ANALYSIS OF DRAINAGE SYSTEM IN GEORGETOWN, GUYANA

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University of Guyana
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This project was executed with the goal of improving the capability of knowledge based decision making regarding the drainage system in Georgetown, Guyana. The results have to be used likewise and not as a basis for direct measures on the system.

PREFACE

In the past and most recently in 2015 Georgetown, the capital of Guyana, suffered from severe flooding during high intensity rainfall. After the 2015 flood a Dutch Risk Reduction team (DRR Team) visited Georgetown on the request of the Government of Guyana and made a first analysis of the flood vulnerability of Georgetown and the coastal lowlands of Guyana. Recommendations were formulated by the DRR team to improve the flood resilience of the city on the short and long term, as well as interventions to increase local capacity that will be required to deal with future water management challenges.

As a very first step a team of seven MSc Civil Engineering students from Delft University of Technology, the Netherlands, started a project in the summer of 2016 based on the recommendations of the DRR Team. The project work and its corresponding results are presented in this report. It consists of three analyses, which all together describe the process from rainfall to discharge out of the capital's urban drainage system.

We want to thank our supervisors from the Netherlands (prof. Winterwerp, ir. Pasterkamp from Delft University of Technology and ir. Westebring and ir. Steijn from the DRR team). Besides that the representatives from the Dutch Embassy in Paramaribo deserve much credit for helping organise this project. Also we want to express our gratitude to all the motivated persons which were involved from several authorities and institutions in Guyana (National Task Force, Ministry of Agriculture, Ministry of Public Infrastructure, Mayor and City Council and University of Guyana). Last but not least we would like to thank our protocol officer Christine Mohammed Douglas for her great help in organizing a variety of events and Mr. Pierce, our driver. Without their extraordinary effort the project would have looked very different.

ABSTRACT

In 2015 Georgetown, Guyana suffered from major flooding due to heavy rainfall. The use of a centuries-old agricultural drainage system for the urban drainage of the largest urbanized area of Guyana, poses problems considering flood safety. In 2016 a report was published by a 'Dutch Risk Reduction Team' (DRR Team) with recommendations on how to reduce the current flood vulnerability. Based on the recommendations from this DRR report. A team of seven students from the Delft University of Technology, the Netherlands, went to Georgetown and analysed the drainage system in more detail. Several methods were developed in collaboration with local students and experts which can be used to analyse the system. This was done to increase the local capability of knowledge-based decision making on drainage issues in Guyana. This student's induced project comprises three elements of the urban drainage system: the primary drainage channels, the local (secondary and tertiary) drainage canals, and the outlet structures. The work focussed primarily on the catchment area named South-Ruimveldt.

The first analysis was made on the conversion of 'rainfall to discharge' into the primary channel by the local drainage system. The local drainage system consists of so called tertiary and secondary channels as well as culverts connecting them. Using a frequency analysis and different computed runoff coefficients, the rainfall intensities of a design storm can be translated into a direct discharge into the local drainage system. A model, based on the storage principle, which consists of basins and culverts, was developed to convert this input into outflow into the primary channels. The model can be used to assess the theoretical effect of different scenarios such as the addition of extra culverts or dredging the channels. Also this model gives an insight in the delay of discharge which occurs in the local drainage system.

Secondly the primary drainage channels were assessed. This was done using the one dimensional hydraulic model HEC-RAS. Using the input from the local drainage system analysis, measured geometric data and simplified boundary conditions a full model of the South-Ruimveldt catchment was developed. This model can be used to assess and compare the effect of different interventions. Eight examples were given in this report which include cleaning of the channels and adding a storage area. In addition to this model a methodology was developed to understand the model, improve the model and implement the approach in other areas.

Finally the outfall sluices, locally called kokers, which discharge water from the primary channels out of the system into the Demerara river were analysed. This resulted in the development of a 'structural assessment tool' which can be used for prioritizing maintenance actions. In addition to this tool several manuals and forms were made to make the use of the tool as easy as possible for local experts.

Together these three analyses, their corresponding tool, manuals and this final report form the result of the students project.

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1 INTRODUCTION

In the beginning of this chapter the motivation (1.1), project goal (1.2), team members (1.3) and involved stakeholders (1.4) will shortly be described. Afterwards an outline of the report is given (1.5).

1.1 Motivation

After the severe flooding in July 2015 the Government of Guyana sent a request to the Government of the Netherlands to advice on the condition of their drainage system. Following this request a Dutch Risk Reduction Team (DRR Team) visited Guyana and Georgetown in particular. This team of three experts formulated eight extensive recommendations for improvement of the flood resilience (Steijn, Westebring, & Klostermann, 2016). Using different parts of these eight recommendations the student project was set up. In APPENDIX C a more extensive overview of the (used) DRR- recommendations are given. During the process of making the final report of the mission in 2015 a multidisciplinary team of Dutch students from Delft University of Technology expressed their keen interest to help out fulfil some of these recommendations. This report was made by this team of student.

Table 1 Overview recommendations DRR Team 2015

	Recommendation	Included in this project (yes/no)
1	Upgrade modelling capability	Yes
2	Improve flood resiliency of people	No
3	Upgrade small-scale floating dredging capabilities	No
4	Develop and apply a rational risk approach	Yes, briefly
5	Pilot 'Living with Water'	No
6	Asset Management	Yes
7	Data Management	Yes
8	Technical short-term improvements	Yes

1.2 Project goal

The student team spent over six months preparing for the mission. In these months they formulated a clear project plan in cooperation with the DRR Team, Guyanese partners and Delft University of Technology. The overall goal of the team was:

'Development of knowledge and material for the local authorities to make knowledge based decisions regarding interventions to improve the drainage system and reduce the flood risk in Georgetown, Guyana.'

To achieve this goal the team worked on an example area which was used to develop both the knowledge on the system and material to analyse it. This area is the catchment area of South-Ruimveldt with its corresponding local drainage system and primary channel. At the same time structural engineers worked on the kokers (outfall sluices) discharging water on the Demerara river. Using these case studies, different methods were developed to create more insight and understanding of the system.

1.3 Group composition

The team consists of seven students of which five are executing this project as a part of their master study.

Table 2 Student team composition

Name	MSc Track	Project DUT
Joost Remmers	Hydraulic Structures and Flood Risk (HE)	Yes
Ruben van Montfort	Hydraulic Structures and Flood Risk (HE)	Yes
Jos Muller	Coastal Engineering (HE)	Yes
Thijmen Jaspers Focks	Structural Mechanics (SE)	Yes
Peter Vijn	Structural Mechanics (SE)	Yes
Martijn van Wijngaarden	Hydraulic Engineering and Water Resource Management (HE)	No
Siebe Dorrepaal	Hydraulic Structures and Flood Risk (HE)	No

1.4 Involved stakeholders

The project has multiple partners involved from both the Netherlands as well as Guyana. Below their names, abbreviations and their content-wise role in the project are given. In APPENDIX A more detailed contact information is given.

Table 3 Involved stakeholders

Partner	Abbreviation	Country	Role
Delft University of Technology	TU Delft	NL	Supervisors
RVO – DRR Team	DRR Team	NL	Initiators + Advisors
Coasts, Deltas and Rivers	CDR	NL	Sponsor
National Task Force	NTF	GUY	Local initiator
Ministry of Agriculture	MoA	GUY	Local partner
National Drainage and Irrigation Authority	NDIA	GUY	Local partner
Ministry of Public Infrastructure	MOPI	GUY	Local partner
University of Guyana	UG	GUY	Local partner
Mayor and City Council Georgetown	M&CC	GUY	Local partner

The supervisors from Delft University of Technology are prof. dr. ir. Winterwerp who has experience working in Guyana and ir. Pasterkamp who is specialized on the structural assessments. Two team members of the DRR Team (Mr. Westebring and Mr. Steijn) functioned as advisors during the project.

1.5 Outline report

After the introduction (1) a system analysis will be made (2). This includes the description of the hydraulic system in and around Georgetown and additional information on knowledge-based decision making.

Based on the system analysis (2) the report elaborates on the three components of the drainage system. The first component is the local drainage system (3) which is the drainage process from rainfall to the discharge on the primary drainage channels. The second component is the behaviour of the primary channels (4) which describes the discharge of the local inflow to the outer water bodies (river and sea). The final component is the functioning of the outfall kokers at the end of the primary channels (5).

During the project extensive fieldwork was done and multiple long interviews were held with representatives and stakeholders. In this process several other observations on the system as a whole were made and these are included in a separate chapter (6). Finally the results of all the analysis are discussed (7) and some relevant recommendations are presented (9).

2 SYSTEM ANALYSIS

In this chapter the drainage system of Georgetown will be described. First the location (2.1), functioning (2.2) and historical developments (2.3) of the drainage system are described. Afterwards an introduction will be given to knowledge-based decision making (2.4) and a failure tree will be given (2.5).

2.1 Location

Georgetown, the capital of Guyana, is a city built on an area and around a drainage system which was originally used for the drainage of sugar cane plantations. Georgetown's northern boundary is the Atlantic Ocean and western boundary is the Demarara river. One of the main characteristics of Georgetown is that its topographical height lies below mean sea level (Icaros Geosystems B.V., 2010). To prevent flooding water has to be discharged from the area on a regular basis. Originally the area was divided in several catchments which each drained on its own koker via a primary channel. In this process the outfall kokers can only discharge water during low tide (Halcrow, 1994).

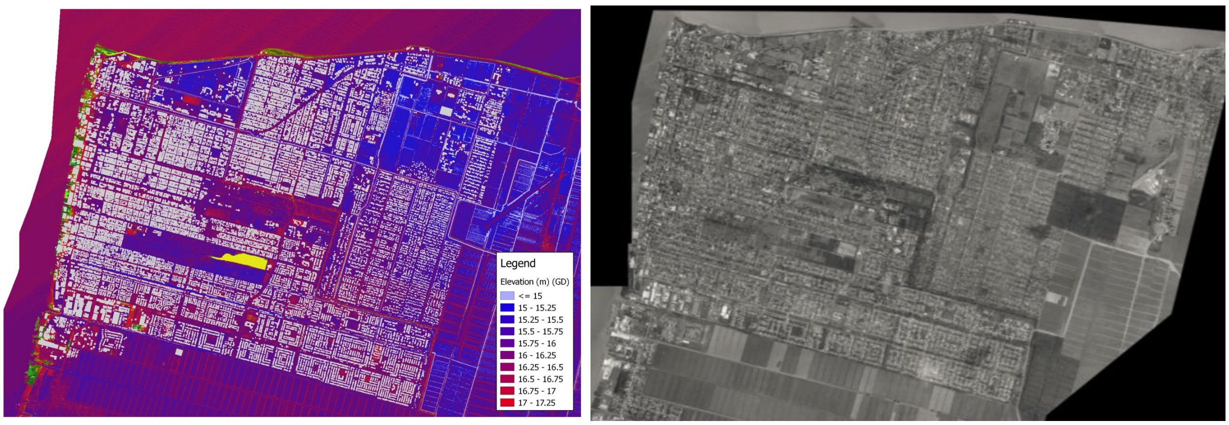


Figure 1 LIDAR (left) (Icaros Geosystems B.V., 2010) and high resolution satellite (right) (Icaros Geosystems B.V., 2010) picture

The case studies in this report are mainly on the catchment area of South-Ruimveldt. This is a catchment area located in the south of Georgetown and consists of living areas but also of main industry terrains. Georgetown is divided in several catchment areas and they all function as parts of the total drainage system.

