following actions should be considered:

- **Review of the current drainage system**
  In the current design of the drainage system floods occur due to high precipitation events with small return periods. A review should be done on the current system to indicate which components are subject to improvement to decrease the probability on a flood event. For example: expanding the drainage system, so the catchment area per manhole will decrease, or increasing certain pipe diameters. The main purpose of a review would be to expand the capacity of the drainage system.

- **Cleaning the drainage system**
  The current system is blocked by garbage and debris in the pipes, manholes and chambers. This also influences the roughness of the pipes. Cleaning the drainage system on a regular basis ensures that the drainage system performs according to its design, making it more resilient to extreme precipitation. The effect per added percent of blockage by garbage is largest for blockage values below 25%, indicating that a frequent cleanup of the drainage system is necessary for adequate performance.

- **Re-installment of a number of stormwater valves**
  A number of the stormwater valves are installed incorrectly, damaged or held open by rubbish, causing them to not close correctly. Those stormwater valves are expected to cause backflow in times of a high water level in the Klip River. During the re-installment of the stormwater valves, it has to be made sure that the stormwater valves are able to close completely.

- **Diverting a part of the drainage system**
  If the water level of the Klip River is higher than the water level in the drainage system, the stormwater valves are closed and the drainage system is unable to drain into the river. Some of the stormwater valves will close due to a high water level of the Klip River with a relatively small return period. The outfalls of those stormwater valves have to be diverted to make sure that they are able to drain. There are two options for diverting:

  1. Increase the invert level of those outfalls.

2. Drain the water to another part in the river, with lower water levels.

- **Installing pumps behind the stormwater valves**
As described in the action above, the stormwater valves will close due to a pressure difference. When this occurs the drainage system is not able to drain the stormwater into the Klip River. The stormwater will be stored in the drainage system. If the rainfall event continues, this could lead to a flood event. To make sure that the pressure in the drainage system is always higher than the pressure caused by the water level of the Klip River, pumps should be installed. The pumps are able to increase the water pressure of the drainage system. Priority is given to the installment of pumps behind stormwater valves C9, C8 and C7. Those stormwater valves close first and are affecting the emergence of a flood most.

7.2. Maintenance of river system

The following actions are recommended to improve the maintenance of the river system:

- **Improvement of maintenance scheme**
Maintenance of the river section is part of the LFPS. Maintenance of the river section is more effective just before the start of the rainy season, instead of afterwards.

- **Removal of the garbage in the river section**
Besides pollution, garbage in the river system increases the roughness of the river bed, resulting in a higher Manning coefficient. A higher Manning coefficient results in a higher water level, which will increase the probability of the occurrence of a flood event.

- **Removal of the trees in the river section**
At several locations, trees are stuck on structures in the river section. Those trees slightly decrease the cross-sectional area. A decrease of the cross-sectional area can contribute to a higher water level locally and upstream due to backwater effects.

7.3. Monitoring within Klip River catchment

An essential input in the research regarding the Klip River catchment is the availability and reliability of data. For the flood problem in Ladysmith, discharge data and hydrological data is of importance. Although there are quite some measurement stations within the Klip River catchment, there is space for improvement on the following:

- **Verifying the Q-H rating curve of V1H038**
The discharge measurement station V1H038 is located near Ladysmith's CBD and therefore of importance for the research. However, the current Q-H rating curve of V1H038 was determined in 1971. Due to morphological changes of the river it is very likely the Q-H relation has changed. A verification of the Q-H relation of this measurement station is advisable in order to get more reliable data.

- **Installing discharge measurement stations in the tributaries of the Klip River**
The tributaries of the Klip River, both upstream and downstream the CBD, will influence the water level of the Klip River along the CBD. The upstream tributaries will increase the total discharge along the CBD and the downstream tributaries will influence the water level due to backwater effects. Installing additional measurement stations in these tributaries will provide a better insight in the water levels along the CBD.

- **Installing discharge measurement station in the Klip River near Ladysmith's CBD**
Currently the only two measurement stations are located near Ladysmith CBD, namely V1H038 and V1R005. In order to get a better insight in the water levels along the CBD an additional discharge measurement station is advisable. In addition, this will be useful for the calibration of the HEC-RAS model in order make the correct conclusions of the model.

- **Placing a rainfall measurement station in Ladysmith's CBD**
At the moment there is only one measurement station for precipitation in the surroundings of Ladysmith. This station is located near the Ladysith Aerodrome, approximately three kilometers from the

CBD. Because precipitation events, in particular extremes, can have a very local character a measurement station near Ladysmith's CBD is recommended.

- **Verifying the daily threshold of interception**
The daily threshold is a factor used for determining the interception. The current used value for the daily threshold is determined in 1973, based on the characteristics of that time. This research was done by Pitman [16], however details of this research are unknown. Most probably the characteristics have changed over the years. In order to verify or change the daily threshold, research about the soil profile and vegetation in the area is recommended.

7.4. Floodline monitoring

Floodline monitoring is done to get knowledge on which discharge overtops the levees and to determine which areas are flooded during extreme events. For an exact monitoring of the floodlines, extensive data is needed. The current available data is not sufficient to determine the floodlines precisely. The actions mentioned below are necessary to generate more accurate floodlines.

- **More complete cross-section monitoring**
In the current monitoring surveys the cross-sections are measured till the top of the levees along the Klip River. However, for some cross-sections not the complete width of the river including floodplains is monitored. Those areas should be taken into account in the future monitoring surveys. In the currently used HEC-RAS model, incomplete cross-sections result in unrealistic vertically standing water bodies on the edges.
- **A better investigation of the Manning coefficients in the river**
During the monitoring survey photographs around the river cross-sections are taken. Based on these photographs Manning coefficients are determined. The range in Manning's coefficients used by Royal HaskoningDHV for the river system in the latest HEC-RAS model is small. This range should be increased, as the channel roughness varies significantly along the Klip River. A large difference is observed between the reaches up- and downstream of the knickpoint.
- **Monitoring extra cross-sections during the monitoring survey**
In the current monitoring survey many cross-sections around Ladysmith's CBD are measured. To get a better insight in the water level, more cross-sections downstream should be included in the monitoring survey. Especially in the river bends, since here the change in geometry is largest.
- **Include the pedestrian bridges in the monitoring survey**
There are three pedestrian bridges crossing the Klip River over the investigated river reach that decrease the cross-sectional area of the river and influences the water level in the Klip River. Monitoring of the pedestrian bridges has to be done and subsequently modelled in HEC-RAS.
- **Include monitoring of the height and condition of levees in the monitoring survey**
The height and condition of the levees is a relevant aspect of the monitoring surveys. The height of the levees is important to know, since it is an important element of the cross-sectional area of the Klip River. The condition is interesting to know, because a levee in a bad condition is prone to failure.
- **Generation of a detailed elevation map** A detailed digital elevation map is essential to set up a 2D floodline model. A 2D floodline model can give a better insight on the impact of possible future flood events. Furthermore, a detailed elevation map could be used to indicate the areas in Ladysmith that flood when levees break or are overtopped.

7.5. Additional research

Based on the results of the analysis several factors that probably influence the occurrence of floods were identified. For some of these factors there is insufficient data available to determine their influence. Therefore additional research is needed for the following factors:

- **The sedimentation behind the Qedusizi dam**
The sedimentation of the reservoir behind the Qedusizi Dam might negatively influence the perfor-

mance of the Qedusizi Dam. The sedimentation process was taken into account in the design of the dam. However, the type and rate of sedimentation is difficult to forecast. Therefore, inspection of the current sedimentation and whether this is in agreement with the design should be checked.

- **The effects of vegetation management**
  In the LFPS maintenance of the river channel is adopted with among others the removal of trees and vegetation management. The vegetation management was observed during the field trip in the form of burned river banks. Currently the effectiveness and possible negative effects of this vegetation management is unknown and additional research is recommended.

A

Measurement stations

A.1. Flow measurement stations

For this study, the data of three flow measurement stations was used to gain insight on the discharge of the Klip River. Those measurement stations, V1R005, V1H038 and V1H051 measure the water height at different sections along the Klip River, prior and after the construction of the Qedusizi Dam. An overview of the locations of the measuring stations is given in Figure A.1, which can also be found in Chapter 2 of the main report. The data from the stations is publicly accessible on the website of the DWS.

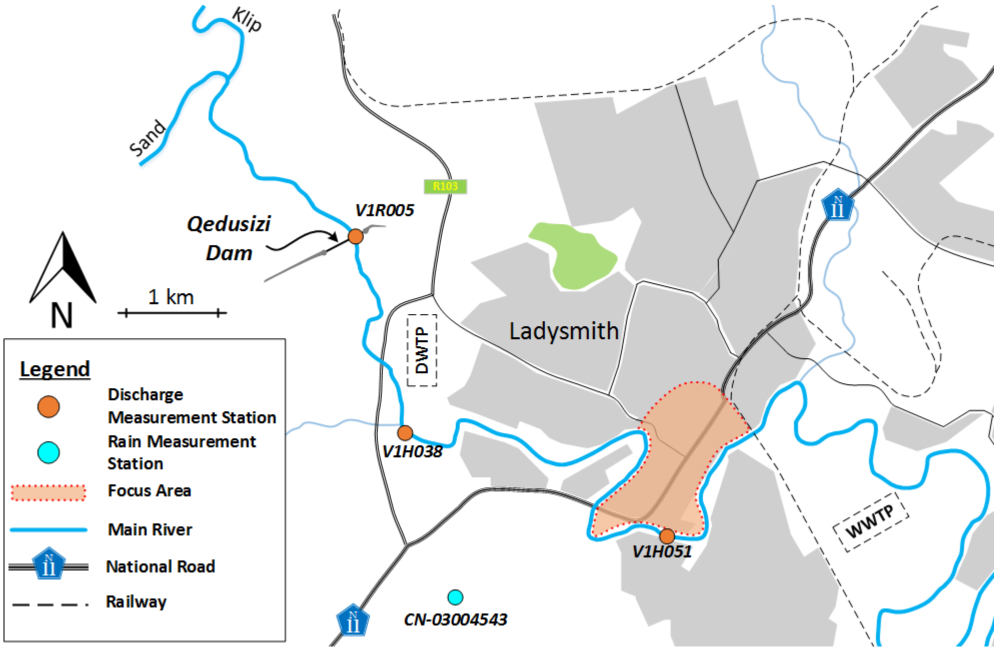


Figure A.1: Ladysmith overview, showing the focus area, measurement stations and watertreatment works

A.1.1. V1R005

Station V1R005 located at the Qedusizi Dam measures the water pressure above the bottom inlet structure. From this data the monthly spill volume (m^3), the daily average flow (m^3/s) and instantaneous flow (m^3/s) are automatically calculated using a rating table. Measurements are taken since September 1997 to present day with intervals of 12 minutes.

A rating table is used to obtain the discharge from the measured water pressure. The rating curve for the discharge through the bottom outlets was determined in 1997 and has not been adjusted since. For simplicity the rating table was converted to Figure A.2 and Figure A.3 below. The influence of the design of the Qedusizi Dam on the water level and discharge relation can be seen in two ways. First of all, Figure A.2 shows a steeper curve for water levels larger than 3.2 meter. This can be explained by the dimensions of the bottom outlet which have a height of 3.2 meter. Secondly, at a water level of 20.5 meter Figure A.3 shows a rapid increase in discharge. At this water level, the maximum capacity of Qedusizi Dam reservoir is reached and the water will top over via the dam's spillway.

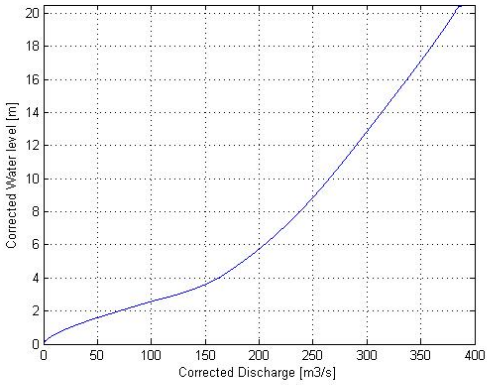


Figure A.2: Q-h relation small discharge

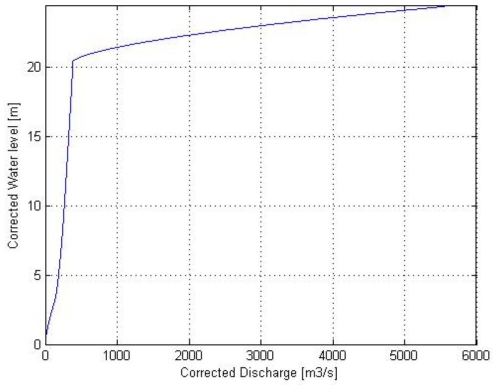


Figure A.3: Q-h relation large discharge

A.1.2. V1H038

A second flow measurement station (V1H038) is located approximately 2.5 kilometers downstream of the Qedusizi Dam. The measurement station consists of a concrete weir structure covering the entire width of the Klip River. In the middle of the structure a spillway is placed through which water is able to flow. Furthermore, water can be stored behind the structure to a height of maximum 1 meter. As well as V1R005, station V1H038 contains a pressure gauge from which the monthly spill volume (m^3), the daily average flow (m^3/s) and instantaneous flow (m^3/s) are calculated. Figure A.4 shows an impression of station V1H038.

Figure A.4: Station V1H038

A few remarks should be made on the design and generation of flow data of station V1H038. First of all, the pressure gauge is placed a couple of metres upstream of the concrete structure. At this location the river cross-section is influenced by aggradation and degradation, and thus varying in time. The variation of the cross-section has not been taken into account, since the rating table that was established in 1971 is still in use (Figure A.5). A second remark can be made on the maintenance of station V1H038. During field work of the project group (September 2016) a dead tree trunk was found on top of the structure. The latest significant high water however was measured in February 2016. In conclusion it can be said that the data of flow measurement station V1H038 should be used with caution. The measured water level will be more reliable than the calculated spill volume or discharge.

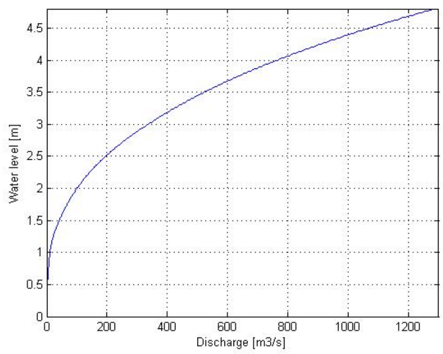


Figure A.5: Q-h relation

A.1.3. V1H051

Station V1H051 is located near the bridge of the Soofi Mosque in Ladysmith and is currently inactive. Data is available from September 1987 till November 1993. Measurements from station V1H051 were used to check hand calculations on the water level change upstream and downstream of the knickpoint.

A.2. Rainfall and evaporation measurement stations

Several rainfall and evaporation measurement stations were used to determine the run off in the catchment area and Ladysmith. This section informs about the location of the stations and the available data.

A.2.1. Catchment

Station V1E005 at Van Reenen measures the precipitation and evaporation in the Drakesbergen. It is located close to the origin of the Klip River. This station was used to get insight on the run-off in the catchment upstream of the Ladysmith. Data is available from 1969 till 2014.

A.2.2. Ladysmith

There is one rainfall measurement station that can be found in Ladysmith, nameley CN-03004543. The automatic weather station is located near the Ladysmith Aerodrome Airfield, approximately 3 kilometres southwest of the CBD. The weather station is equipped with a tipping bucket rain gauge, which is calibrated to register rainfall of 0.2 mm of rain per tip. The weather station is owned by the South African Weather Service (SAWS). Precipitation is measured from July 1993 till present day.

Since the data set of station CN-03004543 is not large enough to determine any possible trends in precipitation and evaporation, other nearby measurement stations are used. Calculations with the inverse distance method is done on the data of 10 measurement stations, that have measured for a longer time period. More information on these calculations can be found in Section F.1 The measurement stations are listed in Table A.1, together with their location and the period in which the data is measured.