Figure 2 Catchment areas with location of South-Ruimveldt (Openstreetmap, 2016)

2.2 Functioning of drainage system

The functioning of the drainage system in the western sectors of Georgetown (discharging water into the Demarara River) is based on gravity. When the water level at one location in a channel is higher than at another location in a channel, it starts flowing. At some places, the drainage is done with pumps, such as the catchment areas that discharge their water into the sea.

When it rains, the water is equally divided over the surface area. Some part of the water directly falls into a drain. However, the major part falls on buildings, roads, green areas or paved surfaces. From here, the water follows its course towards and into a drain. Water flows from the roofs of the houses into a garden or is by a rain pipe directly transferred into a drain. Most gardens consist either of soil (with vegetation on it) or concrete. They need to be built under an angle and above the top level of the drains, so the water can flow out into the drains by gravity. The same holds for roads.

When the water is in a drain, it has to find its way to either the Demerara River or the Atlantic Ocean. It has to flow from the local channels (secondary and tertiary channels) into a primary channel that discharges via the kokers on the river or the ocean.

The first requirement is that the local channels are connected to the primary channel. Without working connections, flow between the channels is not possible. The larger the water level difference between the local channels and the primary channel is, the quicker the water tends to flow.

There are two ways to discharge the water from the primary channels into the river or ocean: either by gravity (opening outfall kokers when the water level in the primary channel is higher than in the Demerara River) or by pumping (this can take place anytime). A lower water level close to the koker, compared to further upstream, leads to a flow of water.

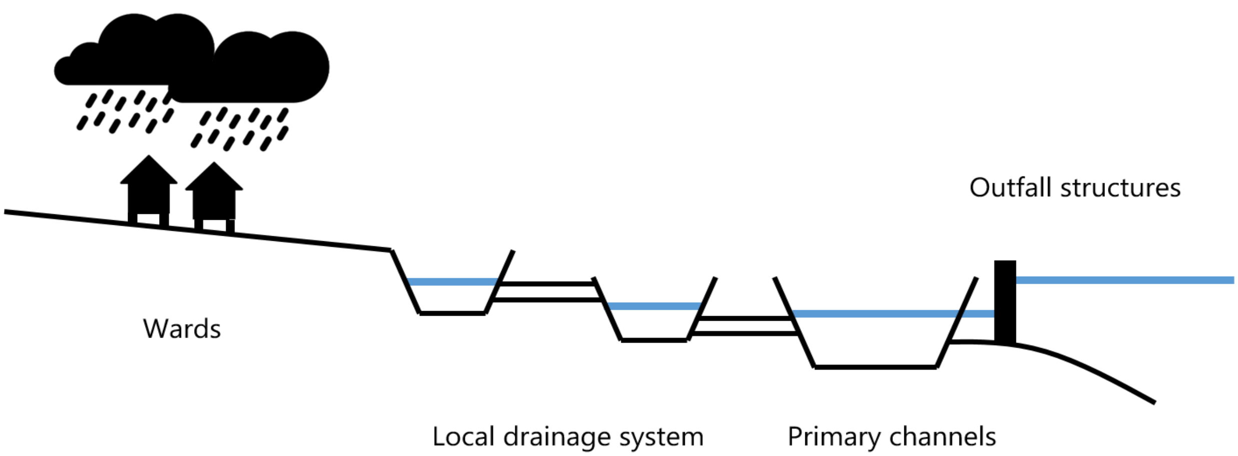


Figure 3 Schematization of functioning drainage system Georgetown (Jos Muller, August 2016)

2.3 Historical developments

The historical developments of the drainage system are described in (Halcrow, 1994). From this report it can be read that:

"*The city of Georgetown is drained by an interconnected network of pumped and gravity systems. The network was originally developed to irrigate and drain the sugar plantations which occupied the land on which Georgetown now stands. A further function of some of the channels was to provide a means of transportation, principally for the harvested cane which was taken from the fields, to the mill by barge.*

Urbanisation of the area, over the years, had two consequences for the irrigation and drainage system. Firstly, the higher level waterways were no longer required to function as irrigation and navigation waterways. Bridges were built to cross the channels. Secondly, paving of previously pervious surfaces resulted in increased quantities and rates of runoff. In an effort to alleviate flooding the function of the redundant higher level channels was converted to that of drainage. It is reported that this measure was not particularly successful as little attempt was made to regrade the canal beds."

Another aspect of urbanisation is related to littering. A lot of litter on the streets gets into the drainage channels, which leads to blockage of channel. There are rarely trash racks or other mechanisms to prevent litter from entering the system.

Blockage of the channels leads to a decrease in flow velocity. This has two consequences. Firstly, this gives vegetation the opportunity to grow. Secondly, when the flow velocity decreases, siltation takes place, leading to higher and rougher channel beds. As a consequence, the conveyance capacity decreases.

2.4 Knowledge-based decision making

At the moment, operation on the drainage system is mainly done based on experience. However, the system has become so complicated that experience-based choices seem not appropriate any more. The government wants to apply knowledge-based decision on the drainage system.

In order to do this, a structured and rational approach on decision making is needed. . Several approaches are possible. During the project elements of the 'integral design loop' was used (de Ridder, Integraal ontwerpen in de Civiele Techniek, 2009). It prescribes some steps to go from a problem to a decision, following a logical path. It is described below.

The integral design loop is a way which is often used to find the best solution of a problem in a rational and structured way. It starts with a problem. For instance, a primary channel in Georgetown often floods. The drainage problem is split up into three parts. Each part will focus on certain components of the integral design loop (Table 17). One needs to pass five 'steps' to be able to make a knowledge based decision. The procedure is as follows:

1. Analysis

The first step of the loop is the analysis. In this step, the scope is determined. The causes of the problem and the characteristics of the system are worked out. An approach that is often used in Dutch hydraulic engineering is the so called 'Flood Risk Analysis'. This is a structured way to determine the risk of a flood event happening in a certain area. This will be further worked out in APPENDIX E.

For instance, one determines the geometry of the channel by satellite maps and measuring during fieldwork. The inflow from the adjacent channels and the tide on the riverside are studied.

2. Synthesis

The second step is the synthesis. In this step, some possible measures to solve or reduce the problem are generated. These are based on the failure mechanisms of the analysis.

For instance, one determines two possible solutions: method 1: deepening the channel, method 2: creating a storage area.

3. Simulation

The third step is the simulation. This step focuses on the effects of the measures on the system. It needs to be clear what the main effects are but also what the side effects on the system are. Computations by hand or by computer models are required.

For instance, the impact of a proposed measure or intervention is computed with a computer model.

4. Evaluation

The fourth step is the evaluation. In this step, the measures are compared. There are different ways to assess which measure should be applied. The 'economical optimization', a useful tool to optimize investments, is worked out in APPENDIX E.

For instance, from the comparison it becomes clear that both methods have the same price. However, method 1 is more effective from a hydraulic perspective. In that case, method 1 is the preferred method.

5. Decision

When the evaluation is finished, a decision will be made. Such decision is often influenced by non-technical considerations as well and will not be elaborated on in the report.

For instance, an engineer presents his results to a policy-maker. The policy-maker decides to go for option 1. If, based on the evaluation, no decision can be made the loop goes back to the synthesis and new possible solution(s) have to be determined.

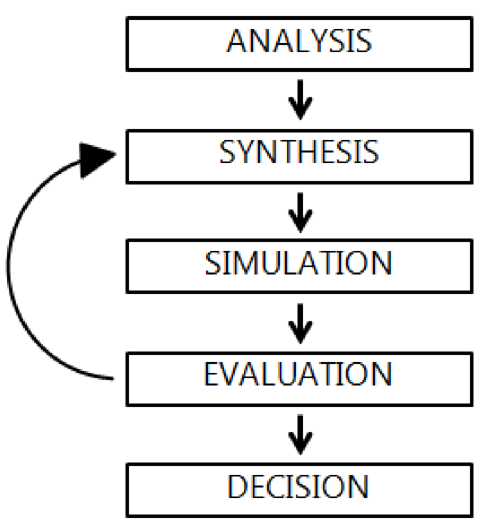


Figure 4 Schematization of integral design loop (de Ridder, 2009)

2.5 Failure tree

A flood occurs when in a certain area the inflow minus the outflow is larger than the storage. Floods can occur as a consequence of insufficient outflow in combination with insufficient storage. Inflow can be caused by rainfall but also as a consequence of local conditions (for instance the runoff). The rainfall is considered as a value that cannot be influenced. These local conditions are included in the analysis (Savenije, 2014).

Based on fieldwork (observations and interviews), existing literature (Halcrow, 1994) and discussions within the team, a general failure tree for floods due to rainfall in of Georgetown was made (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2015). This failure tree gives an overview which mechanisms contribute. It can easily be specified for a specific area in the city by carefully considering which mechanisms of the tree will be important in that area. Each failure mechanism can be solved with a measure which will be the focus of the other chapters.

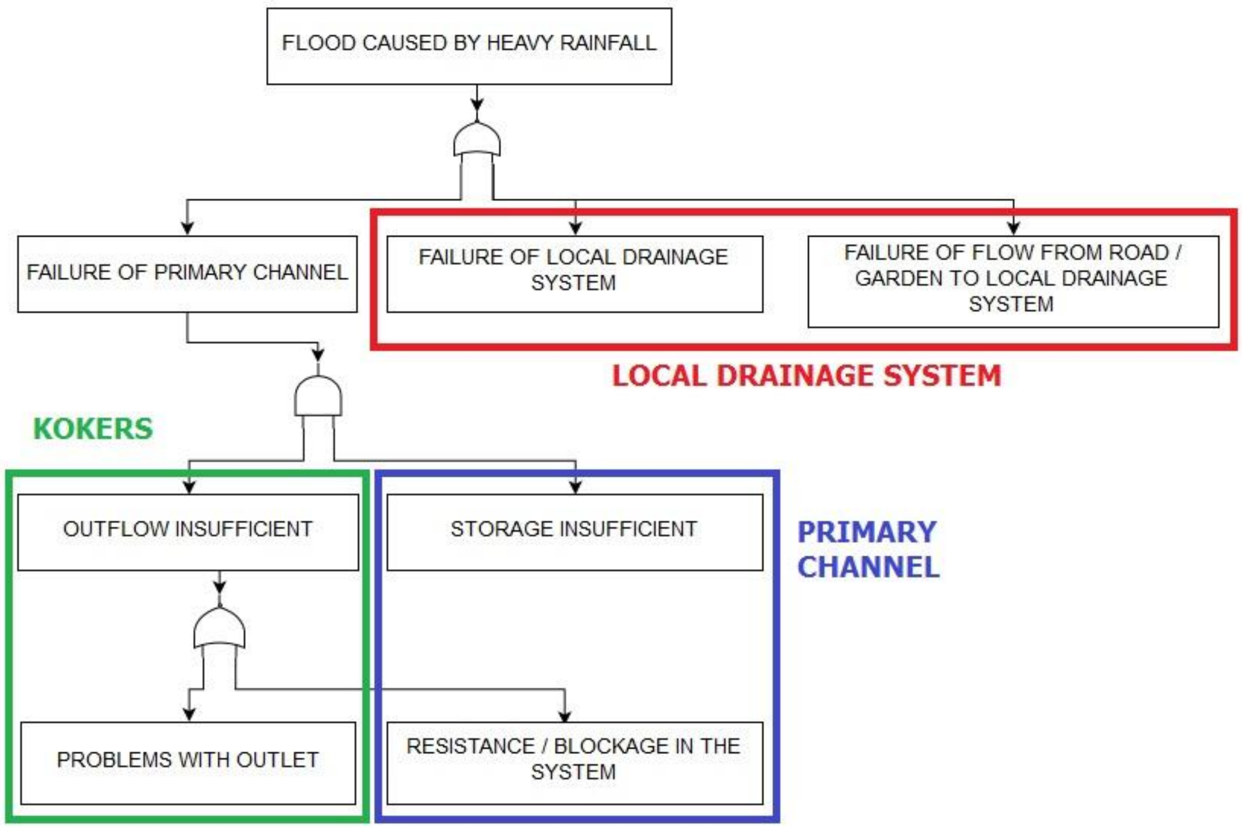


Figure 5 Overview of the failure tree with the different elements of the report (Ruben van Montfort, August 2016)