Station	Location	Available Rainfall Data	Available Evaporation Data
V1E001	Colenso	1935-05-13 1990-09-30	1935-05-03 1990-01-01
V1E002	Cathedral Peak	1949-01-31 1997-02-28	1948-12-31 1997-03-01
V1E006	Rhenosterfontein at Spioenkop Dam	1969-08-01 1995-04-30	1969-08-01 1995-05-01
V1E007	Jagersrust at Pump Station	1976-02-22 1997-03-01	1976-02-02 1997-03-01
V1E010	Rhenosterfontein at Spioenkop Dam	1992-03-31 2010-10-01	1992-04-02 2010-10-01
V2E002	Rietvlei at Craigie Burn Dam	1969-09-30 2010-10-01	1969-09-30 2010-10-01
V2E003	Mooi River at Mearns	2003-11-30 2010-10-01	2003-12-01 2010-10-01
V3E002	Chelmsford at Chelmsford Dam	1964-09-16 2010-10-01	1964-09-02 2010-10-01
V7E001	Estcourt	1954-12-04 1973-03-01	1954-11-30 1973-03-01
V7E003	Wagendrift at Wagendrift Dam	1973-03-01 2009-11-01	1973-11-02 2009-11-01

Table A.1: Rainfall and evaporation in the catchment

B

Reports Royal HaskoningDHV

In this appendix a short summary is given of the relevant reports prepared by Royal HaskoningDHV for the Municipality of Ladysmith. The reports are classified on the subjects stormwater valves and floodline monitoring.
Some of these reports are written several decades ago, by the former company SSI and SSI Engineers and Environmental Consultants. SSI Engineers and Environmental Consultants was one of the first independent consulting engineering companies in Southern Africa and was founded in 1922. In September 2006 the DHV Group became the majority shareholder of the company, and the company changed its name to SSI. In 2012 the company changed its name again due to the merger between DHV and Royal Haskoning.

B.1. Stormwater valves

In total three reports from Royal HaskoningDHV regarding the stormwatervalves in Ladysmith were used. Figure B.1 gives an overview of the locations of the valves in Ladysmith. The three reports are used to compile Table B.1 shows which type of valves are installed on each chamber per year of inspection. A description of the separate reports is given below.

Prepared by:	SSI Engineers and Environmental Consultants
Report:	The investigation of using Tideflex check valve as an alternative to retaining and maintaining the current conventional sluice/flap gates
Year:	July 1997
Prepared for:	Emnambithi-Ladysmith Municipality

The report is a proposal for the Municipality to install Tideflex® valves instead of the sluice and flap gates installed at that time. The following conclusions are of importance:

- The sluice and flap at most of the outlets do not operate satisfactorily and require maintenance and repairs or replacement.
- Installing Tideflex® valves has numerous advantages compared to the current sluice and flap gates.
- Per outlet the costs for supply and installation of Tideflex® valves are shown (see Table Table B.1).
- Because of the high initial costs, SSI proposes to install the valves over an extended time period.

Prepared by:	SSI
Report:	Flood Control Valves, chambers and levees along the Klip River through Ladysmith
Year:	February 2011
Prepared for:	Emnambithi-Ladysmith Municipality

The report shows the results of an inspection of the different valves and chambers done in October 2010, the following conclusions are of importance:

- Of the 24 flood chambers inspected, 14 were Tideflex® valves, 6 have flap gates and 4 have sluice gates (see Table Table B.1).
- Most of the sluice and flap gates are in bad state and need service or replacement by Tideflex® valves.
- In the recommendation more regular intervals of inspections and servicing of flood control chambers is advised.

Prepared by:	Royal HaskoningDHV
Report:	Flood Control Tideflex valves along the Klip River through Ladysmith
Year:	September 2014
Prepared for:	Emnambithi-Ladysmith Municipality

The report shows the results of an inspection of the Tideflex® valves done in March 2014, the following conclusions are of importance:

- Of the 22 chambers inspected, all of them were fitted with Tideflex® valves (see Table Table B.1).
- In the recommendation more regular maintenance with removal of debris and silt is advised, see Figure B.2c
- An additional conclusion is that some Tideflex® valves are gaping open, of which some deliberately kept open. installed incorrectly and some should be replaced, see Figure B.2a and Figure B.2b.

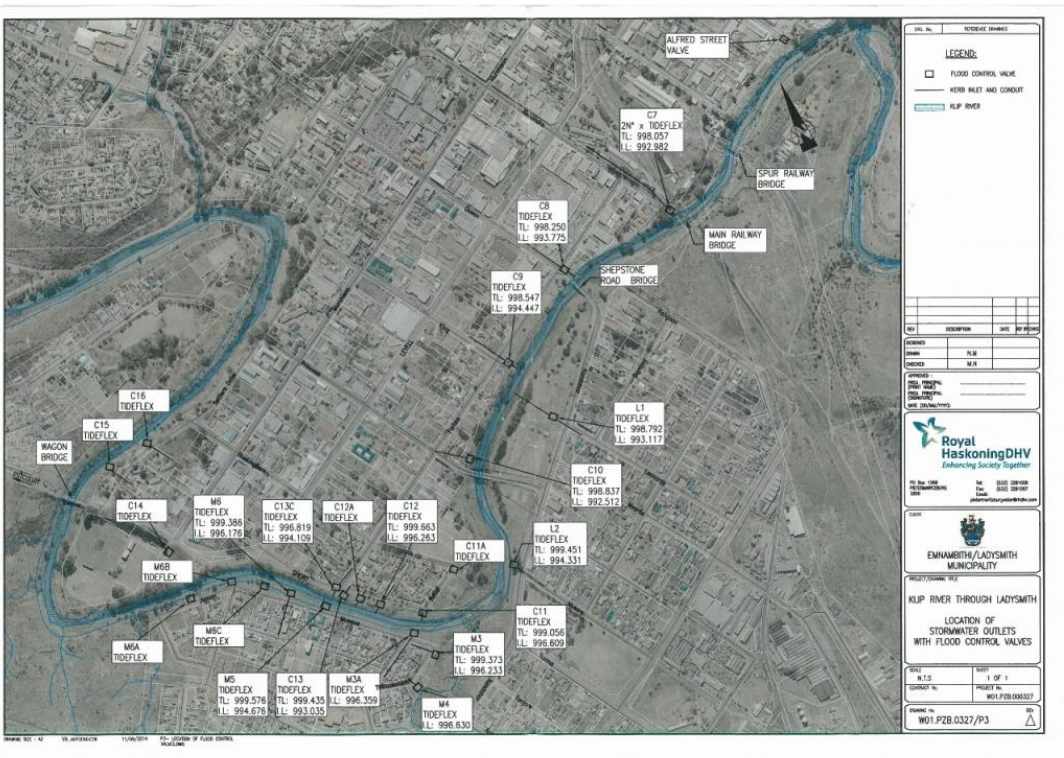


Figure B.1: Overview of the valves inspected for the report of September 2014

(a) Gaping Tideflex®

(b) Not optimal Tideflex® installation

(c) Debris in Tideflex®

Figure B.2: State of some Tideflex® valves, from 2014 report by Royal HaskoningDHV

Valve Number	Valve type in July 1997	Valve type in Oct. 2010	Valve type in March 2014	Cost Tideflex in ZAR (1997)
C7	Rectangular Opening	Tideflex®	Tideflex®	413,140
C8	Sluice gate	Sluice gate	Tideflex®	445,200
C9	Sluice gate	Sluice gate	Tideflex®	280,200
C10	Flap gate	Flap gate	Tideflex®	408,140
C11	Flap gate	Tideflex®	Tideflex®	16,434
C11A	Flap gate	Tideflex®	Tideflex®	41,304
C12	Flap gate	Tideflex®	Tideflex®	16,434
C12A	Flap gate	Tideflex®	Tideflex®	16,434
C13	Flap gate	Tideflex®	Tideflex®	37,112
C13C	Flap gate	Tideflex®	Tideflex®	42,304
C13A		Tideflex®	unknown	
C14		Flap gate	Tideflex®	
C15		Flap gate	Tideflex®	
C16		Flap gate	Tideflex®	
L1	Sluice gate	Sluice gate	Tideflex®	408,140
L2	Sluice gate	Sluice gate	Tideflex®	408,140
M3	Flap gate	Tideflex®	Tideflex®	37,112
M3A	Flap gate	Tideflex®	Tideflex®	37,112
M4	Flap gate	Tideflex®	Tideflex®	41,304
M5	Flap gate	Tideflex®	Tideflex®	41,304
M6	Flap gate	Tideflex®	Tideflex®	37,112
M6A	Flap gate	Tideflex®	Tideflex®	20,387
M6B	Flap gate	Tideflex®	Tideflex®	37,112

Table B.1: Type of valve installed on each chamber per year of inspection. The last column shows the total costs for installing a Tideflex® valve, which is estimated in 1997

B.2. Floodline monitoring

Since the construction of the Qedusizi Dam in 1998, a multi-annual survey is executed by Royal HaskoningDHV to monitor the floodlines and analyse backwater curves in the Municipality of Ladysmith. The survey, commissioned by the municipality, is held every 3 to 4 years and is executed by analyzing 31 cross-sections over a 24 kilometer reach starting just downstream of the Qedusizi Dam to a point 14.5 kilometer downstream of the CBD, see Figure B.3. Included in this survey are 5 bridge sections along the reach. The survey was repeated in 2003, 2006, 2009, 2013 and 2016. With the acquired data of the cross-sections a HEC-RAS model is used to obtain estimations of the floodlines in Ladysmith

Prepared by:	Royal HaskoningDHV
Report:	Klip River floodline monitoring 2013 survey
Year:	September 2014
Prepared for:	Emnambithi-Ladysmith Municipality

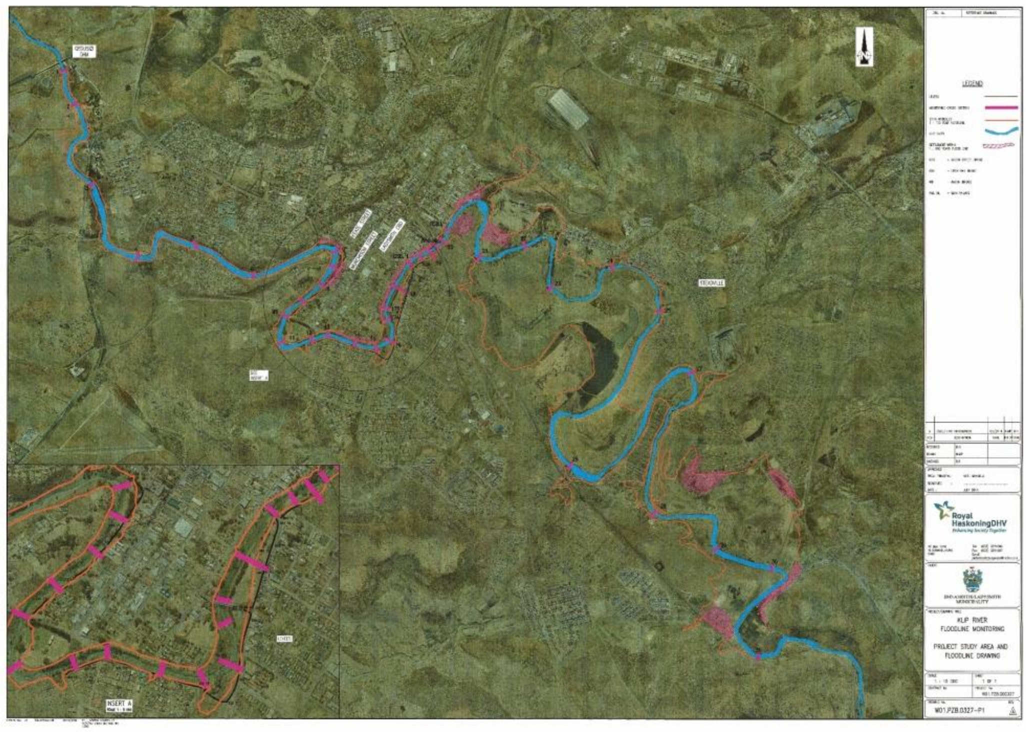


Figure B.3: Cross-sections of the floodline monitoring and backwater analysis

Figure B.4: Insert A from Figure B.3

From the report on the survey of 2013 the following conclusions are of importance:

- During a survey the vegetation, levees and cross-section are monitored.
- The vegetation is of importance for determining the channel roughness, which is monitored per survey. From the report it was shown that the period of the survey per year was not done consistently, some are during rain season while others were not. In the report the worst expected condition is used i.e. lush vegetation growth along the river. Different Manning values are used for the main channel and flood plain.
- The cross-sectional area of the Klip River is monitored at 31 locations. For this monitoring the elevation and shape of the cross-section are of importance. Compared to the survey done in 2009 at some cross-sections natural changes in river geometry occurred due to sedimentation and deposition. But over the total period since the construction of the dam, no permanent pattern of geometry change is observed.
- The 1:100 floodline monitoring shows that at some location the levees are not of sufficient height.

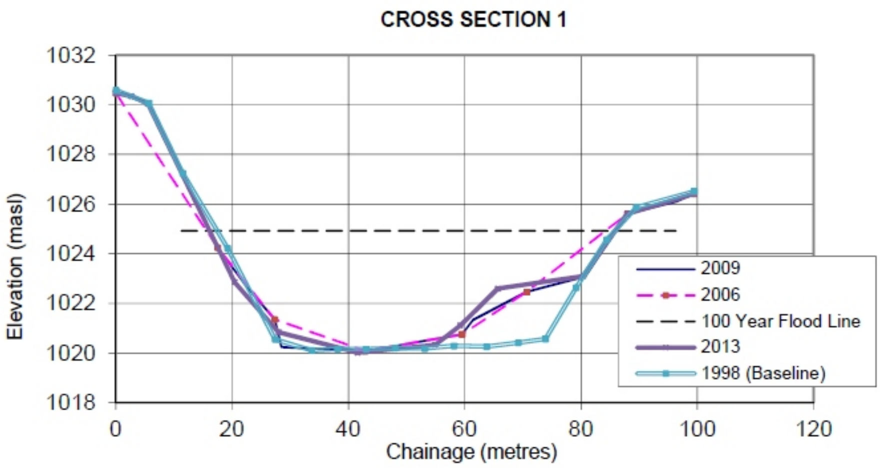


Figure B.5: Example of the cross-section survey

C
News reports

In this appendix different sources are used to create an overview of all flood related events that occurred in Ladysmith since the construction of the Qedusizi Dam. The amount and type of sources differ per event, depending on the impact of the event. In the overview the events are shown in chronological order starting from 1998. Whether the flood events occurred in the focus area, as described in 2.1, is mentioned below as well.

The type of sources that are used for detecting the flood related events are (online) news reports, social media and Youtube videos. It should be noted that these sources are not always reliable and that they should be read critically. Furthermore, it should be noted that the online archive of the Ladysmith Gazette is uploaded since 2013. Therefore an increase in reports per year does not indicate an increase in flood related events. An increase in reports might be due to online availability as well.

Unfortunately, no detailed reports of the events were obtained by contacting the Disaster Management of the Ladysmith Municipality or Uthukela District Municipality. In a phone conversation (October 2016) the disaster manager of Ladysmith explained that such flood events are not recorded, because they are not considered as disasters.

2000, March
In the summer of 2000 Ladysmith experienced multiple high water events. According to measurements the town would have flooded 6 times in the summer of 2000 without the construction of the Qedusizi dam [53] [59].

2001, 19-22 December
The source of the flood event reports about an hour long storm accompanied with a flash flood on Wednesday night, 19 December. It is unknown if the Ladysmith CBD has been affected during the flood [10].

2011, 4-5 January
This flood event has been mentioned in numerous reports because of the large damages in KZN and in the municipality of Ladysmith. From the reports it is often difficult to make a distinction between the floods in the CBD of Ladysmith and the municipality as a whole.

On the internet page of the South African Weather and Disaster Observation Service a flood warning was given on the morning of the 4th of January for the Ladysmith area of KZN [71]. The warning mentioned that a flood of the low-lying lands and roads is reality and that flooding of homes, streets and businesses is expected. The warning concludes with actions that can be taken to stay save.

Different sources mention that in the afternoon multiple locations within the municipality are hit by floods [64] [70]. Hundreds of houses were flooded in the rural areas of the towns Roosboom, Driefontein and Ladysmith. The central business district of Ladysmith is mentioned as well. The CBD was brought to a standstill because the Lyell Street(N11) was full of water. Businesses adjacent to the road were badly affected because of the floods and some shops were forced to close. In several reports it is mentioned that the Klip River did not burst it banks, the blocked stormwater drains around town caused the flood.