3 LOCAL DRAINAGE SYSTEM

The local drainage system is the part of the drainage system in Georgetown which receives the rainfall and transfers it to the primary discharge channels. In this chapter this process will be described by first giving an introduction on the local drainage system (3.1). Afterwards the hydrological analysis will be described which translates rain via runoff into discharge in the local drainage system (3.2). Then a case study on one specific local system (0) will be executed for which a model is developed (3.4). Finally the effect of some scenarios will be computed (3.5) and some remarks will be given on the analysis (3.6).

3.1 Introduction

The local drainage system forms an important part of the drainage system of Georgetown. A sufficient capacity in the primary channels on itself does not prevent floods to occur. Also, the underlying drainage system, the so called local drainage system, should be considered when one wants to diminish the amount of floods.

During the first session of fieldwork in South-Ruimveldt catchment area, it became clear that the local drainage system does not function as it should. Many drains are filled with vegetation, litter and silt. As a consequence, flow is hindered. Due to lack of outflow, the water level in the drains is relatively high, leading to limited storage capacity in the channels. During heavy rainfall the water cannot discharge and cannot be stored. Because of this floods occur.

3.2 Hydrological analysis

An essential part of the local drainage system analysis is the process between rainfall and discharge. This analysis was executed to gather input for both the local drainage system analysis and primary channels. It consists of a rainfall (3.2.1) and runoff (3.2.2) analysis. In this chapter a global overview of both will be given. In APPENDIX F more in depth information is available.

3.2.1 Rainfall analysis

The raw rainfall data required for the analysis was obtained from the meteorological station located in the Botanical Garden in the centre of Georgetown (HydroMet, 2016). With this data the daily and hourly rainfall statistics were computed. The daily rainfall return rate is given in Figure 6.

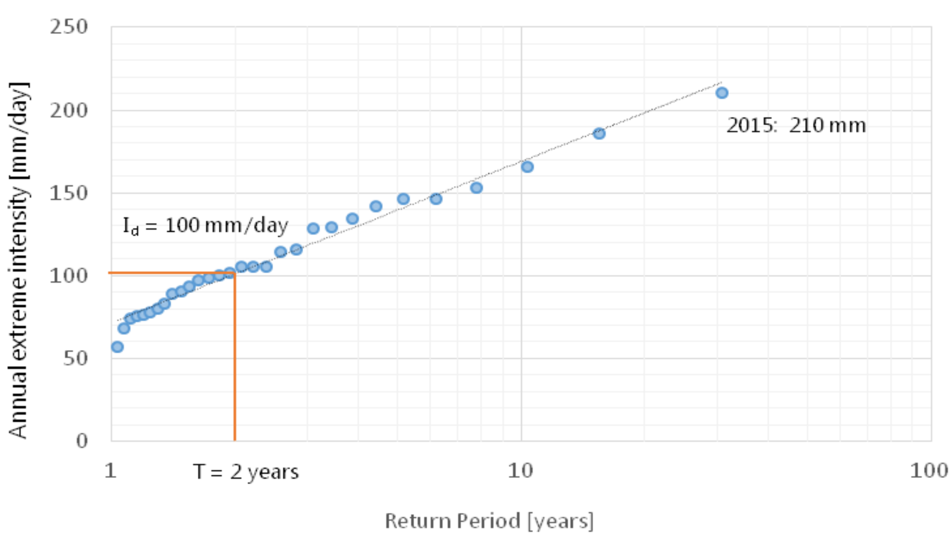


Figure 6 Return rate daily rainfall (mm/day) (Jos Muller, August 2016)

The design daily rainfall intensity is determined by means of a frequency analysis of the annually extreme daily rainfall intensity in millimetres per day. By collecting all the extreme daily rainfall data per year and ranking them in order of their maximum intensity, the probability of occurrence was calculated. The probability was transferred into a return period of each storm. This is the indication of the amount of years in which the expected intensity occurs, given the current record of annual extreme rainfall. Besides a probability of daily rainfall the average hourly rainfall profile was computed by assuming an average storm profile. This profile is given in Figure 7 and is explained in depth in Appendix F.

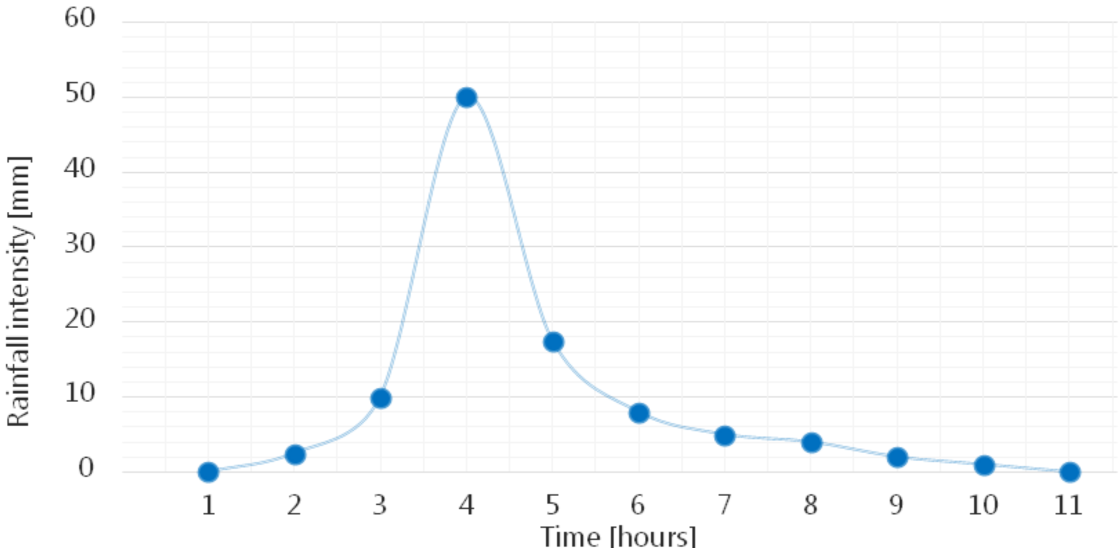


Figure 7 Average rainfall profile (mm/hr) (Jos Muller, August 2016)

3.2.2 Runoff analysis

Rainfall is translated to discharges into the primary channels by using a runoff coefficient. This runoff coefficient gives an indication of the amount of water directly discharged into the drainage system. Because the situation at stake in this report is storm water runoff all other hydrological processes than runoff, interception and storage can be left out of consideration due to their different time scales (Faculty of Civil Engineering (DUT), 2011). The total discharge of every catchment area enters the primary channel at different locations. To estimate these locations a filtered height map with contour lines was made with GIS (Persaud, 2012). From this map the catchment area was subdivided in small discharge areas (Openstreetmap, 2016) (Icaros Geosystems B.V., 2010).

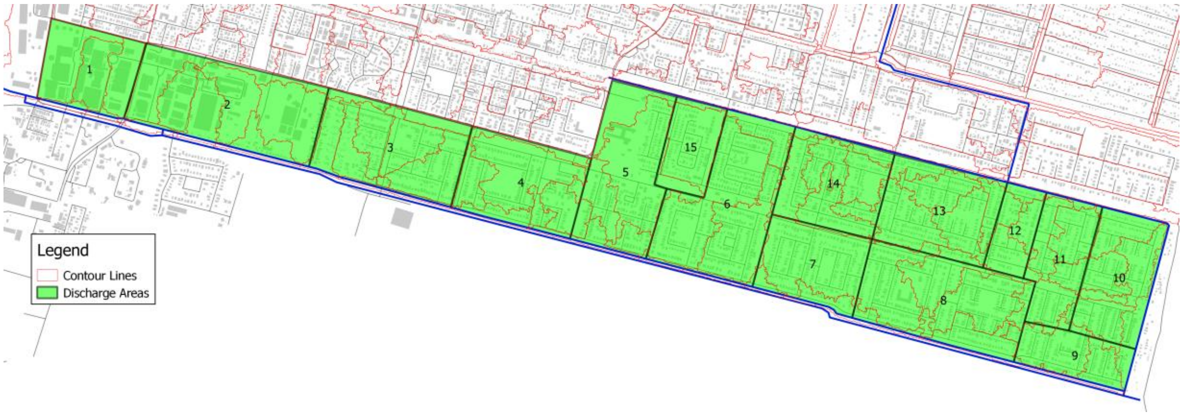


Figure 8 Contour lines and determined discharge areas (Openstreetmap, 2016), (Icaros Geosystems B.V., 2010)

The runoff coefficient (R_c) is a weighted average of different land use with their corresponding runoff coefficients (Faculty of Civil Engineering, 2011). It is the link between rainfall, surface area and the discharge into the channel. The runoff coefficients are chosen at its most conservative value to prevent underestimation (Faculty of Civil Engineering (DUT), 2011). With a GIS analysis the surface areas of different land covers per discharge area were estimated. However, this analysis can also be done by using the satellite imagery available. The total runoff coefficient and analysis is included in the excel program that has been made. This can be applied on other catchment and discharge areas.

$$Q_{\text{inflow}} = Q_{\text{rain}} R_c\, A \;(R_c = \text{Runoff Coefficient})$$

Table 4 Results runoff analysis

Nr.	Name	Surface area (km^2)	Runoff Coefficient
1	Banks Estate	0.18	0.74
2	Guyhog Gardens	0.36	0.67
3	Roxanne Burnham Gardens 2	0.29	0.66
4	Roxanne Burnham Gardens 1	0.24	0.59
5	South Ruimveldt Gardens 2	0.28	0.65
6	South Ruimveldt Gardens 1	0.34	0.68
7	South Ruimveldt Park 3	0.19	0.67
8	South Ruimveldt Park 2	0.33	0.74
9	South Ruimveldt Park 1	0.12	0.72
10	North Ruimveldt 1	0.21	0.69
11	North Ruimveldt 2	0.16	0.74
12	North Ruimveldt 3	0.09	0.72
13	Festival City 1	0.24	0.70
14	Festival City 2	0.22	0.65
15	Festival City 3	0.11	0.69

3.3 Area description

First of all, a zone within the catchment area needs to be chosen. The requirements of this zone are that it is not too small (otherwise the results are not interesting) or too large (otherwise a lot of fieldwork need to be done). The zone that is researched is zone 7. The boundaries of this zone are Aubrey Barker Road (on the north), the park between Penny Lane and Cauli Flower Circle (on the west), the channel along Caneview Avenue (on the south) and Blue Sackie Drive (on the east). This area dewaters on the South-Ruimveldt primary channel.