In the days after the floods news reports mention that the people living on the river banks of the Klip River in Ladysmith will be evacuated and the houses will be demolished [65] [66]. Other news reports mention the lack of drinking water in Ladysmith after the flood [50]. The pumps at the drinking water station were submerged, causing them to get clogged with sand and silt.

(a) Flood junction Lyell Street and Queen Street

(b) Flood junction Lyell Street and King Street

(c) Flood Lyell Street

(d) Aerial photo Lyell Street

Figure C.1: Floods January 2011

2012, 13 January
On the 13th of January 2012 the Ladysmith Gazette, a local newspaper, mentioned a large flood on their Facebook page. The report specifically mentioned: "Stormwater flooding", mentioning the probable cause of the flooding. The multiple reactions from residents in response of the message give an indication of the scale and locations of the flood [45]. The Murchison street is mentioned as well as Acaciaville, a suburb south of the Ladymith CBD. On Twitter a resident complains about the lack of drinking water [56]. Apparently the flood affected the drinking water facility in Ladysmith.

2012, 6-8 September
Although the pretty large extent of the flood event from 6 to 8 September 2012, only a single report has been found [72]. The news report mentioned that the Klip River was running high and there was continuous rainfall on 6 and 7 September. Parts of Lyell Street were half a metre under water. The report specifically mentioned the town's stormwater drainage system as the main cause. On Twitter a resident posted an image of the flooded Forbes Street. Figure C.2a, shows the flood on this location [69]. A Youtube video taken by an editor of the Ladysmith Gazette on the junction of the Alexander Street and Lyell Street shows the problems of the flood at this location [57].

(a) Flood junction Forbes Street and Queen Street

(b) Flood junction Lyell Street and Alexander Street

Figure C.2: Floods September 2012

2012, 10 December
A severe hail storm in combination with local floodings damages more than 500 houses in the Municipality of Ladysmith [52]. The leading insurer, Mutual & Federal, has estimated the damage at more than 2.3 million Rand [67].

2014, 17 March
A heavy storm caused vehicles to get stuck on the Main Road in Acaciavale, a suburb south of Ladysmith. The Ladysmith CBD is not mentioned in the report [54].

2014, 8 December
On Monday 8 December local heavy rain and hail fell in the Ladysmith area, causing flash flooding and road closures. Especially low-lying bridges were hit by the flash floods. The news report mentions the Burma Road and Protea Drive, however the CBD is not mentioned [48].

2015, 3-4 December
On Thursday 3 December a storm with heavy rainfall left parts of the Ladysmith CBD flooded. Several vehicles got stuck in the heavy downpour and some homes and yards were flooded. Different sources claim that the downpour happened between 4 : 30 pm and 6 : 00 pm [40] [58] with an unofficial recorded 100 milimeter within 2 hours in the CBD [58]. Another source mentioned 33 milimeter [68]. Additionally, the reports also mention that the blocked stormwater drains exacerbated the problem while others claim that they are the cause of the problem.

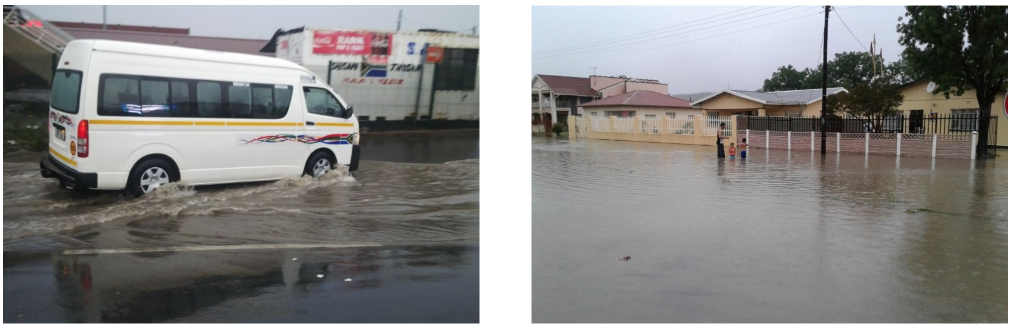


Figure C.3: Floods December 2015

2016, 23 January
Heavy rainfall in the afternoon of Saterday the 23rd of January caused a local flood at the Kandahar Avenue. The water height (approximately knee height) caused some vehicles to get stuck. The Ladysmith CBD is not mentioned in the report[43].

Figure C.4: Floods January 2016

D

Field observations

In total four days, divided over two visits, were spend in Ladysmith and its surroundings in order to investigate the river and drainage system of the town. Both visits took place in September 2016. In this appendix the river system and drainage system are discussed separately.

D.1. River system

In this section the main observations along the Klip River are stated and included with images to give a clear view of the project group findings. It should be kept in mind that the focus area as defined in section 2.1 was the main area of interest during the visits.

D.1.1. Global river characteristics

A clear difference is observed between the characteristics of the Klip River upstream and downstream of the knickpoint. This observation is supported with photographs taken during the field study in September 2016. The location and direction of the photographs taken are represented by the yellow arrows in Figure D.1. The approximate location of the knickpoint is highlighted by the blue circle.

Figure D.2 shows a compilation of photographs taken from the Klip River upstream of the knickpoint. This part of the river is characterized by large irregularities in shape, cross-section, sediment and vegetation. A clear riffle-pool sequence is observed. Shallow areas with accumulated boulders, relatively steep slope and high flow are alternated with deep streams, fine sediment, hardly no slope and low flow. The river banks are varying between wide floodplains and steep rock slopes.

The boulders found in the river become finer more downstream. The last riffle of the focus reach is found in the bend where the knickpoint is located, shown in Figure D.3. There the riffle and pool sequence stops. Figure D.4 shows the photographs taken from the Klip River downstream of the knickpoint. The photographs show a clear difference compared to the upstream reach. From here the floodplains widen and flatten. The river reach is now characterized by a clear main channel with large even floodplains on both sides.

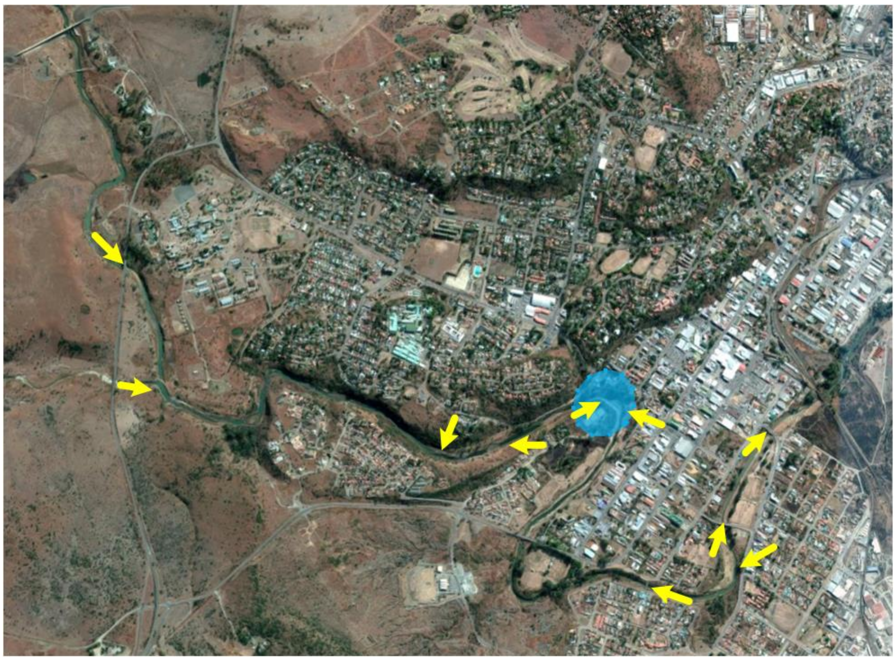


Figure D.1: Location of photographs upstream, downstream and around knickpoint

(a) Downstream view from R103 Bridge

(b) V1H038

(c) Rock carved riverbend

(d) Riffle and pools

Figure D.2: Upstream of knickpoint

(a) River bend at approximate location of knickpoint, eastward view

(b) River bend at approximate location of knickpoint, westward view

Figure D.3: Knickpoint in river bend

(a) Upstream view from Mosque Bridge

(b) South-east river bend

(c) Downstream view from Princess Street Bridge

(d) Downstream view from Shepstone Bridge

Figure D.4: Downstream of knickpoint

D.1.2. Vegetation and garbage

During the field observations, it was observed that there are large differences in the amount and type of vegetation on the floodplains along different sections of the river. In some parts there is hardly any vegetation except for some grass and a few small bushes. In other sections however, vegetation is abundant and consists of high grass, bushes and trees. At some locations along the river the results of recent maintenance could be observed: trees were cut and vegetation was burned down. An example of this can be seen in Figure D.5a. Based on a project description sign of the municipality, shown in Figure D.5b found on the banks of the Klip River, it is assumed that the vegetation is burned as part of a flood management program. However, as of yet, this cannot be confirmed, nor is it known with what frequency the vegetation is removed and to what extend. Furthermore, during the field observations it was concluded that at a lot of locations, the floodplains contained a lot of garbage. This can locally lead to a higher hydrological roughness.

(a) Burned river banks

(b) Flood management sign

Figure D.5: Signs of maintanance along Klip River

D.1.3. Structures

Upstream and along the CBD, the Klip River is crossed by multiple bridges of varying dimensions. The location of these bridges was already identified from maps of the town. However, during observations three additional pedestrian bridges were observed in the focus reach. Two of these bridges are relatively low and cannot be used during high water. The pedestrian bridge near the Soofie Masjid Mosque is high enough to be used during high water. Against one of the low pedestrian bridges a tree trunk was found, as can be seen in Figure D.6a. Another tree was found on top of the weir near measurement station V1H038, see Figure D.6b. Both the tree trunk at the pedestrian bridge and on the weir give the impression that certain maintenance along the Klip River lacks.

(a) Tree against pedestrian bridge

(b) Weir V1H038 including tree

Figure D.6: Lacking maintenance along the Klip River

Another type of structures in the riverbed and floodplains along the CBD were components of the drainage system. These structures mainly consisted of drainage outlets, pumps and chambers around stormwater valves, see Figure D.7. At multiple locations high amounts of garbage were present and erosion was clearly visible in and around these structures.

Figure D.7: Drainage outlet in Klip River

D.1.4. Levees

The levees around the CBD of Ladysmith, as indicated in Royal HaskoningDHV report, have been observed by the project group [29]. From field observations it was concluded that at some location levees were absent, despite their indication in the report. Furthermore, it was noticed that the elevation of the levees differs along the Klip River and that maintenance was lacking.

Figure D.8: Levees at one location along the CBD

D.2. Drainage system

The stormwater valves that prevent backwater flow into the town of Ladysmith were observed at multiple locations along the CBD. Here also the amount of garbage in and around the valves was remarkable. At some locations small plastic bottles even prevented the valves from closing completely. Within the town multiple locations of the drainage system were visited. Both open drainage and closed drainage systems could be observed. These observations showed how the gaps of the drainage map must be filled. They confirmed the layout of the drainage system: a mixture of open and closed pipes of various sizes and forms, that are located at the sides of the streets just below the street level. The manholes have the same diameter as the pipes, since they are often not more than a hole in the pipe. From these observations it can also be concluded that the drainage system is often in bad state with broken pipes and manholes. A high amount of garbage and debris in the system was observed.

(a) Storm watervalve with debris (b) Blocked drain (c) Blocked manhole

Figure D.9: Bad state drainagesystem

E

Interviews

In order to get a better insight in the scale and inconvenience of the recent flood events, multiple interviews were conducted. During the visits to Ladysmith, residents and shop owners were asked about the flood events and their opinion. During the second visit to Ladysmith the project group met with a local newspaper, the Ladysmith Gazette, to gain more insight in the past events. Finally an interview was conducted with the Technical Officer of the Emnambithi/Ladysmith Municipality on the interventions of the Municipality since the recent flood events. The main findings of the interviews are discussed below.

E.1. Residents

From the interviews with residents and shop owners the following important notes can be made:

- Since the construction of the Qedusizi Dam there are no 'floods' in Ladysmith. The Klip River does not burst its banks. The amount of water on the streets and the accompanied inconveniences are far less than before the construction of the dam. Before the construction of the dam inhabitants were warned and evacuated in case of high water.
- High water on the streets often occurs during rainfall events. Explanations for the non-functioning system varies from a bad design to a lack of maintenance. The streets that often flood are the Lyell Street, Forbes Street and Alexander Street. These high waters usually stay for some hours.
- The height of the water level on the street varies per location. At some locations it reaches as far as knee height and at other location to ankle height. Some stores are completely out of business, while others only experience minor inconveniences. Cars are usually not able to drive on flooded roads and redirected to other streets.

E.2. Ladysmith Gazette

From the interview with the Ladysmith Gazette in September 2016 the following important notes were made:

- Fat is dumped in the sewage/drainage system by local fast food restaurants. A grease trap is a reasonable cheap solution to prevent this from happening.
- The drainage and sewage system are old and the maintenance is not executed properly. Residents often complain but the Ladysmith Municipality does not respond. When pipes are replaced often not the right material is used, which does not solve the problem.
- The municipality mostly focuses on draughts. Water restrictions are often applied because of water shortages.

E.3. Technical Officer Ladysmith Municipality

In order to get more detailed information on the recent flood events the disaster manager of Ladymith, Bennie Strydom, was contacted. Unfortunately, there was no additional information available because the disaster management did not recorded the events as disasters. The disaster manager mentioned the contact details of the engineer who was responsible for the remedial solutions after the flood events of 2011 and 2012, Shallon Pachkowdie. From the phone conversation in October 2016 with Shallon Pachkowdie the following important notes were made:

- The flood events of 2011 and 2012 were not due to the Klip River bursting it banks. The low lying areas of Ladysmith were flooded due to stormwater back flow.
- Flooding due to stormwater back flow has been a problem in Ladysmith for a long time. Already in the early 90's we installed flap and sluice gates. The problem with these is that they needed manual operation and sometimes did not function as desired because people would vandalise the spindles.
- We installed Tideflex® valves in 2013-2014 to solve the problems. It seems the problem is solved since.
- An additional measure that we are working on is a full storm water modelling exercise of the CBD. At this stage it is still in the prefase design. Once we get the model going we will try to implement the remedial measures.

F

Data analysis

In this appendix the analysis of the data, mentioned in Chapter 4: Analysis, and the accompanied calculations are shown. These calculations give a more extensive insight in the methods and formula's used in sections 1 and 2 of Chapter 4. In similar fashion as the main report, the subjects are structured according to Catchment Area, Klip River and Ladysmith Flood Protection Scheme.

F.1. Catchment

In Subsection 4.1.1 the different components of the Catchment Area are analysed in the subsections: Precipitation, Evaporation and Land use. Here extensive calculations are shown for all three subsections.

Precipitation In order to analyse the precipitation data for Ladysmith and the catchment area of the Klip River, the different measuring stations as discussed in Appendix A are used. The data from the measurement stations is delivered by the government of South Africa, DWS [9].

For the analysis of the precipitation in Ladysmith the data of a local measuring station, approximately 3 kilometres south-west of the CBD, is used. This measurement station gives hourly data and records since 1993 to present day. Because the record period is not of sufficient length, the data from nine other measurement stations is used as well. By using the Inverse Distance Method the data for Ladysmith is calculated from these stations, see Table A.1 and Figure F.1. The available data from the stations is daily. Daily measurements are good enough for determining whether there is a trend in precipitation or not, however for the analysis of the extreme values it is better to use measurements that are taken more often. Access to this data was however not possible, due to the small budget of this study.

For the analysis of the precipitation within the catchment area the station in van Reenen (V1E005) is used. This is the only measurement station within the catchment with a dataset of sufficient length.

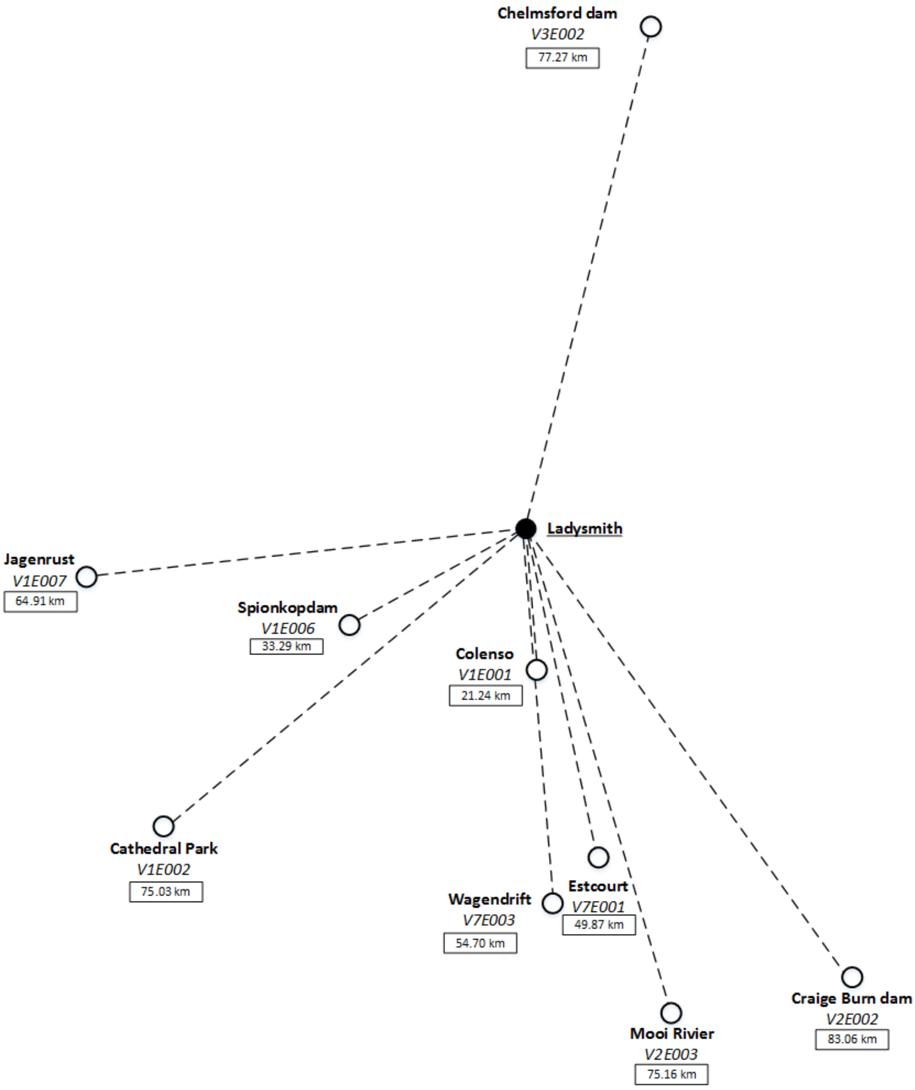


Figure F.1: The used measurement stations for the Inverse Distance Method

Two methods are used to analyse the trend in precipitation for Ladysmith and the upstream part of the catchment, namely:

(i) **Spearman's rank test**

The Spearman rank test is used to analyse whether there is a trend or not. Using this test, the data has to be split up into two groups. The data of these groups has to be ranked, starting with the lowest value [18]. If there is a trend or not is determined by the value of the Spearman coefficient, that could be calculated with the following formula:

$$R_{sp} = 1 - \frac{\sum {d_i}^2}{n \cdot (n^2 - 1)} \tag{F.1}$$

R_{sp}	Spearman coefficient	[−]
d_i	Difference in ranking	[−]
n	years per dataset	[−]

The results of the Spearman's rank test for Ladysmith and the catchment area can be found in Table F.1. The coefficient is low, so it can be concluded that there is no trend in precipitation over the years.

(ii) **Gumbel distribution**
The Gumbel distribution is used to determine the 1 : 100 precipitation per day. The maximum precipitation for each year is ranked. Based on this ranking, the return period (T) is determined by using the following formulas:

$$T = \frac{1}{p} \tag{F.2}$$

$$p = \frac{rank}{n+1} \tag{F.3}$$

The return periods are plotted against the corresponding precipitation values. By creating a trend line through these points, the 1 : 100 precipitation can be determined. Those are summarized in Table F.1. For both, Ladysmith and the catchment, there are no large differences in the extreme values for precipitation.

	Ladysmith	**Upstream**
Spearman rank coefficient	-0.294	-0.194
1 : 100 until 1997 (Gumbel)	93.03mm	250.27mm
1 : 100 until 2015 (Gumbel)	92.74mm	225.13mm

Table F.1: Overview precipitation analysis

To determine the return periods of the rainfall events during the observed flood events, the data of the rainfall measuring station at the airport is used. In this analysis a distinction between the hourly maximum and the daily maximum is made. For both the Gumbel distribution is used. The formula of the trend line is used to calculate the return periods of the precipitation during the flood events and the 1 : 10, 1 : 50 and 1 : 100 precipitation.

The hourly precipitation:

$$P = 11.372 \cdot LN(T) + 22.601 \tag{F.4}$$

The daily precipitation:

$$P = 16.671 \cdot LN(T) + 39.508 \tag{F.5}$$

For determining the return period of the precipitation during the past flood events, the maximum value during the flood event is used. The results of determining the return periods and the values of the extreme events are listed in Table F.2.

Event	**Maximum Hourly Precipitation** [mm]	**Return Period** [$year$]	**Maximum Daily Precipitation** [mm]	**Return Period** [$year$]
January 2011	15	0.47	75	8.40
January 2012	37.6	3.56	76	8.92
September 2012	14.8	0.46	91	21.9
December 2015	25	1.16	44.8	1.37
1:10	49.15	10	77.89	10
1:50	67.17	50	104.73	50
1:100	74.93	100	116.28	100

Table F.2: Return period precipitation in Ladysmith

F.1.1. Evaporation

In order to analyse the evaporation data for Ladysmith and for the catchment area, the different measuring stations as discussed in Appendix A are used. Here again the evaporation data for Ladysmith is determined with the Inverse Distance Method, see Figure F.1. For the evaporation data of the catchment the station at Van Reenen is used. There are different types of evaporation, for this study the most important one is interception. All water that is not intercepted will run off in the direction of the river or will infiltrate into the ground. Soil moisture will also reach the river, however at a larger time scale. So, if the interception decreases, a larger volume of water will directly flow into the river.

The interception is calculated by using the following formula [16]:

$$I_m = P_m(1 - exp(\frac{-Dn_r}{P_m})) \quad \text{(F.6)}$$

I_m	= Monthly interception	$[mm/month]$
P_m	= Monthly precipitation	$[mm/month]$
D	= Daily threshold	$[mm/day]$
n_r	= Rainy days	$[day/month]$

The monthly precipitation has been analysed in Section F.1, here it was concluded that this variable has not changed over the years. The other two variables in this formula, are studied below:

(i) **Daily treshold**, the daily threshold depends on soil characteristics, the potential evaporation and the vegetation cover. As can be read in Subsection F.1.2, a change in the vegetation cover is observed. Even if there is enough information available it remains difficult to determine the daily threshold. So, for this research the daily threshold, determined by Pitman (1973), is used. Based on a calibration of 23 catchments in South Africa, he calculated that a daily interception threshold of 1.5 mm/d could be used for the whole country [17].

(ii) **Number of rainy days**, a trend in the number of rainy days can be determined by plotting the number of rainy days against the precipitation. The formula used to determine the trend line is the formula that can be used to determine the number of rainy days, see Equation F.7 and Equation F.8. Between 1965 and 2015, there is no trend in the number of rainy days per month (for both Ladysmith and the upstream catchment). So, the same formula for the number of rainy days could be used for all years. Because the number of rain days depends on the monthly precipitation, P_m, this can be regarded as constant as well.

$$n_{r,Ladysmith} = 3.023 \cdot \exp(0.0099P_m) \quad \text{(F.7)}$$

$$n_{r,upstream} = 0.0327P_m + 4.3456 \quad \text{(F.8)}$$

From the analysis above it can be concluded that the interception in Ladysmith and within the catchment did not change since the construction of the Qedusizi dam.

F.1.2. Land use

There are three important factors that influence the land use: vegetation, soil moisture and urbanization.

a) Vegetation

The change of vegetation can be noticed by observing online Geographical Information System (GIS) data, generated by the United States Geological Survey (USGS) and National Aeronautics and Space administration (NASA). In 1975, 1990, 2000, 2005 and 2010 they did a Global Land Survey (GLS). For the analysis of the vegetation, the following data is used:

(i) *False colour/near Infrared.* [22]
The combination of the false colour and near infrared satellite data is used to analyse the presence and/or density of vegetation. The lighter the red, the more sparsely the area is vegetated.

(ii) *NDVI* [23]
Normalized Difference Vegetation Index (NDVI) is used to determine the health of vegetation in an

area. Near Infrared (NIR) and Visible Imaging System (VIS) is needed to determine the NDVI, the value is calculated by using the formula:

$$NDVI = \frac{NIR - VIS}{NIR + VIS} \tag{F.9}$$

If there is much more reflected light in NIR wavelengths than in VIS wavelengths, then the NDVI is high and vegetation is likely to be healthy (NASA [24]).

(iii) *Healthy vegetation* [20]
A combination of colour bands (5 4 1) shows how healthy the present vegetation is. Healthy vegetation will appear bright green; less healthy vegetation will appear brown etc.

An overview of the change in vegetation is listed in table Table F.3. There are no absolute amounts, only a comparison between the different years. In conclusion, since 1990 the vegetation has become less dense and less healthy. This means that a smaller amount of water will be absorbed by the vegetation. Thus, more water will runoff into the river. In 2010 there was a small increase in the density of the vegetation, the runoff coefficient decreased.

Data	False colour/ near infrared	NDVI	Healthy vegetation
1990	Dense vegetation	no data	Healthy
2000	Less dense compared to 1990	Less healthy vegetation compared to 1990	Less healthy vegetation compared to 1990
2005	Less dense compared to 1990, soil without vegetation	no data	Non-healthy vegetation (brown)
2010	Vegetation comparable with 2000	no data	no data

Table F.3: Vegetation change in different years

b) Soil moisture

Also for the soil moisture the data of the GIS is used. For this variable there is only data available from 2000 onwards. Since this year the average monthly soil moisture has been constant. Therefore no conclusion can be drawn about the change in soil moisture compared to the situation before the construction of the dam.
Three different types of soil are distinguished in the catchment of the Klip River, see Figure F.2. The soil types and their meaning are explained in Table F.4 [25].

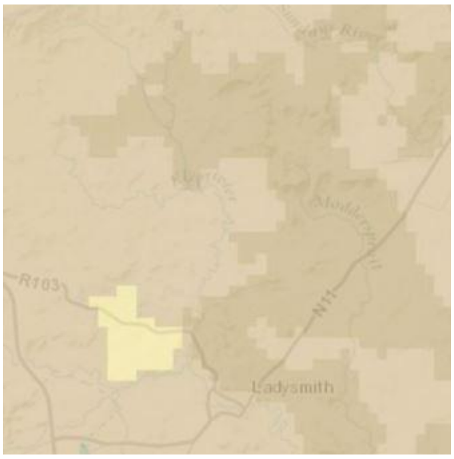


Figure F.2: Soil in and surround Ladysmith

	Soil Type	Runoff	Infiltration
Dark Brown	Sandy loam	Low	High
Light Brown	Sandy clay loam	High	Low
Yellow	Sand	Low	High

Table F.4: Soil groups surrounding Ladysmith

c) Urbanization

The runoff coefficient of urban areas is significantly higher compared to rural areas. This is mainly due to the pavement in those areas. So hardly any or no water can infiltrate. According to a study overseen by the Provincial Development Commission [39], there is a high rate of urbanization in KZN, but this urbanization is mostly along the coast or in the vicinity of the large cities. Around Ladysmith and in the Drakensbergen the changes in urbanization are small. So, we will consider the runoff of urban areas as a stable factor.

F.2. Klip River

The subsections about the Klip River in chapter 4 are: River Geometry and Resistance. The determination of manning values to express the hydraulic roughness of the Klip River along Ladysmith is described in this section.

One way to express the hydraulic roughness of a river channel and its floodplains is with the Manning roughness coefficient. In this section, estimations are made for the Manning roughness coefficient based on *Table 5-5. Values for the computation of the Roughness Coefficient by Eq. (5-12)* from *Open Channel Hydraulics* by Ven Te Chow(1959)[6] and *Table 2. –Factors that effect roughness of the channel* [Modified from Aldridge and Garret, 1973, table 2] from the *Guide for selecting Manning's Roughness coefficients for natural channels and floodplains* by the U.S. Department of Transportation(1984)[2].

The Manning's roughness coefficient n is established with the following equation:

$$n[s/m^{\frac{1}{3}}] = (n_b + n_1 + n_2 + n_3 + n_4) * m \tag{F.10}$$

In which:

n_b = **base factor** (0.020 – 0.028)
The n_b factor is a base value of n for a straight uniform, smooth channel in natural materials. It varies from 0.020 for Earth material and 0.028 for coarse gravel [2].

n_1 = **irregularity factor** (0.000 – 0.020)
The n_1 factor is a value added to correct for the effect of surface irregularities. Values vary from 0.000 for a very smooth channel to 0.020 for a severely irregular channel [2].

n_2 = **shape and size factor** (0.000 – 0.015)
The n_2 factor is added to take into account variations in shape and size of the channel cross section. The values for this factor vary from 0.000 for gradually changing cross sections to 0.015 for frequently changing cross sections [2]. This factor is disregarded, $n_2 = 0$, when calculating the roughness of floodplains.

n_3 = **obstruction factor** (0.000 – 0.050)
The n_3 factor is added to include the effect of obstructions present over the channel reach. Varying from 0.000 for a negligible amount of obstructions to a value of 0.050 for a severe amount and effect of obstructions [2].

n_4 = **vegetation factor** (0.002 – 0.100)
The n_4 factor is a value added to include vegetation and flow conditions. It varies from a value of 0.002 for small amount and effect of vegetation to a value of 0.100 for very large amounts and effect of vegetation [2].

m - **meandering factor** (1.0 – 1.3)
The meandering factor m is a correction factor for the meandering of the channel [2]. It varies between 1.0 and 1.3 and is determined by the ratio of the channel length over the valley length; the sinuosity index. Its minimum value is assigned to rivers with a sinuosity index of 1.0 to 1.2 and its maximum value is assigned to

rivers with a sinuosity index of at least 1.5. When calculating the roughness of floodplains, the meandering factor is set to 1.0.

To get insight on the hydraulic roughness of the Klip River, a short assessment is done by grading this roughness with values for the Manning roughness coefficient. The Manning values of the Klip River's main channel and its floodplains are based on the field observations done in September 2016, see Appendix D. A distinction is made between the reaches up- and downstream of the knickpoint. The river's physical characteristics vary a lot for these two reaches. The upstream reach has a length of 5.5 kilometer and runs from the Qedusizi Dam unto the knickpoint. The downstream reach has a length of 18.5 kilometer and runs from the knickpoint unto cross-section 31 from the monitoring surveys. The valley lengths for these reaches were obtained with the *Path function* from Google Earth^{TM}. The channel length, valley length and the respective sinuosity indexes are shown in Table F.5. Both reaches satisfy for the maximum meandering factor of 1.3.

Reach	**Channel length [m]**	**Valley length [m]**	**Sinuosity Index [-]**	**Meandering factor [-]**
1	5.5	3.65	1.51	1.3
2	18.5	6.62	2.79	1.3

Table F.5: Meandering factor justification

Apart from the meandering factor, the other factors completing the total Manning value are assigned with three degrees: minimum, medium and maximum. This is done to take into account the empirical uncertainty and high variability of the river's physical characteristics. The minimum and maximum factor values are those values that are deemed as the lowest and highest occurring roughness situation for that particular factor and reach. Table F.6 and Table F.7 show the estimated Manning factors and their different degrees, for the reaches up- and downstream of the knickpoint. The bottom row of the tables shows the final Manning values obtained with those factors and Equation F.10.