Figure 9 Situation of local drainage area under consideration

In order to find out how the local drainage system works, one needs to know where the secondary and tertiary channels are and if and how they are connected. Since the aerial pictures did not show where the culverts are located, the geometry of the system had to be found out by observation. During the fieldwork, these observations were made. The system is split up into three different elements, namely the culverts, the secondary and the tertiary channels. Of each element a description is made. In APPENDIX G some more examples are given.

3.3.1 Tertiary channels

The tertiary drainage channels are the channels that are located in front of the houses. Water falls on the property and should discharge from there into the tertiary channels. The depth of the discharge channels is approximately 0.5 m. There were basically two types of tertiary channels:

- There are tertiary channels that have wooden walls (1). They seem to be well regulated. However, vegetation is often seen in these channels.
- There are also channels that do not have any walls (2). Adjacent soil is likely to erode, leading to channels with deteriorating conveyance and storage capacity.

Figure 10 Difference between tertiary channels

3.3.2 Secondary channels

The secondary channels are important discharge channels through the different parts of the local drainage system. Culverts connect the secondary channels of different parts of the system with each other. The dimensions of the secondary channels are generally around 1 to 2 meter wide and 1 meter deep. The secondary channels consist of concrete flumes. The state of the concrete on the channel bed could not be analysed, since the bed level is below water level. The state of the channels (conveyance and storage) differed. Generally, two states can be distinguished:

- At some places, the channels were of good quality (1). The vegetation was taken away and the water table was low. There is a lot of storage capacity.
- The majority of the channels however were of bad quality (2). There was a lot of vegetation and waste disposal. When trying to walk through the channels, one could feel that there was a layer of sediment of about 0.3 meter at the bottom of the concrete flume. The smell was bad, and conveyance and storage could barely take place. At some locations the difference between maintained and non maintained channels was very visible (3).

Figure 11 Differences between secondary channels

3.3.3 Culverts

In this case study a culvert is a rectangular-shaped concrete element. There are 12 culverts in this system. Eight of them were inspected visually while standing in the channels; the other four were only inspected from street level. The observations were as follows:

- In all culverts, there was a layer of sediment of about 0.2 meter present. The culverts do not contain any vegetation, probably since sunlight cannot reach the culverts.
- In front of the entrance and the exit of the culverts, there are two states that can be distinguished: with and without vegetation (Figure 12). When there is vegetation present, the extent depends. At some places, the amount of vegetation is large (1) and at some places it is limited (2). At the places where no vegetation was present, it is not clear whether this was the consequence of maintenance (3). At other places, maintenance clearly has taken place (4).

Figure 12 State of channels in front of culverts

3.4 Model

In order to understand the dynamics in the local drainage system an easy to understand model was developed to model the effects of different scenarios. Below the scope (3.4.1) and methodology (3.4.2) of the model are explained. Afterwards the final model result (3.4.3) and some example scenarios (3.5) are explained.

3.4.1 Scope

The model increases the understanding of the local drainage system in one area in particular. The area under consideration is discharge area seven from the catchment area South-Ruimveldt (0). The goal of the model is to understand storage time and discharge rates both in sub-areas as well as at the final outflow point. Also the model should be able to give insight in different scenarios that can occur in the local drainage system. The final goal of the model is to translate rainfall rates into flow rates into the primary channel and thereby taking the delay due to the local drainage system into account.

3.4.2 Methodology

To develop the geometry of the model extensive fieldwork was done in the area under consideration. For modelling purposes the area is schematized and split up into twelve parts. These twelve parts are connected by culverts that underpass streets. Rainfall is equally distributed over the whole area. The final discharge into the primary channels happens (in this case) at three culverts.

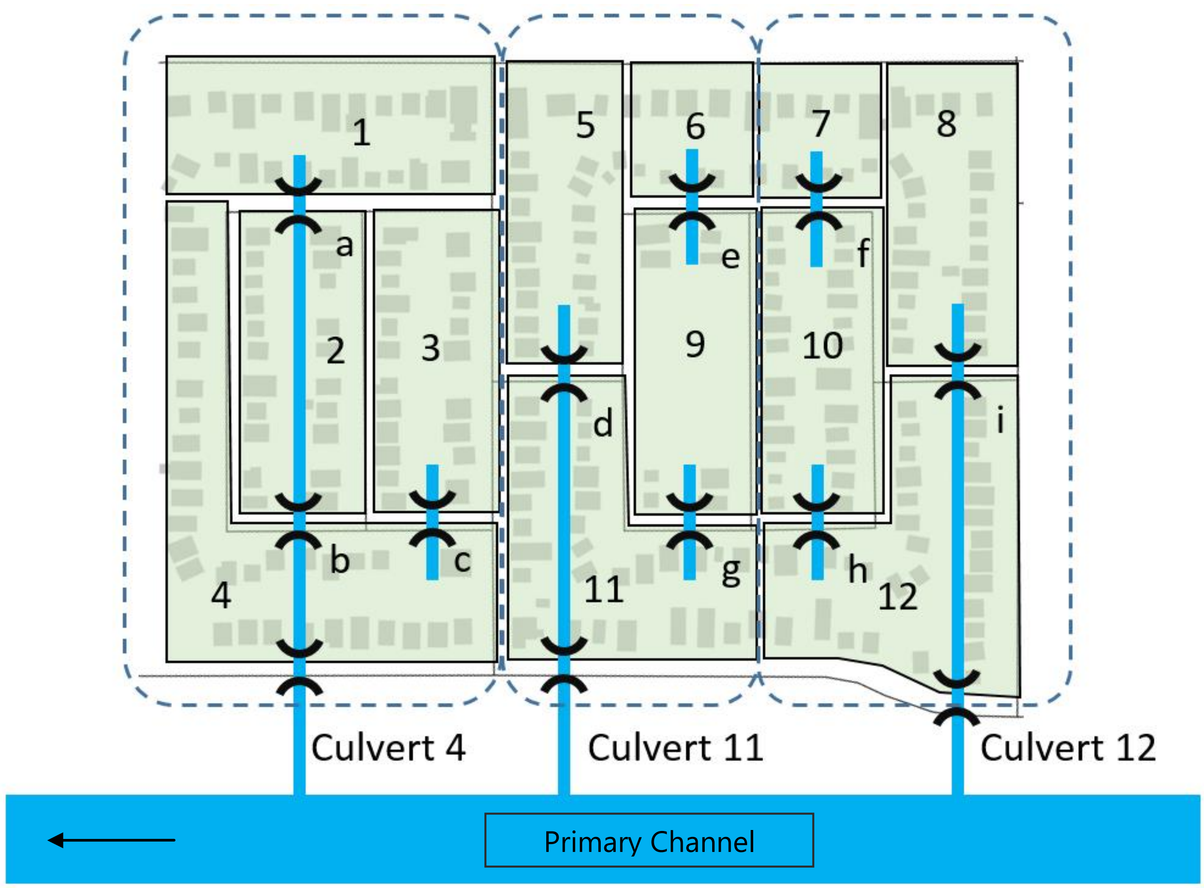


Figure 13 Schematization of local drainage system in considered area (Jos Muller, August 2016)

Using this schematization two basic formulas were used to compute the discharge in channels and through culverts. With an excel spreadsheet the twelve local areas were linked with each other. All formulas and the reasoning behind the computations can be found in APPENDIX G. The most basic assumptions on hydraulic and geometric parameters are:

- An equal roughness coefficient (Manning) for all the channels in the system (Battjes, 2002).
- A primary channel water level which is situated at a level of 30 centimetres with respect to the bottom of the culvert discharging on the primary channel.
- Equal culvert and channel dimensions for the whole area.
- Equal in- and outflow losses for all the culverts.

These assumptions were done with the goal of the model in mind. This model should give insight in the delay and the dynamics of the local drainage system. Because the small scale system is very vulnerable to changes it would have been counter effective to exactly replicate reality.

Table 5 Assumptions on hydraulic and geometric parameters in model for local drainage system

Parameter	Symbol	Value	Unit
Manning's coefficient	n	0.08	-
Width of the culverts	B	1	m
Effective height culverts	H	0.25	m
Initial depth drain	h	0.5	m

3.4.3 Results

The result of the model is an inundation map of the area and discharge rates at all three outflow locations on the primary channel. The inundation map indicates the theoretical inundation time of different local areas in the discharge area. The outflow rates indicate the discharge over time. Below the output of the model can be seen for the area with the characteristic that were present during the execution of this project. In the right top of the figures one can view the theoretical inundation times of each section during the assumed design storm. The most top left area is inundated for 445 minutes during such a storm.

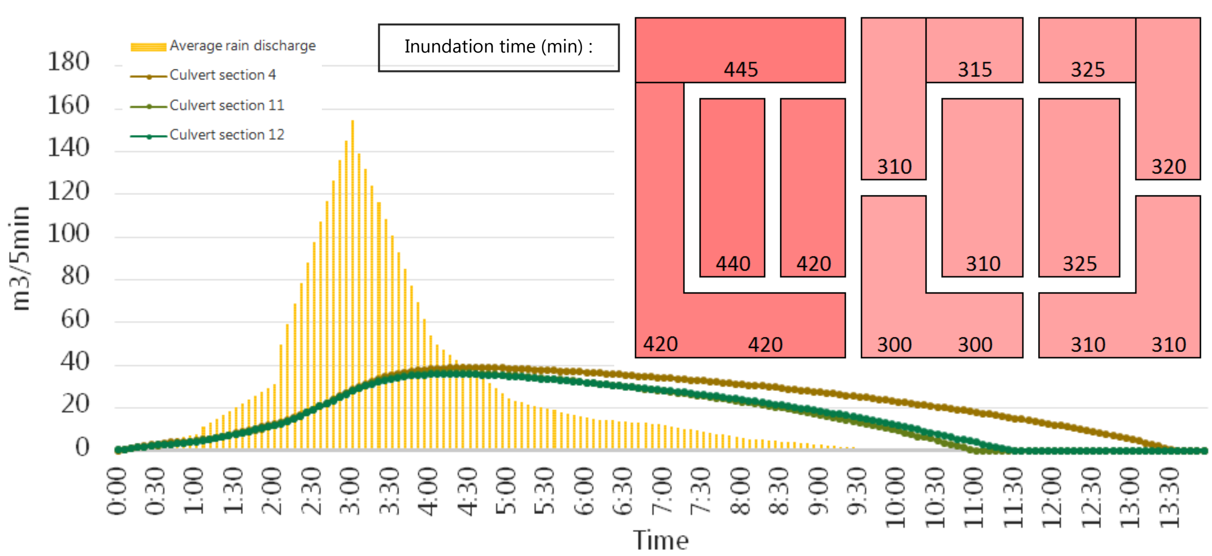


Figure 14 Inundation time (minutes) and discharges for the current situation in the example area.

3.5 Example scenarios

To examine the local drainage system different scenarios were modelled to get more insight in its behaviour and the consequences of interventions. Based on the failure tree, five cases of possible measures are designed that affect the water level rise and fall. All these scenarios were modelled with a rainfall input with a return rate of 2 years.

3.5.1 Dredging local drainage system

The first measure is the dredging of the channels. This was implemented by deepening the channels in the model with half a meter. Currently at some locations siltation (clay and confined plants) are situated at the bottom of the channels. Dredging could increase storage and discharge capacity.

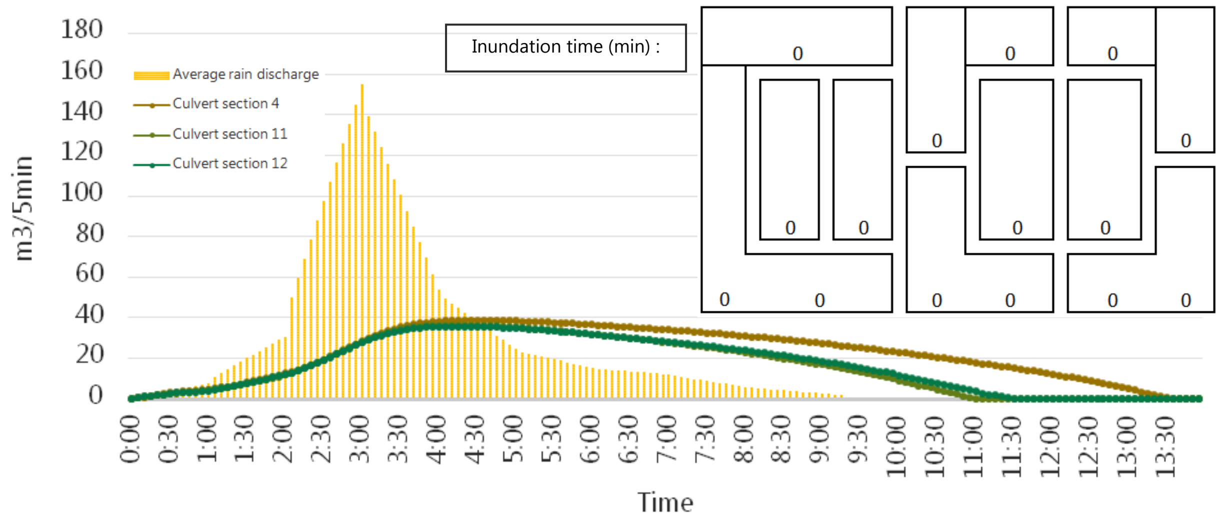


Figure 15 Output local drainage model for scenario: dredging channels

3.5.2 Cleaning local drainage system

The second measure could be the regular maintenance of the local channels and removal of vegetation. When channels are fully overgrown by vegetation, the flow is reduced due to turbulent motions. This difference is put into the model by changing Manning's coefficient (vegetated: 0.08, cleaned: 0.013).

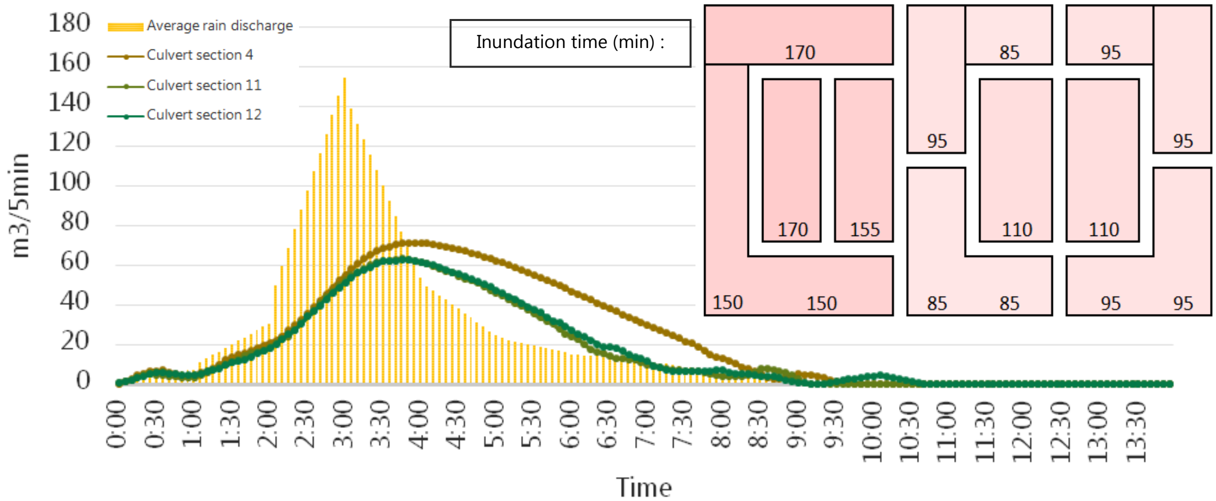


Figure 16 Output local drainage model for scenario: cleaning channels

In this case the water will still overflow the local drains. However, the outflow is much quicker with respect to the initial condition. This result can be seen in the overall zone, where the time the zone is inundated by the design storm is much lower. It can also be seen that the effect is notable throughout the zone, since all the channels are cleaned.

3.5.3 Cleaning culverts

The third measure is the clearing of the entrance and exit of the culverts. During the fieldwork the team noticed the significant blockage of the culverts inside zone 7. In the model cleaning was implemented by increasing the flow area of a culvert from 0.25 m^2 to 0.78 m^2.

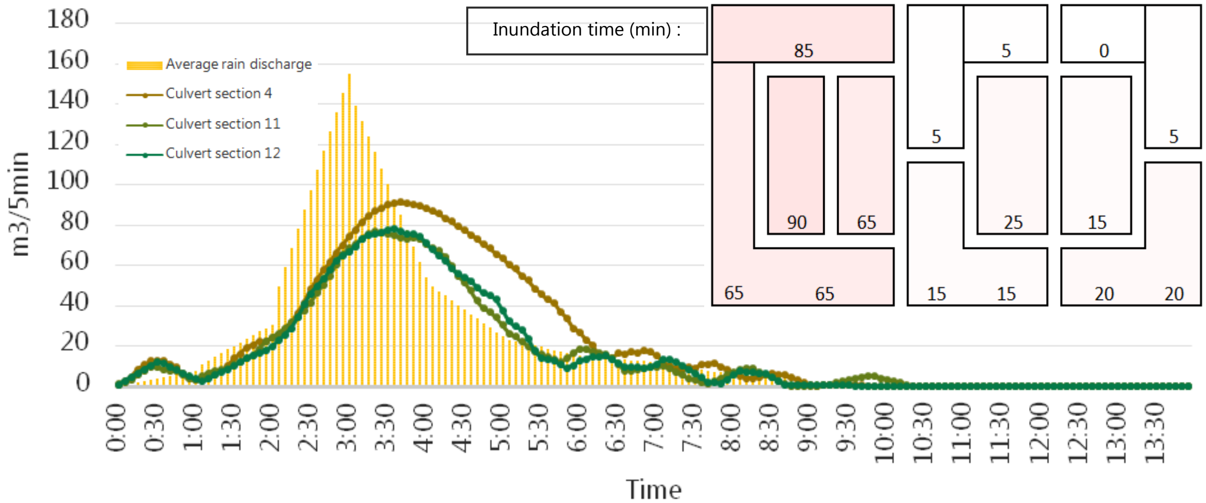


Figure 17 Output local drainage model for scenario: cleaning culverts

It can be seen that the dewatering is increased and the inundation time is much less. However in the sections where the connection of the culverts is not yet optimal, the adjacent sections will still be flooded, as is the case on the left part of the system.

3.5.4 Adding culverts

The fourth case is the adding of an extra culvert at the outflow point in the downstream sections. By adding an extra culvert to the most downstream section, the outflow from this bottleneck section will double. This option is in combination with the cleaning of the old culvert, since it is not realistic to reinforce the old culvert if this one is still constipated.

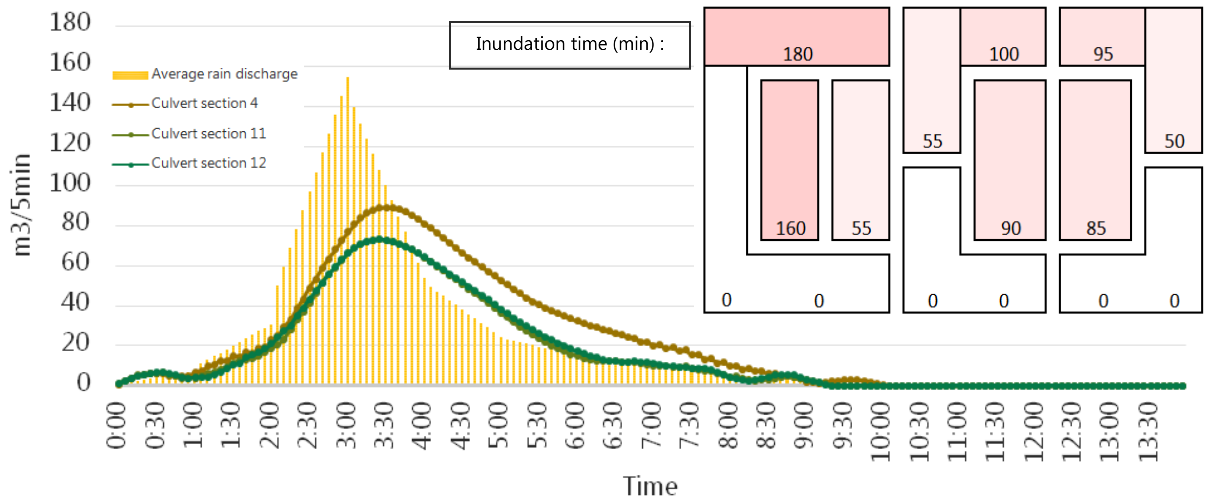


Figure 18 Output local drainage model for scenario: adding culverts

3.5.5 Lowering water level in primary channel

The fifth case is the lowering of water level in the primary channel. Discharge of water in the whole area does not take place if the most downstream culvert is not capable of dewatering due to high water levels in the primary channel. If the water level there is too high the system cannot drain. In this scenario the water level of the primary channel is at a lower level than the whole local drainage system.