Manning factor	**Main river channel**			**Floodplains**		
	Min.	*Med.*	*Max.*	*Min.*	*Med.*	*Max.*
n_b	0.020	0.023	0.026	0.020	0.023	0.026
n_1	0.006	0.012	0.020	0.006	0.011	0.020
n_2	0.005	0.0075	0.010	0.000	0.000	0.000
n_3	0.005	0.010	0.015	0.005	0.010	0.019
n_4	0.002	0.012	0.025	0.005	0.012	0.050
m	1.3	1.3	1.3	1.0	1.0	1.0
n	**0.0494**	**0.0839**	**0.1248**	**0.036**	**0.056**	**0.115**

Table F.6: Manning values $[s/m^{\frac{1}{3}}]$, upstream of knickpoint

Manning factor	**Main river channel**			**Floodplains**		
	Min.	*Med.*	*Max.*	*Min.*	*Med.*	*Max.*
n_b	0.020	0.023	0.026	0.020	0.023	0.026
n_1	0.001	0.006	0.010	0.006	0.011	0.020
n_2	0.000	0.005	0.010	0.000	0.000	0.000
n_3	0.000	0.005	0.015	0.005	0.010	0.019
n_4	0.002	0.006	0.010	0.005	0.012	0.050
m	1.3	1.3	1.3	1.0	1.0	1.0
n	**0.0299**	**0.0585**	**0.0923**	**0.036**	**0.056**	**0.115**

Table F.7: Manning values $[s/m^{\frac{1}{3}}]$, downstream of knickpoint

The obtained Manning values show large differences between the reach upstream of the knickpoint and the reach downstream of the knickpoint. It is stressed that the Maximum values are stacks of worst-case situations and thus to be seen as a very extreme situation. It can be remarked that the values for the flood plains are less varying and maybe lower than expected. This is explained by the fact that the n_2 factor is disregarded and the meandering factor is set to 1.0, of which the latter is of great influence to the final Manning value.

In the HEC-RAS model made by Royal HaskoningDHV for the 2013 floodline report, the roughness was indicated per cross-section with Manning values between 0.040 and 0.045 for the main channel and values between 0.050 and 0.065 for the flood plains. Considering the large variety between the reaches upstream and downstream of the knickpoint, it is remarkable that such a short range is used for the Manning values, especially for the main channel. In addition, the minimum Manning value from Table F.6 for the main channel is even higher than the highest value 0.045 used in the 2013 flood line report.

It should be noted that no field observations were done for the Klip River downstream of the CBD. But since the slope is relatively constant below the knickpoint, the river's characteristics are assumed to be comparable to the river characteristics along the studied area of the CBD.

F.3. Ladysmith Flood Protection Scheme

In Subsection 4.1.3 the different components of the LFPS are analysed in the subsections: Qedusizi dam, Levees and Stormwater valves. The calculations that were done to support the conclusions given in Subsection 4.1.3 are explained.

F.3.1. Hand calculation for the discharge capacity of the CBD

It is difficult to determine whether this assumption is correct. In order to get an indication of the water depth around the CBD for a discharge of 450 m^3/s, a hand calculation was made. For this calculation a rectangular river bed with a width of 50 m and a Manning coefficient of 0.05 was used for all cross-sections. Furthermore, a slope of 0.005 was assumed for all cross-sections before the knickpoint and a slope of 0.000267 was assumed for all cross-sections after this knickpoint. Equation F.11 was used for the calculation of the water depth.

$$Q = \frac{1}{n} * R^{\frac{2}{3}} * \sqrt{i} * A \tag{F.11}$$

Q	= Discharge	$[m^3/s]$
n	= Manning	$[s/m^{\frac{1}{3}}]$
R	= Hydraulic radius	$[m]$
i	= slope	$[-]$
A	= cross-sectional area	$[m^2]$

For a discharge of 450 m^3/s, a water depth of 8 m was calculated around the CBD of Ladysmith. This water depth seems quite high compared to the crest levels of the different cross-sections in Appendix... This calculation is very rough and only gives an indication of the water level. A model should be used in order to obtain a more accurate water depth.

F.3.2. Discharge Flagstone Spruit between Qedusizi Dam and Ladysmith

A potential large contribution to the discharge of the Klip River between the Qedusizi Dam and Ladysmith is the discharge of the Flagstone Spruit. This tributary adjoins the Klip River 2.5 km downstream of the Qedusizi dam and drains the catchment southwest of Ladysmith. In reports regarding the design properties it is mentioned that an additional 50 m^3/s is attributed to the intermediate catchment between the dam and Ladysmith. Two analysis were conducted in order to check whether this assumption seems correct.

First the catchment area of the Flagstone Spruit was determined in order to calculate accompanied rainfall for 50 m^3/s. Using the elevation function in Google Earth™, the catchment area of the Flagstone Spruit is calculated to be around 50 km^2. The rational method was used to calculate which rainfall would cause a discharge of 50 m^3/s in the Flagstone Spruit. The equation that is used in this method is depicted in Equation F.12. For the runoff coefficient a value of 0.3 was assumed.

$$Q = \frac{C * I * A}{3.6} \tag{F.12}$$

This calculation shows that a rainfall of 12 mm/h would cause a discharge of 50 m^3/s in the Flagstone Spruit.

Q	= Discharge	$[m^3/s]$
C	= Runoff coefficient	$[-]$
I	= Rainfall intensity	$[mm/h]$
A	= catchment area	$[km^2]$

This seems quite low for a 1 in 100 year event when you look at the rainfall characteristics of this area. Using the Gumbel distribution for the yearly extreme rainfall events, it is calculated that the return period in this area for a rainfall of 12 mm/h is 0.36 year. Further research should be carried out in order to look more carefully at this assumption. In this research other methods can be used to determine the rainfall related to a discharge of 50 m^3/s. The rational method that is used here is usually applied for catchments of up to 30 km^2. Other methods might give more accurate results.

The second analysis uses the difference in discharge data measured from measurement stations V1R005 and V1H038 in order to determine if the assumption of 50 m^3/s seems correct. For this analysis, a matlab script is used. The matlab script filters out the data points at which the water level measured at measuring station V1H038 is higher than a certain level k. For this analysis, this level was set to 2 m. For these data points, the discharges at V1R005 and V1H038 were plotted for three days: the day that the water level exceeded k, the day before and the day after. Also, the hourly rainfall in Ladysmith during these days was plotted. By comparison of these plots, it can be concluded that the attribution of the Flagstone Spruit to the discharge in the Klip River often exceeds 50 m^3/s. An example can be seen in Figure F.3. From these results, it could also be concluded that a large difference in discharge between these two measuring stations was often accompanied by heavy rainfall in Ladysmith. This suggests that the catchment of the Flagstone Spruit is very sensitive to rainfall and has a very direct response.

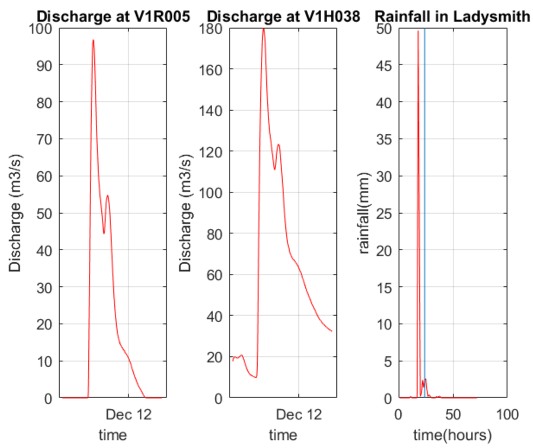


Figure F.3: Discharge at V1R005 and V1H038 in December 2012

F.3.3. Water level behind Qedusizi dam

The water level just behind the Qedusizi Dam and the accompanying discharge through the dam have been monitored since the construction of the dam in 1998. If the water level behind the dam is equal to the spillway level, the flow through the dam is expected to be 400 m^3/s. The height of the dam is 20.5 m. The dam should be able to contain a 1 in 100 year flood event, thus the water level behind the dam with a return period of 100 years should be lower than 20.5 m.

For this analysis the Gumbel type I and the Log-Pearson type III were used. Both analyses were performed using the yearly maximum water levels. The maximum water level per year can be seen in Table F.8.

Gumbel type I

The Gumbel distribution is the most common extreme value distribution, and is often referred to as the type I distribution. In order to determine the Gumbel distribution the annual extreme values are of importance,

Year	Max. water level [m]
1998	4.984
1999	5.771
2000	12.155
2001	9.987
2002	2.310
2003	2.666
2004	4.212
2005	3.887
2006	11.750
2007	5.500
2008	5.668
2009	9.096
2010	6.991
2011	10.352
2012	10.353
2013	3.952
2014	4.127
2015	5.112

Table F.8: Water levels used in the Gumbel and Pearson extreme value analysis

as shown in Table F.8.
To determine the probability of non-exceedance of the annual extremes the following equation is used:

$$q = exp(-exp(-y)) \tag{F.13}$$

y = reduced variate
q = probability of non-exceedance

Subsequently, this probability of non-exceedance q can be used to calculate the return period T using Equation F.14.

$$T = \frac{1}{1-q} \tag{F.14}$$

T = return period
q = probability of non-exceedance

According to Gumbel, the reduced variate y is defined as a linear function of X according to Equation F.15.

$$y = a(X-b) \tag{F.15}$$

a = dispersion coefficient
b = node
X = extreme value

The dispersion coefficient and the node can be calculated by Equation F.16 and Equation F.17 respectively.

$$a = \frac{s_y}{s} \tag{F.16}$$

$$b = X_m - s\frac{Y_m}{s_y} \tag{F.17}$$

s_y = standard deviation of the reduced variate
s = standard variation of the annual extremes
X_m = mean of the annual extremes
y_m = mean of the reduced variate

The standard deviation of the reduced variate S_y and the mean of the reduced variate Y_m are dependent on the amount of data points that are used for the analysis. In this case, 18 maximum water levels were used for the analysis. This corresponds to a standard deviation S_y of 1.0396 and a mean Y_m of 0.5182.

For all annual maximum water levels, the return period was calculated using Equation F.13 and Equation F.14. The annual maximum water levels and their return periods was plotted and a logaritmic trendline was added. This can be seen in Figure F.4.

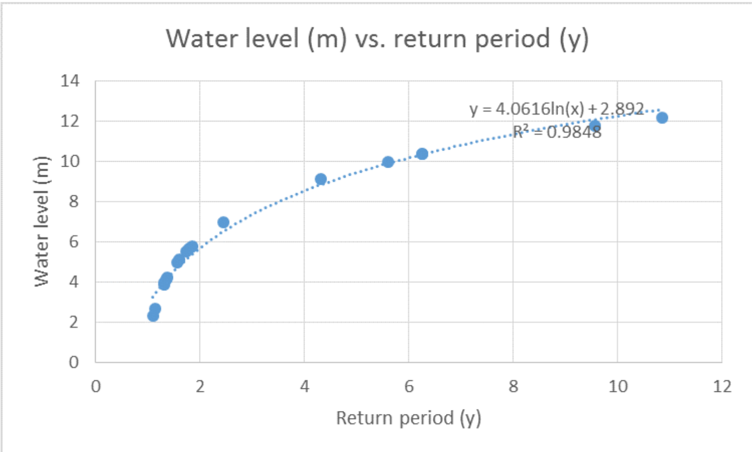


Figure F.4: Gumbel distribution for water levels, V1R005

For the determination of the water level behind the dam for different return periods, the reduced variate per return period was calculated using Equation F.13 and Equation F.14. Using Equation F.15 with the a and b values related to this specific data set, the water level with a certain return period was calculated. The same was done for the water levels with a return period of 10, 50 and 10 years. The results can be seen in Table F.9.

Log-Pearson type III

The Pearson distribution used for this calculation is the Type III distribution, or gamma distribution. In order to determine the Pearson distribution the annual extreme values were ranked from maximum to minimum values. Then, the logaritmic of these water levels was determined. The Log-Pearson distribution calculates the water level with a certain return period using formula Equation F.18.

$$log(h) = X_{log(h)} + K_{weighted} * s_{log(h)} \tag{F.18}$$

$X_{log(h)}$ = mean of the logaritmic of the water levels
$K_{weighted}$ = weighted skew coefficient
$s_{log(h)}$ = standard deviation of the logaritmic of the water levels

The weighted skew coefficient can be read from tables when the skew coefficient and the return period for which you want to determine the water level are known. The skew coefficient is calculated using equation Equation F.19.

$$K = \frac{n * \sum log(h) - X_{log(h)}{}^3}{(n-1)(n-2)(s_{log(h)})^3} \tag{F.19}$$

n	= number of datapoints
$X_{log(h)}$	= mean of the logaritmic of the water levels
$s_{log(h)}$	= standard deviation of the logaritmic of the water levels

Using these formulas, the water levels for different return periods are calculated. These water levels and their return periods are plotted and a logaritmic trendline is added. This can be seen in Figure F.5.

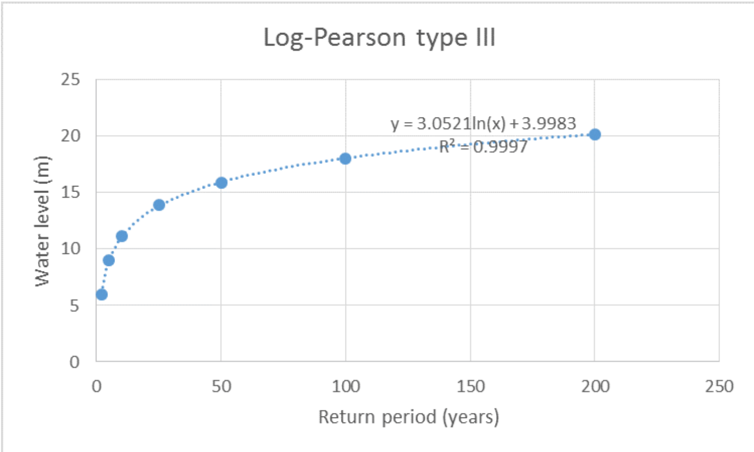


Figure F.5: Log-Pearson distribution for water levels, V1R005

In Table F.9 the water levels behind the Qedusizi Dam for different return periods are shown. It can be seen that the water level behind the dam for a return period of 100 years is lower than 20.5 m for both analyses. These results indicate the height of the dam is indeed sufficient to maintain the 1 in 100 year flood event. However, these results should be handled with care, since they are based on a short record of 18 years.

Return period $[years]$	**Water level** $[m]$ Gumbel	**Water level** $[m]$ Pearson
1:10	11.9	11.1
1:50	16.9	15.9
1:100	19.1	18.0

Table F.9: The water level behind the dam with associated return period according to the Gumbel and Pearson distribution

F.3.4. Discharge Qedusizi dam

The two uncontrolled bottom outlets of the Qedusizi dam have a height of 3.2 m and width of 5.0 m. According to the reports regarding the design discharge the water level would reach the spillway in case of a 1 in 100 flood event. From design drawings the spillway height was determined to be 20.5 m, see Figure F.6. From the rating table used at measurement station V1R005 the discharge can be determined per accompanied water level. From the rating table it was determined that at spillway level the accompanied discharge is approximately $400 m^3/s$.

In order to verify the relation between these values, a simple hand calculation was conducted. With the assumption that dissipation is absent and streamlines are straight just before the outflow, the Bernoulli equation can be used to determine the outflow for different head differences with the accompanied contraction coefficient. From the calculation the contraction coefficient of the outlets used in the rating table could be determined for different velocities. The contraction coefficient shows realistic values (μ is approximately 0.65 when the water level behind the dam is 20.5 m and discharge 400 m^3/s) and decreases with increasing velocity through the outlets. This behaviour is physically correct, due to the increase of the boundary layer.

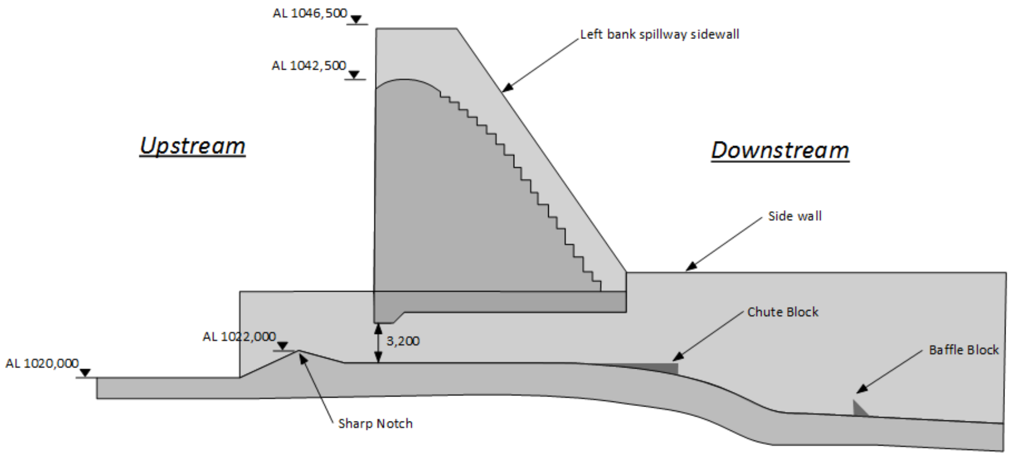


Figure F.6: Design bottom outlet (based on design drawing from Dam Safety Inspection in March 2010)

G

HEC-RAS

G.1. Computation procedure

The explanation of the computation procedure of the HEC-RAS model given in this section is based on the reference manual of HEC-RAS [5].

For the steady flow 1D model, HEC-RAS computes the water levels from one cross-section of a river to the next using the 1D energy equation. The computation starts either at the downstream or at the upstream boundary of the modelled system; this depends on whether a subcritical or a supercritical flow regime is assumed. The computation of the water level at the next cross-section is iterated until Equation G.1 is in balance. Then the program moves on to the next cross-section and the procedure is repeated. This process is continues until the water level in the last cross-section is calculated. The iterative procedure used in HEC-RAS is the standard step model. In the remainder of this section, the steps that the model follows during one iteration are described.

$$Z_1 + Y_1 + \frac{\alpha_1 V_1^2}{2g} = Z_2 + Y_2 + \frac{\alpha_2 V_2^2}{2g} + h_e \tag{G.1}$$

Z_1, Z_2	= bed level elevations in cross-sections 1 and 2	$[m]$
Y_1, Y_2	= water depths in cross-section 1 and 2	$[m]$
V_1, V_2	= flow velocity in cross-section 1 and 2	$[m/s]$
α_1, α_2	= velocity weighting coefficients	$[-]$
h_e	= energy head loss	$[m]$
g	= gravitational acceleration	$[m/s^2]$

In the 1D energy equation the bed level elevations Z_1 and Z_2, the water level at the first cross-section Y_1 and the gravitational acceleration g are directly fed into the program. The velocity in the first cross-section V_1 is calculated from the known water level, the known geometry and the known discharge in this cross-section. A water level in the next cross-section is assumed. With this water level, the known geometry and the known discharge at this cross-section the velocity V_2 is calculated. The alpha coefficients α_1 and α_2 and the energy head loss h_e are calculated by the program. This calculation is explained in further detail below.

The energy head loss h_e between two cross-sections consists of loss due to friction losses and loss due to contraction or expansion losses. This is calculated using Equation G.2.

$$h_e = LS_f + C|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}| \tag{G.2}$$

L	= discharge weighted reach length	$[m]$
S_f	= representative friction slope between two sections	$[-]$
C	= expansion or contraction loss coefficient	$[-]$
V_1, V_2	= flow velocity in cross-section 1 and 2	$[m/s]$
α_1, α_2	= velocity weighting coefficients	$[-]$
g	= gravitational acceleration	$[m/s^2]$

The first factor in this equation represents losses due to friction. The second term represents the losses due to either expansion (if the cross-section becomes larger) or compression (if the cross-section becomes smaller). The default value for the contraction coefficient is 0.1, for compression the default is 0.3. In this equation, the slope of the energy gradeline S_f, the contraction or expansion coefficient C and gravitational acceleration g are fed directly into the model. The alpha coefficients α_1 and α_2 and the discharge weighted reach lengths L are computed by the program. This is explained in further detail below.

The discharge weighted reach length L is calculated using Equation G.3.

$$L = \frac{L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}} \tag{G.3}$$

L_{lob}	Cross-section reach length, left overbank flow	$[m]$
L_{ch}	Cross-section reach length, main channel flow	$[m]$
L_{rob}	Cross-section reach length, right overbank flow	$[m]$
Q_{lob}	Arithmetic flow average between sections, left overbank	$[m^3/s]$
Q_{ch}	Arithmetic flow average between sections, main channel	$[m^3/s]$
Q_{rob}	Arithmetic flow average between sections, right overbank	$[m^3/s]$

In this equation, the cross-section reach lengths L_lob, L_ch and L_rob are directly fed into the model. The average flows of the river channel and the two floodplains are calculated using Equation G.4.

$$Q = K \cdot S_f^{\frac{1}{2}} \tag{G.4}$$

K	= conveyance for subdivision	$[m^3/s]$
S_f	= slope of the energy gradeline	$[-]$

The conveyance is calculated for the river channel and both floodplains using Equation G.5.

$$K = \frac{1.486}{n} AR^{\frac{2}{3}} \tag{G.5}$$

n	= Manning roughness coefficient	$[s/m^{\frac{1}{3}}]$
A	= cross-sectional area	$[m^2]$
R	= hydraulic radius	$[m]$

The contraction or expansion losses are dependent on the difference in average velocity head between two cross-sections. The velocity head can differ over a cross-section, because the velocity is not constant over the cross-section. This is illustrated in Figure G.1. In HEC-RAS, the mean kinetic energy in a cross-section is calculated using the velocity head weighting coefficient α. This coefficient is calculated using Equation G.6.

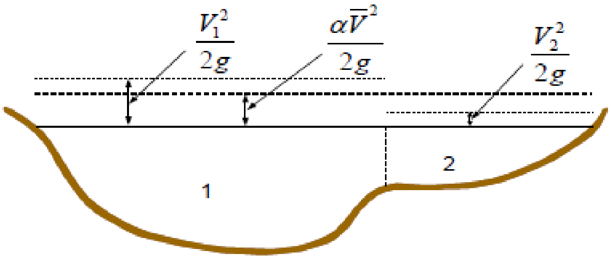


Figure G.1: Example of how the mean energy head is obtained

$$\alpha = \frac{(A_t)^2 [\frac{K_{lob}^3}{A_{lob}^2} + \frac{K_{ch}^3}{A_{ch}^2} + \frac{K_{rob}^3}{A_{rob}^2}]}{K_t^3} \tag{G.6}$$

A_t	= total flow area of cross-section	$[m^2]$
A_{lob}, A_{ch}, A_{rob}	= flow areas of left overbank, main channel and right overbank respectively	$[m^2]$
K_t	= total conveyance of cross-section	$[m^3/s]$
K_{lob}, K_{ch}, K_{rob}	= conveyances of left overbank, main channel and right overbank respectively	$[m^3/s]$

Once the alpha coefficients α_1 and α_2 and the head energy loss h_e are calculated for the estimated water level in cross-section 2, the energy equation can be used to calculate the new water level in cross-section 2. If this value differs from the assumed value with a certain range, the procedure is repeated for a new assumption of the water level in cross-section 2.

The computation procedure is described above is only valid under a few assumptions:

1. Flow is steady. The 1D energy equation does not contain time dependent terms. The flow is thus assumed to stay constant in time.
2. Flow is gradually varied, because the 1D energy equation is based on the premise that a hydrostatic pressure distribution exists at all cross-sections. If the flow is rapidly varying, this premise cannot hold.
3. Flow is one dimensional, because the 1D energy equation is based on the premise that the total energy head is the same for all points in a cross-section.
4. River channels have ''small slopes''. This is based on the true derivation of the energy equation, where the vertical pressure head is calculated using Equation G.7.

$$H_p = d \cos\theta \tag{G.7}$$

H_p	= vertical pressure head	$[m]$
d	= depth of the water measured perpendicular to the channel bottom	$[m]$
θ	= the channel bottom slope expressed in degrees	

By using the depth d instead H, an error is introduced. However, if the slope of the river is small, this error remains small.

G.2. Results HEC-RAS

In this section, the results of the HEC-RAS model are summarized and briefly explained. In order to keep things clear, only the results of the model are given for the cross-sections around the CBD of Ladysmith. The results of the river section before and after the CBD are not published in this report. The HEC-RAS model is used to model the water levels in the Klip River for the past flood events and extreme future flood events. These water levels are compared with the street levels of the storm valves and with the crest levels. This section starts with the comparison of the modelled water heights with the crest levels. A distinction is made between the results of the past flood events and the future flood events. Subsequently the comparison with the street levels of the storm valves is given, again both for the past flood events and for the future flood events.

G.2.1. Comparison of the water levels in the Klip River with the crest levels

In the tables below, the freeboards are listed for the past and future flood events. The freeboards are calculated by subtracting the water level in the Klip River from the crest level. This is done for both sides of the river bed, resulting in two freeboards per cross-section: one for the Left-Hand-Side(LHS) and one for the Right-Hand-Side(RHS). The LHS is defined as the left side of the river, when looking in downstream direction. This is the most important side in this project, since the CBD of Ladysmith is located at the left bank of the Klip River. Therefore, the columns that contain the freeboards for the LHS of the river are blue. The crest levels at both sides are defined as the highest heights measured during the cross-section survey in October 2016, commissioned by Royal HaskoningDHV. A negative value in the table implies that the water level in the Klip River is higher than the crest level. This would mean that the levee at this cross-section is topped over. In order to give a good overview, all negative values in the tables are bold.

The first column in the table contains the number of the cross-section in the HEC-RAS model. Then every two columns contain the freeboards at the LHS and RHS of the river respectively, for different Manning coefficients.

Past flood events

	RHDHV		**Low**		**Medium**		**High**	
cross section	**LHS**	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**
8	1.97	2.18	2.81	3.02	0.93	1.14	**-0.35**	**-0.14**
9	2.61	1.31	3.44	2.14	1.58	0.28	0.31	**-0.99**
10	2.26	1.59	3.04	2.37	1.25	0.58	**-0.01**	**-0.68**
10.6	1.26	1.9	2.12	2.76	0.21	0.85	**-1.06**	**-0.42**
11	2.21	2.1	3.05	2.94	1.21	1.1	**-0.06**	**-0.17**
12	1.78	2.15	2.61	2.98	0.83	1.2	**-0.44**	**-0.07**
13	2.65	2.31	3.46	3.12	1.72	1.38	0.48	0.14
14	1.82	2.22	2.62	3.02	0.93	1.33	**-0.25**	0.15
15	1.66	1.59	2.43	2.36	0.8	0.73	**-0.37**	**-0.44**
16	**-2.51**	2.61	**-1.75**	3.37	**-3.35**	1.77	**-4.5**	0.62
17.1	1.95	1.1	2.75	1.9	1.08	0.23	**-0.08**	**-0.93**
17.2	2.45	1.36	3.37	2.28	1.53	0.44	0.34	**-0.75**
18	2.2	2.78	3.09	3.67	1.27	1.85	0.02	0.6
18.6	2.39	2.48	3.27	3.36	1.44	1.53	0.24	0.33
18.7	1.79	2.47	2.66	3.34	0.86	1.54	**-0.33**	0.35
19	1.03	2	1.89	2.86	0.1	1.07	**-1.1**	**-0.13**
19.6	2.6	2.31	3.47	3.18	1.66	1.37	0.49	0.2
20	1.68	2.66	2.56	3.54	0.73	1.71	**-0.44**	0.54
20.6	2.13	0.54	3	1.41	1.2	**-0.39**	0.07	**-1.52**

Table G.1: Freeboards for flood event 1

cross section	RHDHV		Low		Medium		High	
	LHS	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**
8	2.53	2.74	3.38	3.59	1.56	1.77	0.26	0.47
9	3.12	1.82	3.96	2.66	2.16	0.86	0.88	**-0.42**
10	2.75	2.08	3.56	2.89	1.8	1.13	0.55	**-0.12**
10.6	1.68	2.32	2.55	3.19	0.7	1.34	**-0.57**	0.07
11	2.58	2.47	3.44	3.33	1.63	1.52	0.38	0.27
12	2.07	2.44	2.93	3.3	1.14	1.51	**-0.08**	0.29
13	2.88	2.54	3.73	3.39	1.97	1.63	0.77	0.43
14	1.98	2.38	2.83	3.23	1.1	1.5	**-0.07**	0.33
15	1.77	1.7	2.61	2.54	0.91	0.84	**-0.25**	**-0.32**
16	**-2.39**	2.73	**-1.56**	3.56	**-3.25**	1.87	**-4.39**	0.73
17.1	2.02	1.17	2.88	2.03	1.15	0.3	0	**-0.85**
17.2	2.42	1.33	3.33	2.24	1.53	0.44	0.36	**-0.73**
18	2.1	2.68	2.99	3.57	1.2	1.78	0	0.58
18.6	2.18	2.27	3.06	3.15	1.28	1.37	0.11	0.2
18.7	1.58	2.26	2.46	3.14	0.69	1.37	**-0.48**	0.2
19	0.8	1.77	1.67	2.64	**-0.09**	0.88	**-1.26**	**-0.29**
19.6	2.33	2.04	3.2	2.91	1.43	1.14	0.28	**-0.01**
20	1.38	2.36	2.26	3.24	0.49	1.47	**-0.66**	0.32
20.6	1.83	0.24	2.7	1.11	0.94	**-0.65**	**-0.19**	**-1.78**

Table G.2: Freeboards for flood event 2

cross section	RHDHV		Low		Medium		High	
	LHS	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**	**LHS**	**RHS**
8	2.11	2.32	2.94	3.15	1.08	1.29	**-0.21**	0
9	2.75	1.45	3.56	2.26	1.72	0.42	0.45	**-0.85**
10	2.39	1.72	3.16	2.49	1.39	0.72	0.13	**-0.54**
10.6	1.38	2.02	2.23	2.87	0.34	0.98	**-0.93**	**-0.29**
11	2.33	2.22	3.16	3.05	1.34	1.23	0.07	**-0.04**
12	1.89	2.26	2.71	3.08	0.95	1.32	**-0.31**	0.06
13	2.75	2.41	3.56	3.22	1.84	1.5	0.6	0.26
14	1.92	2.32	2.71	3.11	1.04	1.44	**-0.15**	0.25
15	1.75	1.68	2.52	2.45	0.9	0.83	**-0.27**	**-0.34**
16	**-2.42**	2.7	**-1.66**	3.46	**-3.25**	1.87	**-4.4**	0.72
17.1	2.04	1.19	2.84	1.99	1.18	0.33	0.02	**-0.83**
17.2	2.53	1.44	3.44	2.35	1.62	0.53	0.43	**-0.66**
18	2.28	2.86	3.16	3.74	1.36	1.94	0.11	0.69
18.6	2.45	2.54	3.33	3.42	1.52	1.61	0.31	0.4
18.7	1.85	2.53	2.72	3.4	0.93	1.61	**-0.26**	0.42
19	1.09	2.06	1.94	2.91	0.17	1.14	**-1.03**	**-0.06**
19.6	2.65	2.36	3.52	3.23	1.73	1.44	0.56	0.27
20	1.73	2.71	2.61	3.59	0.79	1.77	**-0.37**	0.61
20.6	2.18	0.59	3.05	1.46	1.26	**-0.33**	0.13	**-1.46**

Table G.3: Freeboards for flood event 3

Future flood events

cross section	RHDHV LHS	RHDHV RHS	Low LHS	Low RHS	Medium LHS	Medium RHS	High LHS	High RHS
8	**-0.55**	**-0.34**	0.5	0.71	**-1.67**	**-1.46**	**-2.97**	**-2.76**
9	0.08	**-1.22**	1.16	**-0.14**	**-1.05**	**-2.35**	**-2.36**	**-3.66**
10	**-0.32**	**-0.99**	0.76	0.09	**-1.44**	**-2.11**	**-2.74**	**-3.41**
10.6	**-1.31**	**-0.67**	**-0.2**	0.44	**-2.44**	**-1.8**	**-3.82**	**-3.18**
11	**-0.38**	**-0.49**	0.73	0.62	**-1.49**	**-1.6**	**-2.87**	**-2.98**
12	**-0.85**	**-0.48**	0.29	0.66	**-1.95**	**-1.58**	**-3.32**	**-2.95**
13	**-0.01**	**-0.35**	1.13	0.79	**-1.1**	**-1.44**	**-2.45**	**-2.79**
14	**-0.86**	**-0.46**	0.27	0.67	**-1.93**	**-1.53**	**-3.25**	**-2.85**
15	**-1.05**	**-1.12**	0.08	0.01	**-2.1**	**-2.17**	**-3.39**	**-3.46**
16	**-5.25**	**-0.13**	**-4.15**	0.97	**-6.26**	**-1.14**	**-7.53**	**-2.41**
17.1	**-0.79**	**-1.64**	0.37	**-0.48**	**-1.83**	**-2.68**	**-3.11**	**-3.96**
17.2	**-0.27**	**-1.36**	0.95	**-0.14**	**-1.33**	**-2.42**	**-2.66**	**-3.75**
18	**-0.65**	**-0.07**	0.61	1.19	**-1.7**	**-1.12**	**-3.04**	**-2.46**
18.6	**-0.32**	**-0.23**	0.74	0.83	**-1.33**	**-1.24**	**-2.69**	**-2.6**
18.7	**-0.93**	**-0.25**	0.11	0.79	**-1.92**	**-1.24**	**-3.26**	**-2.58**
19	**-1.72**	**-0.75**	**-0.68**	0.29	**-2.7**	**-1.73**	**-4.05**	**-3.08**
19.6	**-0.08**	**-0.37**	0.91	0.62	**-1.06**	**-1.35**	**-2.42**	**-2.71**
20	**-1.03**	**-0.05**	**-0.04**	0.94	**-2.02**	**-1.04**	**-3.36**	**-2.38**
20.6	**-0.52**	**-2.11**	0.42	**-1.17**	**-1.48**	**-3.07**	**-2.81**	**-4.4**