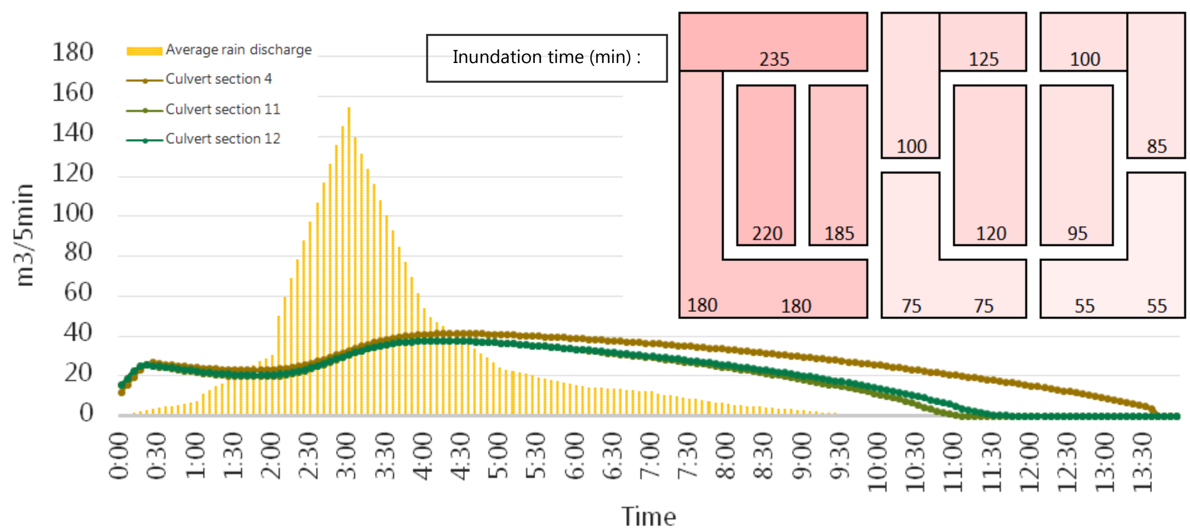


Figure 19 Output local drainage model for scenario: lowering primary channel water level

The model assumes that the initial water level was still present inside the zone. Therefore the biggest advantage of the reduction of water level from the primary channel is the outflow of water from the first moment rainfall event occurs. Since there is no further measure taken inside the zone, the outflow rate remains approximately the same.

3.5.6 Comparison of scenarios

The outcome of the different cases gave two kinds of results. The first is the compared impact per measure. By comparing the time at which the zone is considered flooded with the original inundation time the theoretical reduction per measure on the system can be expressed in percentage. The second is the delayed discharge time into the primary channel which can be seen in all output graphs.

Table 6 Tentative result and effect of interventions

Measure	Inundation (min)	Relative effectiveness (%)
No adjustments	445	-
Dredging local drainage system	0	100
Cleaning local drainage system	170	62
Cleaning culverts	85	81
Adding culverts	180	60
Lowering primary channel	295	56

3.6 Remarks

During the system analysis a model was made of one discharge area in the catchment area South-Ruimveldt. The recommendations for improvement of the modelling (capacity) will be given in chapter 9. However, the following remarks should be taken into account while using this analysis.

- The model is deterministic. This means that the calculations are made with one input value.
- Because the analysis is on short water storm runoff, a full conservation of mass is applied to the system, so there is no loss due to evaporation or intrusion.
- There is no gradient present in the model. This is due to the fact there is no detailed data available on the surface elevation of the wards and no measurements were executed. It is also questionable if there is a significant difference in surface elevation present in the wards.
- In the model water cannot move back into a zone from more downstream areas. In reality water could flow back into the zone if the water level downstream section higher.
- The model only works within the volume of the local channels. Once the water level reaches the maximum depth of the channels, the water level will keep on rising as it was contained by the initial channels. Overflow of banks is not taken into account.
- The culverts outflow is determined using a theoretical approximation. The values obtained from these computations were judged to high and not realistic for the culverts in zone 7. Therefore a factor was applied for reduction. This factor has to be calibrated for further use.

A general delayed profile is one of the results of this part. It was composed based on the analysis above and is used for all local drainage systems in the hydraulic model of the primary channel in chapter 4.

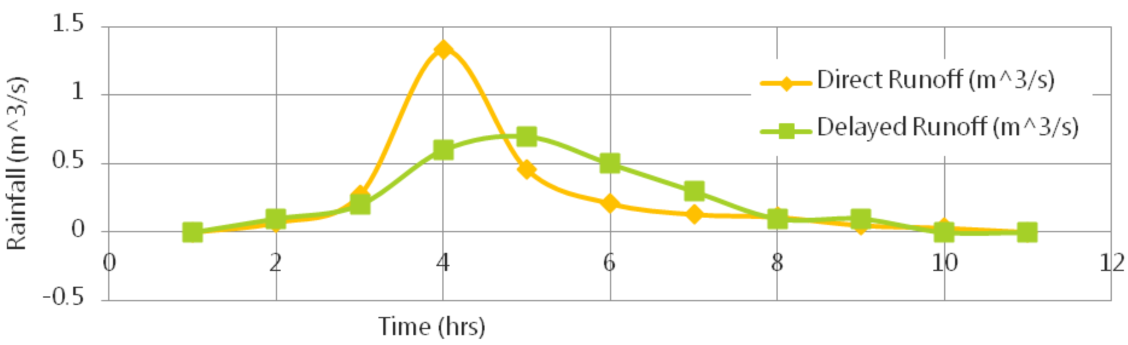


Figure 20 Delayed runoff due to local drainage system

4 PRIMARY DRAINAGE CHANNELS

In this chapter the primary drainage channels are examined. First an introduction to the subject (4.1) and scope (4.2) of the analysis is given. Afterwards the full methodology used for making the hydraulic model which is used for analysing the system is given (4.3). Finally some interventions are modelled and their effects are examined (4.4). Last but not least some additional remarks are given which are needed to take into account when the analysis is considered (0).

4.1 Introduction

A primary channel discharges water between the local drainage system and the system boundaries (sea or river). Discharging happens via either a pumping station or a koker. A hydraulic model can be used to analyse the effectiveness of different interventions in the primary channel. The development and use of such a hydraulic model will be the subject of this chapter. An example analysis was done on the primary channel of the example area South Ruimveldt. This channel is situated south of the catchment area South-Ruimveldt and connected to the Houston North channel and a small part of the La Penitence channel.

Figure 21 Location of the Ruimveldt Channel including connections

4.2 Scope

To analyse the primary channels a hydraulic model will be developed. This was done to increase the knowledge-based decision making on the drainage system of Georgetown, Guyana. Besides that it is a follow up action on one recommendation of the DRR team: increasing the modelling capability in Guyana. The scope of the hydraulic model in this project includes the following criteria:

- The model must analyse the hydraulics in the Ruimveldt-South primary channel over time.
- The model should be open source and the Guyanese stakeholders must be able to use it.
- The model should be able to increase the understanding of the impact of different interventions.
- All steps needed for the production of a hydraulic model have to be executed in order to transfer a finished product.

Three software packages will be used for developing the model. The main program is the one dimensional flow model HEC-RAS (v4.1) developed by the United States Army Corps of Engineers (USACE, 2010). Besides this Quantum GIS (QGIS Development Team, 2016) and spreadsheet models were used to support the work in HEC-RAS.

4.2.1 HEC-RAS Model information

The model HEC-RAS is a hydraulic computer model to compute flow regimes in rivers and channels. It is developed by the US Army Corps of Engineers and can be used for a wide amount of modelling purposes. It has an interface which is easy to use and hardly any knowledge of programming is required to get the model running. Also generating and using the output of the model is relatively easy and can therefore be done without a lot of knowledge on the program. The major advantage of HEC-RAS is that is free to download and use, so the model can be used both by authorities, students and responsible engineers in Guyana. This is the main reason why HEC-RAS is chosen as the modelling tool for this project.

The model requires two different kinds of input data:

1. Geometric data
   The geometric data describes the shape of the channel. This includes the geometry regarding structures like bridges, culverts, kokers and a vast amount of cross sections measured along the channel.

2. Boundary and input conditions
   The boundary and input conditions are required for the hydraulic computations. In the specific case of this example model this will mainly consist of rainfall and discharge data, tide readings at the riverside, the use of pumps and kokers and their corresponding discharge.

HEC-RAS can execute different kind of models but the focus in this report lies on the modelling of the hydraulics in the primary channels in an unsteady flow situation. This is necessary for understanding of the behaviour of the channel over time and correct interpretation of the effect of interventions.

4.2.2 Additional remarks

The most important limitations of the model are described below. They are predominantly based on the current knowledge on modelling in Guyana and on the limitations within the program HEC-RAS itself.

1. The current achievements in modelling in HEC-RAS at the involved stakeholders in Guyana (UG, NDIA, MOPI) reach until the input of geometric and steady flow data for known scenarios. The model should continue from this step in the analysis and expand the knowledge to unsteady flow modelling. This is necessary for analysing the hydraulics in the channel over time.

2. HEC-RAS requires a vast amount of cross sectional and geometry data as input. Because knowledge and material on the geometric analysis is available in Guyana time was saved on geometrical fieldwork and interpolations and assumptions were used where possible in the model.

3. HEC-RAS is able to do computations on sediment transport. However, due to the variation of thick vegetation, silt and clay which is transported in the channel and the current knowledge on sediment transport it was chosen to eliminate these computations (USACE, 2010).

4. The goal of the modelling exercise is to start with an increase of the qualitative understanding of the hydraulic system and not an exact representation of reality. Therefore simplifications and adjustments of some physical processes are added for increased user friendliness.

5. HEC-RAS is a one dimensional flow model. River bends and inundation of surrounding lands are hard to include in a proper manner. Extreme scenarios of flood inundation and sudden failure of kokers or breaching of banks are hard to model as well.

4.3 Methodology

In this section, the different steps in the development of the hydraulic model for the Ruimveldt-South area are explained. First, a description is given on the collection and use of geometric data (4.3.1). Next, the hydraulic boundary and input conditions are described (4.3.2), followed by a description of model assumptions (4.3.3 calibration (4.3.4) and (numerical) accuracy (4.3.5).

4.3.1 Geometric data

The first step that was executed in making the model was retrieving the geometric data. In the past the University of Guyana did work in analyzing the geometry of for instance the Cummings Canal in Georgetown (Linton, 2015). Using elevation reference points from Lands and Surveillance they created a geometric representation of the system. For effective time use it was therefore chosen to execute a basic geometric analysis and put more focus on unsteady flow modelling which is not used yet in Guyana.

During two days of fieldwork a global geometric overview was made of the first kilometres of the primary channel of the catchment area South-Ruimveldt. At locations of considerable geometric changes (for instance near bridges and culverts) the cross section and elevation with respect to one reference point at the outfall koker was measured. This was done using a theodolite and a measuring rod. Afterwards the following tools where used to expand the geometric dataset:

- Using the available LIDAR data a depth elevation map was constructed (APPENDIX D). This was used to estimate the elevations of the final half of the channel (Icaros Geosystems B.V., 2010).
- To improve numerical accuracy several cross sections were added based on the tool cross section interpolation in HEC-RAS.
- The Houston North channel was not measured but implemented as a replica of the South-Ruimveldt channel. This introduces the effect of delayed discharge due to the length of the channel.

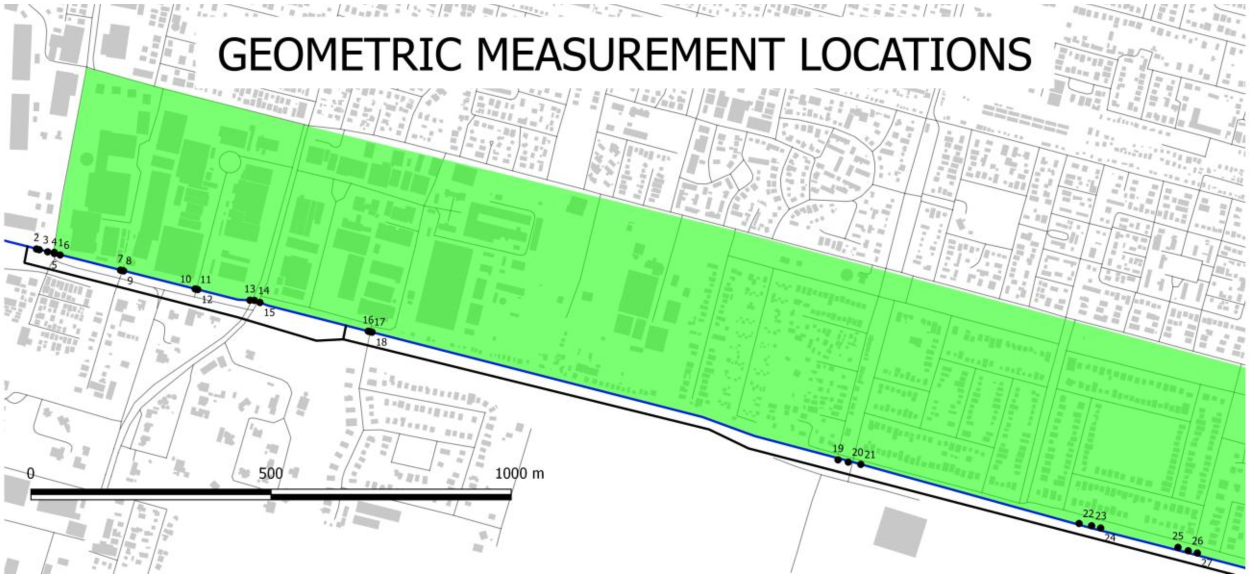


Figure 22 Measurement points during the three measurement campaigns

4.3.2 Hydraulic input and boundary conditions

The hydraulic input and boundary conditions used for the model consist of two components. At first the discharges which come from the local drainage system are the most important input condition. Besides of that also the tidal cycle near the koker is important for the operations of the system.

The discharges into the system are determined using the hydrological analysis presented in paragraph 3.2. Because no exact map of working culverts is available and the operability of these culverts is highly variable it was chosen to use the natural drainage lines of the different discharge areas as a weighted average. The intersection of these natural drainage lines with the primary channel are uses as inflow points in the model.

Figure 23 Natural drainage lines and inflow points hydraulic model

The primary channel is bounded via the koker with the Demarara river. Because Georgetown lies below sea level the drainage time via the kokers is therefore limited by the tide. Tide readings are available and do not vary significantly along the river shore of Georgetown (Ministry of Public Infrastructure, 2014). Both average spring and neap tide profiles where generated from this data. Higher water levels at the river side of the koker could also occur during storm surges or high river discharges. However, in this stage of modelling it is not interesting to investigate this, because both processes are not expected to be correlated.

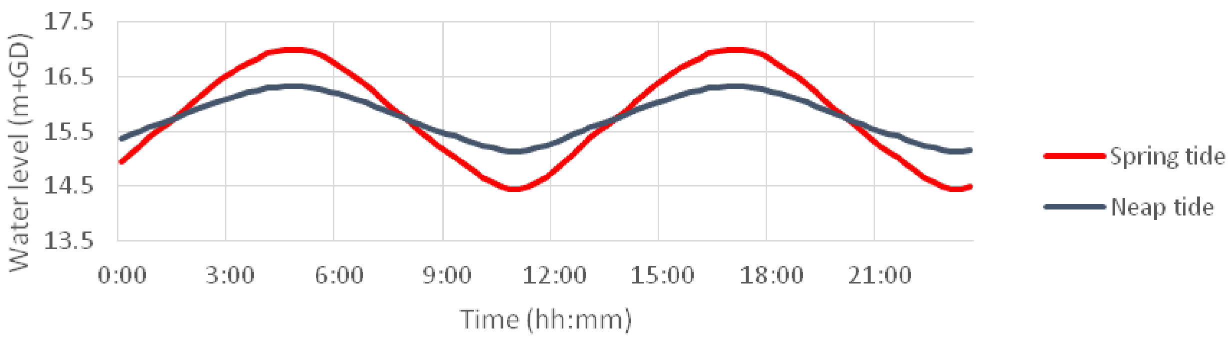


Figure 24 Average spring and neap tide at Georgetown's measuring station (Ministry of Public Infrastructure, 2014)

4.3.3 Assumptions

The most important complications in the catchment areas are described below. These complications where discovered during fieldwork in the first two weeks. To include these complications in the hydraulic model some assumptions had to be made which are explained below.

1. Connection Houston North and South-Ruimveldt:

The outfall koker of Houston-North does not function properly. To drain the abandoned sugar cane estate Houston, the Houston-North and Ruimveldt-South channel were connected at two locations in the past (Figure 21). This implies that the geometry and hydraulics of the Houston-North catchment has a strong influence on the Ruimveldt-South channel behaviour.

Figure 25 Clarification on connection South-Ruimveldt and Houston North (Joost Remmers, August 2016)

To account for this connection the Houston North channel was added to the model as a simplified primary channel. This was done using an average cross section of the South-Ruimveldt primary channel. In this way the delayed discharge of Houston North is taken into account. The discharge to this channel consists of both discharges from sugar cane fields and the local drainage system of the Meadowbank area (the village between the cane fields and the Demerara).

2. Division La Penitence and South-Ruimveldt:

A wall divides the La Penitence channel in two sections of which one drains on the Ruimveldt-South channel. It is unsure if this wall functions as a division during high water conditions. In the final model this part of the primary channel is not included due to time limitations. Therefore the connection was not added to the model. When the model is improved it should be figured out what the functioning around this structure is.

3. Connection Liliendaal and South-Ruimveldt:

The South-Ruimveldt and Liliendaal channel are connected to each other by a siphon at Lamaha Street. However, the siphon does not function well and it is unclear what the interaction is between the two catchment areas. Discharge has been observed in both directions in along large parts of the South-Ruimveldt channel.

To account for the discharge which flows from South-Ruimveldt to the Liliendaal catchment a pumping station was included at the end of the modelled reach. The capacity of the 'imaginary' pumping station is equal to the capacity of Liliendaal reduced with the needed capacity to drain the Liliendaal area.

4. Koker and pump operators:

It was observed that operators do not discharge according to set procedures, mainly due to the lack of procedures at all. Liliendaal pumping station discharges whenever possible. The koker (at the end of Ruimveldt-South) discharges depending on water levels inside and outside the koker. Both outlets are operated manually.

The Liliendaal pumping station is assumed to have effect constantly during high water levels (+15 GD) at the end of Ruimveldt. The koker is modelled as an elevation controlled gate and opens when water levels in front and at the back of the gate are equal. It was observed that opening of the gate takes ten minutes.

5. Roughness along longitudinal and lateral direction:

The channels have varying vegetation along its longitudinal and lateral direction. Choosing the correct roughness coefficients for each section is necessary for proper calibration of the model.

During constructing of the model appropriate roughness coefficients were chosen according to rules of thumb from the HEC-RAS manual. However, after calibration these values were adjusted (4.3.4).

6. Blocked inflow from local drainage system due to equal water levels with primary channel:

Along the South-Ruimveldt areas several inflow points do not function anymore during high water levels in the primary channels because water levels on both side of the inflow point are equal.

To account for this effect the discharge areas were modelled as storage areas with lateral structures connecting them to the channel. The size of the storage area is the estimated volume of the local drainage system. This estimation is equal to the road length multiplied width an average drain with of 2 meters per road meter. Both bottom levels where modelled 0.5 meter below bank level. This accounts for the storage and blocking effect during high water levels in the primary channel.

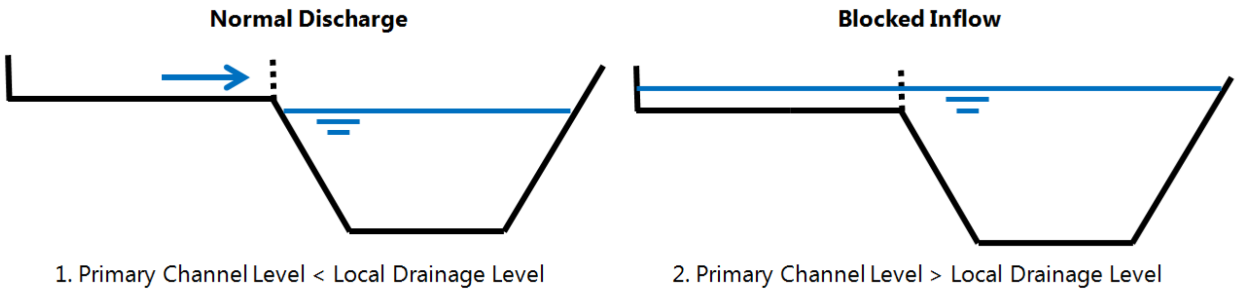


Figure 26 Clarification on assumption of blocked inflow (Jos Muller, August 2016)

7. Correct implementation of bridges:

Bridges along the channels have small discharge openings which do not allow for bridge modelling in HEC-RAS. This poses computational problems.

To compensate for this effect the East Bank Public Road bridge was modelled as a culvert instead of a bridge. This accounts for the small discharge opening which is present under the bridge.

8. Influence of bend and inflow losses:

Because HEC-RAS is a 1D program it does not explicitly take losses in bends into account. This accounts for both losses due to bends in the primary channel and at the inflow points.

In the final model no bends were taken into account. However, bend losses could be taken into account by increasing the overbank length and roughness.