Table G.4: Freeboards for the 1:100 flood event

cross section	RHDHV LHS	RHDHV RHS	Low LHS	Low RHS	Medium LHS	Medium RHS	High LHS	High RHS
8	0.17	0.38	1.13	1.34	**-0.97**	**-0.76**	**-2.19**	**-1.98**
9	0.82	**-0.48**	1.79	0.49	**-0.35**	**-1.65**	**-1.56**	**-2.86**
10	0.47	**-0.2**	1.37	0.7	**-0.72**	**-1.39**	**-1.93**	**-2.6**
10.6	**-0.55**	0.09	0.42	1.06	**-1.74**	**-1.1**	**-3.06**	**-2.42**
11	0.4	0.29	1.36	1.25	**-0.78**	**-0.89**	**-2.1**	**-2.21**
12	**-0.03**	0.34	0.95	1.32	**-1.22**	**-0.85**	**-2.54**	**-2.17**
13	0.83	0.49	1.79	1.45	**-0.36**	**-0.7**	**-1.66**	**-2**
14	**-0.01**	0.39	0.94	1.34	**-1.18**	**-0.78**	**-2.45**	**-2.05**
15	**-0.18**	**-0.25**	0.76	0.69	**-1.35**	**-1.42**	**-2.59**	**-2.66**
16	**-4.38**	0.74	**-3.47**	1.65	**-5.51**	**-0.39**	**-6.73**	**-1.61**
17.1	0.09	**-0.76**	1.05	0.2	**-1.08**	**-1.93**	**-2.31**	**-3.16**
17.2	0.58	**-0.51**	1.6	0.51	**-0.6**	**-1.69**	**-1.84**	**-2.93**
18	0.24	0.82	1.28	1.86	**-0.96**	**-0.38**	**-2.21**	**-1.63**
18.6	0.45	0.54	1.42	1.51	**-0.63**	**-0.54**	**-1.84**	**-1.75**
18.7	**-0.15**	0.53	0.79	1.47	**-1.21**	**-0.53**	**-2.41**	**-1.73**
19	**-0.92**	0.05	0.01	0.98	**-1.99**	**-1.02**	**-3.19**	**-2.22**
19.6	0.67	0.38	1.59	1.3	**-0.36**	**-0.65**	**-1.56**	**-1.85**
20	**-0.27**	0.71	0.65	1.63	**-1.31**	**-0.33**	**-2.5**	**-1.52**
20.6	0.2	**-1.39**	1.11	**-0.48**	**-0.78**	**-2.37**	**-1.95**	**-3.54**

Table G.5: Freeboards for the 1:50 flood event

cross section	RHDHV		Low		Medium		High	
	LHS	RHS	LHS	RHS	LHS	RHS	LHS	RHS
8	1.68	1.89	2.54	2.75	0.62	0.83	**-0.69**	**-0.48**
9	2.32	1.02	3.18	1.88	1.28	**-0.02**	**-0.04**	**-1.34**
10	1.96	1.29	2.78	2.11	0.95	0.28	**-0.39**	**-1.06**
10.6	0.95	1.59	1.85	2.49	-0.09	0.55	**-1.43**	**-0.79**
11	1.91	1.8	2.79	2.68	0.91	0.8	**-0.45**	**-0.56**
12	1.49	1.86	2.35	2.72	0.53	0.9	**-0.84**	**-0.47**
13	2.35	2.01	3.2	2.86	1.42	1.08	0.06	**-0.28**
14	1.53	1.93	2.36	2.76	0.63	1.03	**-0.69**	**-0.29**
15	1.36	1.29	2.18	2.11	0.5	0.43	**-0.82**	**-0.89**
16	**-2.81**	2.31	**-2.01**	3.11	**-3.66**	1.46	**-4.95**	0.17
17.1	1.65	0.8	2.5	1.65	0.78	**-0.07**	**-0.53**	**-1.38**
17.2	2.15	1.06	3.1	2.01	1.23	0.14	**-0.09**	**-1.18**
18	1.89	2.47	2.82	3.4	0.97	1.55	**-0.42**	0.16
18.6	2.07	2.16	2.99	3.08	1.14	1.23	**-0.1**	**-0.01**
18.7	1.47	2.15	2.37	3.05	0.56	1.24	**-0.67**	0.01
19	0.71	1.68	1.6	2.57	-0.2	0.77	**-1.44**	**-0.47**
19.6	2.28	1.99	3.18	2.89	1.36	1.07	0.17	**-0.12**
20	1.35	2.33	2.27	3.25	0.43	1.41	**-0.76**	0.22
20.6	1.81	0.22	2.71	1.12	0.9	**-0.69**	**-0.25**	**-1.84**

Table G.6: Freeboards for the 1:10 flood event

G.2.2. Comparison of the water levels in the Klip River with the street levels of the stormwater valves

In the following tables the difference between the street levels of all stormwater valves around the CBD of Ladysmith and the water levels in the Klip River are listed for the past and future flood events. Each table contains all the values for one flood event. In each table the difference between these levels is listed for different Manning values. The difference is calculated by subtracting the water level in the Klip River from the street level of the stormwater valve.

$$Difference = h_{Klip} - streetlevel \tag{G.8}$$

A negative value in the table thus implies that the water level in the Klip river was higher than the street level of the stormwater valve. This means that the stormwater valve will be closed and is not able to discharge rainwater to the Klip River. In order to give a good overview, all negative values are bold.

The first and second column in the tables contain the name of the stormwater valve and the street level of this stormwater valve respectively. The third column contains the number of the cross-section in the HEC-RAS model that is closest to the stormwater valve. The last four columns contain the differences per stormwater valve for different Manning coefficients.

Past flood events

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	1.65	2.49	0.61	**-0.67**
C19	1000.5	8	2.15	2.99	1.11	**-0.17**
C17	1000	10	1.92	2.7	0.91	**-0.35**
C16	999.5	10	1.42	2.2	0.41	**-0.85**
C15	999.5	10	1.42	2.2	0.41	**-0.85**
C14	999	13	1.69	2.5	0.76	**-0.48**
C13A	998.5	13	1.19	2	0.26	**-0.98**
C13B	999.5	13	2.19	3	1.26	0.02
C13C	998	14	0.86	1.66	**-0.03**	**-1.21**
C13	999	14	1.86	2.66	0.97	**-0.21**
C12A	998.5	14	1.36	2.16	0.47	**-0.71**
C12	998	14	0.86	1.66	**-0.03**	**-1.21**
C11A	998	16	0.96	1.72	0.12	**-1.03**
C10	997.5	17.2	0.77	1.69	**-0.15**	**-1.34**
C9	997	18	0.42	1.31	**-0.51**	**-1.76**
C8	997	18.6	0.62	1.5	**-0.33**	**-1.53**
C7	997	19	0.66	1.52	**-0.27**	**-1.47**

Table G.7: Difference between Klip River water levels and street levels, flood event 1

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	2.21	3.06	1.24	**-0.06**
C19	1000.5	8	2.71	3.56	1.74	0.44
C17	1000	10	2.41	3.22	1.46	0.21
C16	999.5	10	1.91	2.72	0.96	**-0.29**
C15	999.5	10	1.91	2.72	0.96	**-0.29**
C14	999	13	1.92	2.77	1.01	**-0.19**
C13A	998.5	13	1.42	2.27	0.51	**-0.69**
C13B	999.5	13	2.42	3.27	1.51	0.31
C13C	998	14	1.02	1.87	0.14	**-1.03**
C13	999	14	2.02	2.87	1.14	**-0.03**
C12A	998.5	14	1.52	2.37	0.64	**-0.53**
C12	998	14	1.02	1.87	0.14	**-1.03**
C11A	998	16	1.08	1.91	0.22	**-0.92**
C10	997.5	17.2	0.74	1.65	**-0.15**	**-1.32**
C9	997	18	0.32	1.21	**-0.58**	**-1.78**
C8	997	18.6	0.41	1.29	**-0.49**	**-1.66**
C7	997	19	0.43	1.3	**-0.46**	**-1.63**

Table G.8: Difference between Klip River water levels and street levels, flood event 2

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	1.79	2.62	0.76	**-0.53**
C19	1000.5	8	2.29	3.12	1.26	**-0.03**
C17	1000	10	2.05	2.82	1.05	**-0.21**
C16	999.5	10	1.55	2.32	0.55	**-0.71**
C15	999.5	10	1.55	2.32	0.55	**-0.71**
C14	999	13	1.79	2.6	0.88	**-0.36**
C13A	998.5	13	1.29	2.1	0.38	**-0.86**
C13B	999.5	13	2.29	3.1	1.38	0.14
C13C	998	14	0.96	1.75	0.08	**-1.11**
C13	999	14	1.96	2.75	1.08	**-0.11**
C12A	998.5	14	1.46	2.25	0.58	**-0.61**
C12	998	14	0.96	1.75	0.08	**-1.11**
C11A	998	16	1.05	1.81	0.22	**-0.93**
C10	997.5	17.2	0.85	1.76	**-0.06**	**-1.25**
C9	997	18	0.5	1.38	**-0.42**	**-1.67**
C8	997	18.6	0.68	1.56	**-0.25**	**-1.46**
C7	997	19	0.72	1.57	**-0.2**	**-1.4**

Table G.9: Difference between Klip River water levels and street levels, flood event 3

Future flood events

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	**-0.87**	0.18	**-1.99**	**-3.29**
C19	1000.5	8	**-0.37**	0.68	**-1.49**	**-2.79**
C17	1000	10	**-0.66**	0.42	**-1.78**	**-3.08**
C16	999.5	10	**-1.16**	**-0.08**	**-2.28**	**-3.58**
C15	999.5	10	**-1.16**	**-0.08**	**-2.28**	**-3.58**
C14	999	13	**-0.97**	0.17	**-2.06**	**-3.41**
C13A	998.5	13	**-1.47**	**-0.33**	**-2.56**	**-3.91**
C13B	999.5	13	**-0.47**	0.67	**-1.56**	**-2.91**
C13C	998	14	**-1.82**	**-0.69**	**-2.89**	**-4.21**
C13	999	14	**-0.82**	0.31	**-1.89**	**-3.21**
C12A	998.5	14	**-1.32**	**-0.19**	**-2.39**	**-3.71**
C12	998	14	**-1.82**	**-0.69**	**-2.89**	**-4.21**
C11A	998	16	**-1.78**	**-0.68**	**-2.79**	**-4.06**
C10	997.5	17.2	**-1.95**	**-0.73**	**-3.01**	**-4.34**
C9	997	18	**-2.43**	**-1.17**	**-3.48**	**-4.82**
C8	997	18.6	**-2.09**	**-1.03**	**-3.1**	**-4.46**
C7	997	19	**-2.09**	**-1.05**	**-3.07**	**-4.42**

Table G.10: Difference between Klip River water levels and street levels, 1:100 flood

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	**-0.15**	0.81	**-1.29**	**-2.51**
C19	1000.5	8	0.35	1.31	**-0.79**	**-2.01**
C17	1000	10	0.13	1.03	**-1.06**	**-2.27**
C16	999.5	10	**-0.37**	0.53	**-1.56**	**-2.77**
C15	999.5	10	**-0.37**	0.53	**-1.56**	**-2.77**
C14	999	13	**-0.13**	0.83	**-1.32**	**-2.62**
C13A	998.5	13	**-0.63**	0.33	**-1.82**	**-3.12**
C13B	999.5	13	0.37	1.33	**-0.82**	**-2.12**
C13C	998	14	**-0.97**	**-0.02**	**-2.14**	**-3.41**
C13	999	14	0.03	0.98	**-1.14**	**-2.41**
C12A	998.5	14	**-0.47**	0.48	**-1.64**	**-2.91**
C12	998	14	**-0.97**	**-0.02**	**-2.14**	**-3.41**
C11A	998	16	**-0.91**	0	**-2.04**	**-3.26**
C10	997.5	17.2	**-1.1**	**-0.08**	**-2.28**	**-3.52**
C9	997	18	**-1.54**	**-0.5**	**-2.74**	**-3.99**
C8	997	18.6	**-1.32**	**-0.35**	**-2.4**	**-3.61**
C7	997	19	**-1.29**	**-0.36**	**-2.36**	**-3.56**

Table G.11: Difference between Klip River water levels and street levels, 1:50 flood

Storm valve	Streetlevel [m]	Cross section	RHDHV	Low	Medium	High
C20	1000	8	1.36	2.22	0.3	**-1.01**
C19	1000.5	8	1.86	2.72	0.8	**-0.51**
C17	1000	10	1.62	2.44	0.61	**-0.73**
C16	999.5	10	1.12	1.94	0.11	**-1.23**
C15	999.5	10	1.12	1.94	0.11	**-1.23**
C14	999	13	1.39	2.24	0.46	**-0.9**
C13A	998.5	13	0.89	1.74	**-0.04**	**-1.4**
C13B	999.5	13	1.89	2.74	0.96	**-0.4**
C13C	998	14	0.57	1.4	**-0.33**	**-1.65**
C13	999	14	1.57	2.4	0.67	**-0.65**
C12A	998.5	14	1.07	1.9	0.17	**-1.15**
C12	998	14	0.57	1.4	**-0.33**	**-1.65**
C11A	998	16	0.66	1.46	**-0.19**	**-1.48**
C10	997.5	17.2	0.47	1.42	**-0.45**	**-1.77**
C9	997	18	0.11	1.04	**-0.81**	**-2.2**
C8	997	18.6	0.3	1.22	**-0.63**	**-1.87**
C7	997	19	0.34	1.23	**-0.57**	**-1.81**

Table G.12: Difference between Klip River water levels and street levels, 1:10 flood

G.3. Hand calculation backwater curve

Hand calculations on backwater curves in the river system are made to get a feeling for how the water levels change during flood events. Additionally, these calculations are used to compared to the results of the HEC-RAS model. The calculations in this section are based on the theory from the lecture notes of the course CIE4345: River Engineering given at the Delft University of Technology [8].

To perform calculations on backwater curves in the river system, it is assumed that the Klip River has a rectangular cross-section with a width of 50 meter. The flow is a subcritical flow. The river reach upstream of the knickpoint, indicated as KP in Figure G.2, has an average slope i_b of $1:200$. Downstream of the knickpoint the slope of the river becomes less steep : $i_b = 1:4000$. Furthermore, a friction coefficient c_f of 0.035 is used for the upstream reach and 0.018 for the downstream reach. These values are based on the medium Manning roughness coefficients, derived in Section F.2. The section of the Klip River that is considered is joined by 3 tributaries: the Flagstone Spruit, Bell Spruit and Fourie Spruit. Lengths between the tributaries and other important nodes are given in Table G.13. The flood events with associating discharges in Table G.14 are used in the hand calculations on backwater curves.

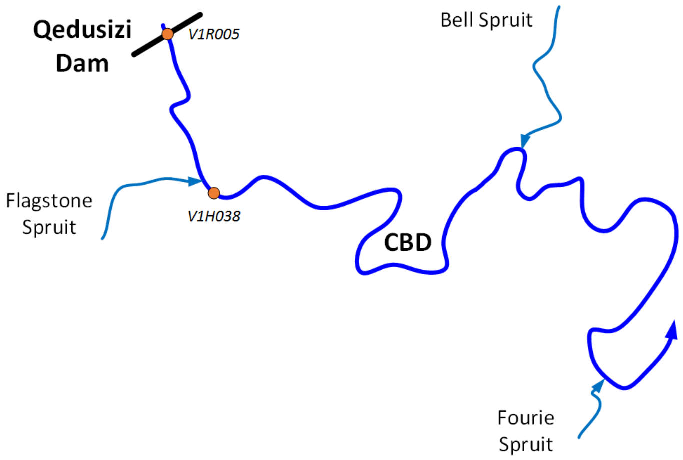


Figure G.2: Schematic overview Klip River and tributaries

Klip River section	**Length** $[m]$
Qedusizi Dam – Flagstone Spruit	2500
Flagstone Spruit – Knickpoint	2500
Knickpoint – Bell Spruit	5000
Bell Spruit – Fourie Spruit	8500

Table G.13: Riversections including lengths

	Discharge Q $[m^3/s]$			
Klip River section	*Dec 2001*	*Jan 2011*	*Jan 2012*	*Sept 2012*
Q1: Qedusizi Dam - Flagstone Spruit	174.6	269.7	3.4	237.9
Q2: Flagstone Spruit - Bell Spruit	187	331.6	241.4	314.6
Q3: Bell Spruit - Fourie Spruit	196	378	419.8	372.1
Q4: Downstream Fourie Spruit	203	411.4	548.4	413.5

Table G.14: Flood events and discharges in the Klip River

Calculations on backwater curves start with determining the normal flow depth d_e and critical flow depth d_g. These are calculated with Equation G.9 and Equation G.10. The acquired normal depths for the recent flood events are listed in Table G.15. Since $c_f > i_b$ the Klip River can be characterised to have a mild bed slope (M).

$$d_e = \left(\frac{c_f \cdot q^2}{i_b \cdot g}\right)^{1/3} \tag{G.9}$$

$$d_g = \left(\frac{q^2}{g}\right)^{1/3} \tag{G.10}$$

c_f	= friction coefficient	$[-]$
q	= specific discharge	$[m^2/s]$
i_b	= bed slope	$[-]$
g	= gravitational acceleration	$[m/s^2]$

	Normal flow depth d_e $[m]$			
Klip River section	*Dec 2001*	*Jan 2011*	*Jan 2012*	*Sept 2012*
Qedusizi Dam - Flagstone Spruit	2.03	2.71	0.15	2.49
Flagstone Spruit - Knickpoint	2.12	3.11	2.51	3.00
Knickpoint - Bell Spruit	4.75	6.96	5.63	6.72
Bell Spruit - Fourie Spruit	4.91	7.59	8.14	7.51
Downstream Fourie Spruit	5.02	8.03	9.73	8.06

Table G.15: Flood events and normal flow depths in the Klip River

Then, the empirical fit to Bresse was used to calculate the water depth at different sections along the river. Water depths are calculated from downstream in upstream direction. The half length $L_1/2$ gives an indication for how fast the water level converts to the normal water depth, see Table G.16.

$$d(x) = d_e + (d_0 - d_e)2^{\frac{x-x_0}{L_{1/2}}} \tag{G.11}$$

$$L_{1/2} = 0.24\frac{d_e}{i_b}\left(\frac{d_0}{d_e}\right)^{4/3} \tag{G.12}$$

d_0	= water depth downstream of the backwater curve	$[m]$
$L_{1/2}$	= half length	$[m]$

	Half length $L_{1/2}$ $[m]$			
Klip River section	*Dec 2001*	*Jan 2011*	*Jan 2012*	*Sept 2012*
Qedusizi Dam - Flagstone Spruit	99	156	334	153
Flagstone Spruit - Knickpoint	292	462	547	459
Knickpoint - Bell Spruit	5006	8114	10587	8173
Bell Spruit - Fourie Spruit	5055	8187	10327	8253

Table G.16: Half lengths per flood event

An overview of the calculated water depths at specific locations during flood events is given in Table G.17. Furthermore, Figure G.3 shows the discharge and backwater curves for the flood event of September 2012.

	Water depth *d [m]*			
Location in Klip River	*Dec 2001*	*Jan 2011*	*Jan 2012*	*Sept 2012*
Qedusizi Dam	2.03	2.71	0.16	2.49
Flagstone Spruit tributary	2.13	3.21	2.75	3.10
Knickpoint	4.85	7.51	8.09	7.41
Bell Spruit tributary	4.94	7.81	9.04	7.78
Fourie Spruit tributary	5.02	8.03	9.73	8.06

Table G.17: Calculated water depths for different locations

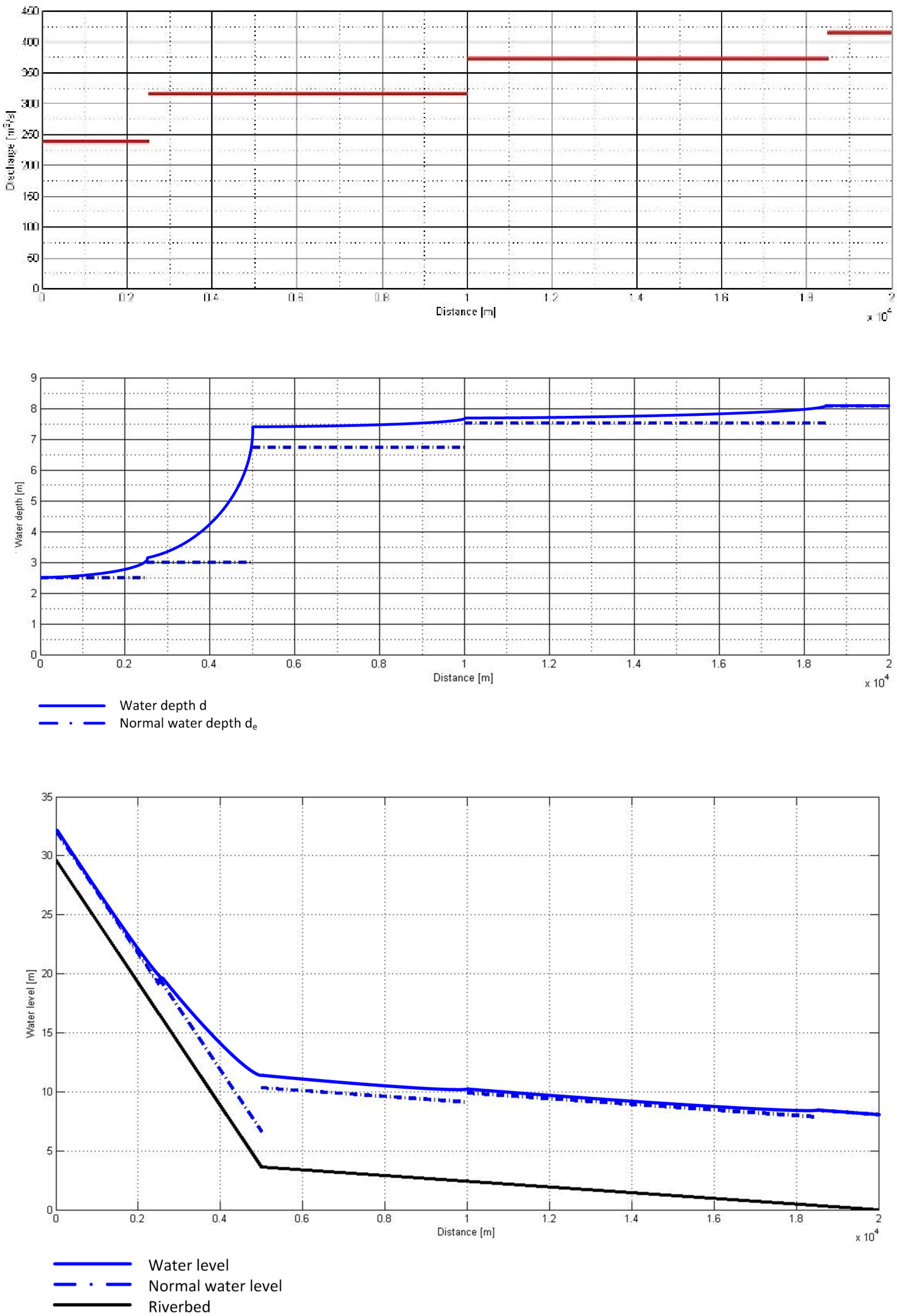


Figure G.3: Backwater curve September 2012

H

SWMM

After an overview of SWMM in Section 5.3, a more detailed description of the model is given in this appendix. The appendix will start with the assumptions in SWMM, followed up by a description of determining the characteristics of the area.

H.1. Assumptions in SWMM

There are many standard characteristics of a drainage system in SWMM. Those characteristics are already described in the project summary in Section 5.3. The assumptions that are done regarded to each separate variable are:

- **Raingage**: The data of the raingage near the Ladysmith Aerodome Airfield, approximately 3 kilometres south-west of the CBD, is used.
- **Subcatchment**: The assumptions regarding the subcatchments are described in Section H.2.
- **Aquifers**: No information available about the presence of aquifers, so set as zero.
- **Snowpacks**: No snow in this area.
- **RDII Hydrographs**: This characteristics concerns the rainfall-dependent infiltration or inflow into the sewer system [31]. There is no data available about this infiltration/inflow, that is why it is set as zero. Moreover, it is partly included in the characteristics of the subcatchments.
- **Infiltration**: Infiltration is the amount of precipitation which will infiltrate in the unsaturated soil layers. Also this factor is included in the characteristics of the subcatchments.
- **Junction nodes**: No assumptions about the junction nodes.
- **Outfall nodes**: No assumptions about the outfall nodes.
- **Flow divider nodes**: Not used in the model, because of the absence of the ratio deviation.
- **Storage unit nodes**: Not enough information about the potential storage in the system. During the field trip, no storage units have been observed. So it is assumed to be zero.
- **Conduit links**: No assumptions about the conduit links. So, assumed is that they are not present in the system.
- **Orifice links**: The drainage map [12] which is used is not detailed enough to indicate where the orifice links are. So the model only consist of conduits, junctions and outfalls.
- **CMS**: CMS is about the dimensions of the model: Cubic, Meters and Seconds.
- **Flow Routing**: Flow routing in SWMM is based on the conservation of mass and the momentum balance [31]. There are three available options for flow routing in SWMM: steady flow routing, kinematic

flow routing and dynamic wave routing. The model of the drainage system of Ladysmith makes use of the kinematic flow routing. The steady flow routing method is not able to indicate nodes that have flooded. The dynamic wave routing cannot be used, because that method requires a small time step (6 - 12 minutes). The data is not available in such a small time frame. This type of flow routing is able to indicate flooded nodes. An assumption in this method is about the slope of the water surface, which is equal to the slope of the junctions.

- **Control Rules**: The control rules are used to close the stormwater valves at a certain moment. This process is described in Section 5.3.
- **Time Series Inflows**: Time series inflows is about external inflows besides the inflow through precipitation. No information about this is available, so it is assumed to be zero. The previous flood events are analysed in the model. Most probably, the external inflows will be very small compared to the precipitation during these events.
- **Dry Weather Inflows**: Not relevant during this study, so assumed to be zero. It is not relevant, because during the flood events that were studied, there was no dry weather situation.
- **Groundwater Inflows**: Groundwater inflow is about the water that will flow from the drainage system into the ground. Also no data is available about this variable. Most probably the groundwater inflow during a flood event is negligible compared to the total flow in the system. That is why it is assumed to be zero.
- **RDII Inflows**: This inflow is about water that will leak due to, for example, leaking pipes. This variable will be the same for all events and will affect the flood events in the same way. That is why it is disregarded.
- **Treatment units**: The treatment is out of the scope of this project.
- **Pollutants**: The treatment is out of the scope of this project.
- **Outlet links**: No assumptions about the outlet links.
- **Weir links**: No weirs in this drainage system.

H.2. Characteristics of the subcatchments

The subcatchments of the CBD are determined by using the elevation contour lines of the drainage map [12]. The water will flow from a higher contour line to a lower contour line. Structures in the area will influence the flow direction of the water. This makes it complex to determine the subcatchments taking into account all those structures. SWMM needs a number of characteristics of the subcatchments to model the drainage of a rainfall event. The characteristics are: area, width, slope, percentage of impervious area, Manning coefficient, Dstore and Zero-Impervious. An explanation and the process of determining those characteristics is given in the following subsections.

H.2.1. Area, width and slope

The area, width and slope are easily determined, using Google Earth^{TM} and the drainage map. The area is just about the surface of a subcatchment in ha. The width is the largest distance(in m) a water particle has to travel in the area. This factor is determined by using the ruler-function in Google Earth^{TM}. The slope is determined by the difference in height in a sub catchment divided by the width. Giving an average percentage of the slope as a result.

H.2.2. Impervious area

The impervious area, is the area in a (sub)catchment in which no water will infiltrate. Water will not infiltrate easily in urban areas. So, Google Earth^{TM} is used to make an estimation of which percentage of a catchment is built-up, and which not.

H.2.3. Manning-Coefficient

The Manning coefficient n estimates the average flow velocity towards the drainage system. It is not only influenced by whether the area is pervious or impervious. The type of pervious or impervious area is also important. Again, Google Earth^{TM} is used to determine the types of pervious and impervious area, and the percentage of those types in the subcatchment. The characteristics of the area are linked to the typical values of Table H.1 [31]. SWMM makes a distinction between N-impervious and N-pervious.

Surface	**n**
Asphalt	0.011
Buildings	0.014
Fallow soils	0.05
Cultivated soils	0.06
Short grass	0.15
Dense grass	0.24
Threes	0.6

Table H.1: Manning's coefficient overland flow

An example:

	Characteristics	**n**
Impervious	60% asphalt, 40% constructions	0.6 * 0.011 + 0.4 * 0.014 = 0.0128
Pervious	30% trees, 70% short grass	0.3 * 0.06 + 0.7 * 0.15 = 0.285

Table H.2: Example of determining the Manning coefficient

H.2.4. Depression storage

The depression storage of the area, Dstore, is the ability of the system to hold water in pits and depressions. This depends on the type of coverage of the area. The coverage types are determined by using Google Earth^{TM}. Also for the Dstore the cover types are linked to typical values of Table H.3 [31][30]. Asphalt does not have a depression coefficient [26]. In SWMM this is the percentage of the impervious area with no depression storage (Zero-Impervious) [31]. It is a separate factor in SWMM, so it is not included in the calculation for depression storage of the impervious area.

Surface	**Dstore** mm
Asphalt	0
Slooped roofs	1.27
Flat roofs	2.54
Fallow soils	1.27
Cultivated soils	1.4
Short grass	2.54
Dense grass	5.08
Threes	7.62

Table H.3: Depression storage

	Characteristics	**Dstore**
Impervious	60% asphalt and 40% constructions, in which 60% flat roofs and 40% slooped roofs	0.4 * 1.27 + 0.6 * 2.54 = 2.032
Pervious	30% trees, 70% short grass	0.3 * 7.62 + 0.7 * 2.54 = 4.064

Table H.4: Example of determining the depression storage

Glossary

bedform a feature that develops at the interface of fluid and a moveable bed, the result of bed material being moved by fluid flow.. 13

evaporation is the process of transferring water from a liquid state into a gaseous state.. 11, 71

interception The water which do not reach the root-zone or the drainage system, because of evaporation.. 12, 45, 71

intermittent river A semi-permanent river, which is characterized by seasonal flow. Normally having a flow during wet periods in contrast with hardly any or no flow in dry periods.. 3

knickpoint A knickpoint is defined as a relatively abrupt change in slope over a longitudinal stream. 5, 27

main stem The primary downstream segment of a river, as contrasted to its tributaries. 2

pool Section of a stream with reduced current velocity, often with deeper water than surrounding areas and with a smooth surface.. 5

riffle Section of a stream, usually more shallow than reaches up- and downstream, with rapid current and a surface broken by gravel, rubble or boulders.. 5

Sinuosity Index The Sinuosity index measures the deviation of a line from the shortest path, calculated by dividing the total length by the shortest possible path. Giving an indication on the degree of meandering of a river.. 3

Strahler Method The most common stream ordering method. Streams without any tributaries are assigned as first order. Stream orders increase when they are adjoined by a same order stream.. 3

stream order Stream ordering is a method of assigning a numeric order to links in a stream network. This order is a method for identifying and classifying types of streams based on their numbers of tributaries.. 3

subcatchment a subcatchment is the area in which the water will flow to a certain manhole or another subcatchment.. 32, 33, 96

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