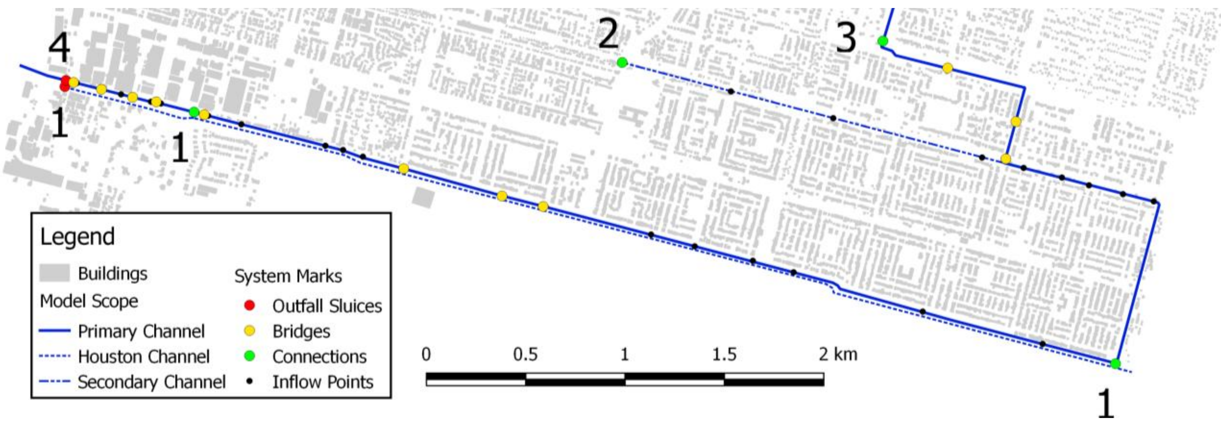


Figure 27 Location and mapping of model complications (Joost Remmers, August 2016)

4.3.4 Calibration

Calibration and validation of the hydraulic model is necessary for the quantification of the effect of different interventions. Because during the execution of the project the dry season started it was only possible to do one field measurement exercise.

On the 24th of August 2016 a flow and water level measurement was executed at the Meadowbank bridge during a full period of discharging by the koker. The flow measurements were taken with a basic flow measurement device which is also available at the University of Guyana and the water level measurements were executed with the a simple measurement rod. In this way the measurements can be reproduced by the Guyanese stakeholders.

To achieve a correlation between the model results and the measurements the following steps were undertaken in HEC-RAS.

- All Manning roughness values were increased with 5% with respect to the original assumption.
- A measurement error was discovered with respect to the reference of the tide and 0.1 meters was added to the tidal record to achieve correct water levels inside the primary channels.
- The gate opening time was reduced from 30 to 10 minutes.

To achieve a fully calibrated model it is necessary to do more calibration measurements, especially during high discharge time series. To achieve a fully calibrated model the following steps can be followed:

1. Calibrate the inflow from different storage areas with the rainfall record per hour to assure the correct unsteady flow input data.
2. Calibrate the primary channel with different stages and high waters to assure the correct geometry data and boundary conditions.
3. Tune the full model with measurements during high discharge periods to complete the calibration.

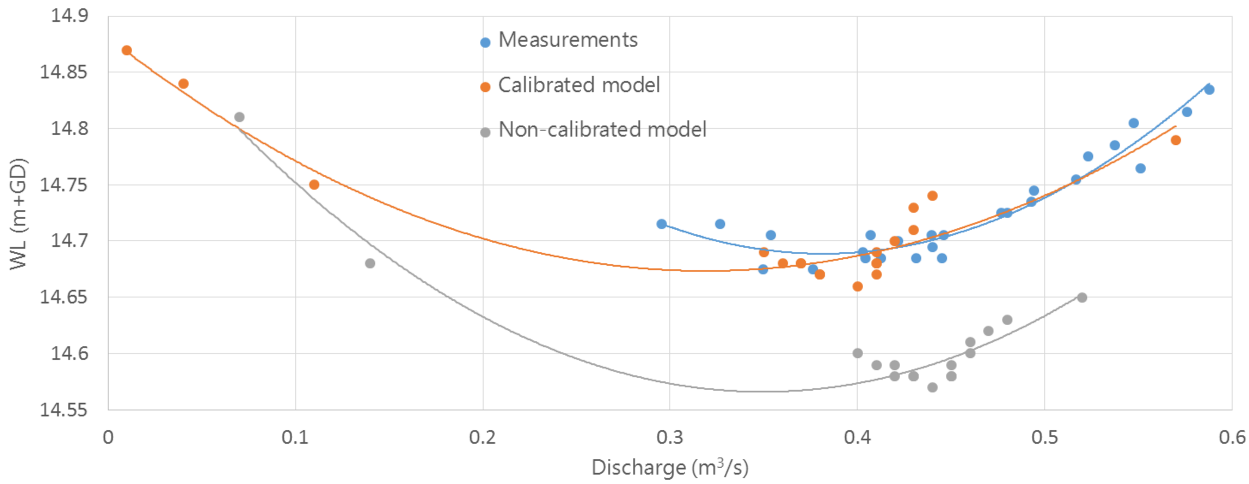


Figure 28 Comparison HEC-RAS output with measurements

4.3.5 Accuracy

The accuracy of the model depends on four different aspects: the accuracy of the assumptions which were made during the process (1), the quality of geometric data (2), the accuracy of flow data and boundary conditions (2) and the numerical processes.

1. Accuracy of assumptions

In paragraph 4.3.3 the assumptions which were made were explained. Especially the influence of the pump at Liliendaal, hydraulic roughness assumptions and the connection of Houston North to Ruimveldt are assumptions which compromise the accuracy of the model. However, the influence of all seven assumptions is present in the model.

2. Geometric Data

As stated in paragraph 4.3.1 the geometric analysis which was executed was very basic. Improvement using the same technique as in the model of Cummings Canal (Linton, 2015) can be made. Besides the accuracy of the measurement cross section spacing is important as well. Using the Freads and Samuals equation it was noted that sufficient cross section spacing is applied (USACE, 2010). However, due to rapid changes in geometry near structures cross sections were added using interpolation to improve the accuracy of the analysis.

3. Flow data and boundary conditions

In unsteady flow HEC-RAS only computes using a subcritical regime. Mixed flow options were disabled. Besides that the initial conditions were designed in such a way so they comply with the boundary conditions. Lateral structures were made as small as reasonably possible to improve accuracy.

4. Numerical accuracy

Using the Courant conditions for different sections of the channel a safe assumption of minimal two minutes was made for numerical computation. The weighting factors and number of iterations are set to the maximal amount to make computation errors as small as possible. During unsteady simulation runs the maximum water surface error was estimated to be four centimetres which is sufficient for this analysis.

By improving the geometric data and calibrating the model the global accuracy can be improved. By adding sufficient cross sections, using the correct time step and improving the hydraulic inflow amount and mechanism the model accuracy can be improved as well.

4.4 Example scenarios

To put more emphasis on the use of the model eight different interventions or scenarios have been implemented in the model. All interventions and scenarios were modelled with discharges corresponding to a storm with a return rate of 1 year. In all output graphs the stage is the left axis, and the discharge (flow) is the right axis. In each output figure the base scenario (as it is currently) is given as well with green lines. Please note that a sudden rise in discharge of the channel indicates that the koker door opened. A sudden stop indicates the koker door closing. Also do note that the rainfall is modelled to fall at the least favourite moment which is during high tide (when the channel cannot discharge).

4.4.1 Influence of the East Bank Public Road bridge

The assumption was that this bridge has a significant influence on reducing the conveyance of the channel. The two scenarios that will be compared are the original geometry of the bridge and one with more conveyance.

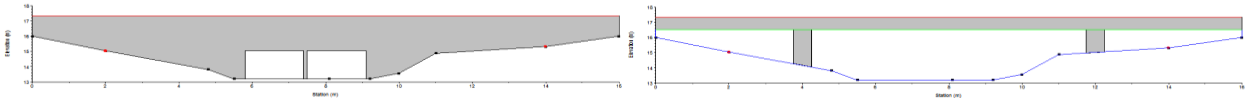


Figure 29 Adjustments in geometry East Bank Public Road bridge

The effect of reducing the cross sectional area is hardly noticeable. The discharge slightly increases and water levels slightly drop. It could be possible that the apparent influence that was noted is not due to the East Bank Public bridge but the confluence with the Houston channel.

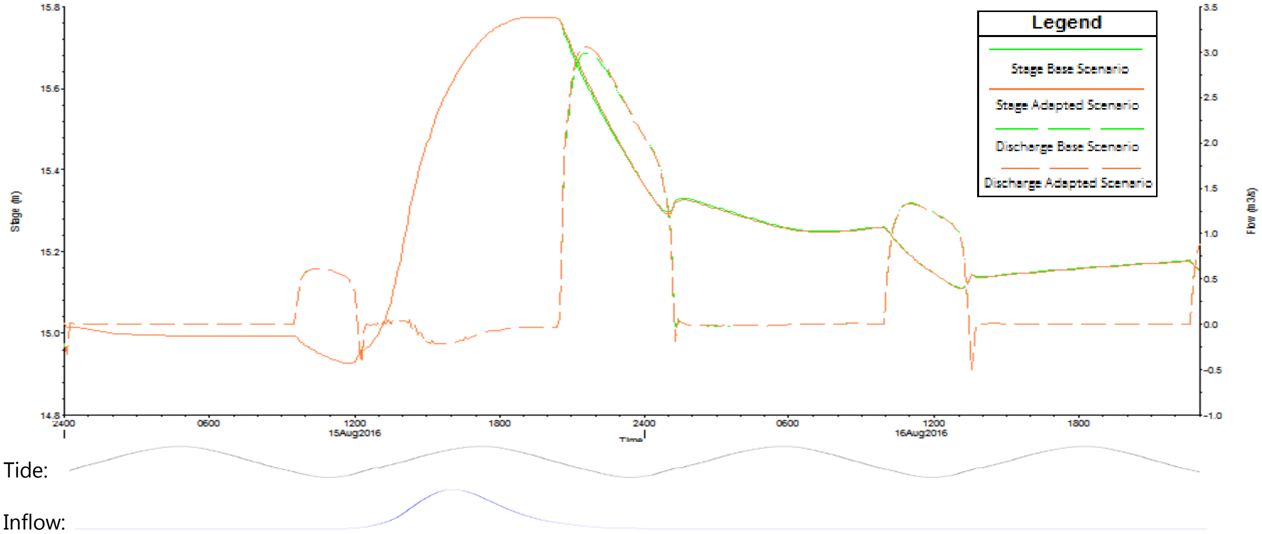


Figure 30 Model output on adjusted geometry of East Bank Public Road bridge

4.4.2 Construction new bridge

The Houston North catchment area below South-Ruimveldt is prone to new developments and therefore plans for the construction of new large bridges over the primary channel could be considered in the future. As a scenario a large conveyance reducing bridge platform was added to the base model structure.

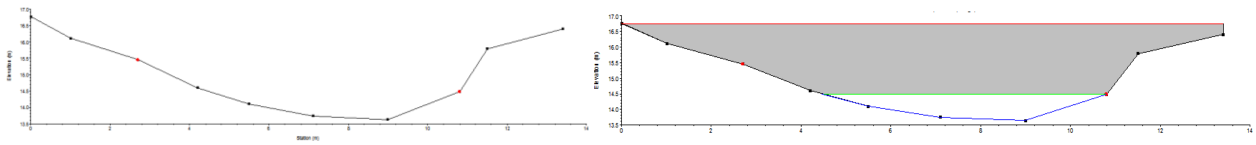


Figure 31 Adjustment in geometry by adding a new bridge

By comparing discharge and stage levels at the cross section below the bridge it can be noted that the bridge has a significant influence on the discharge capacity and water levels.

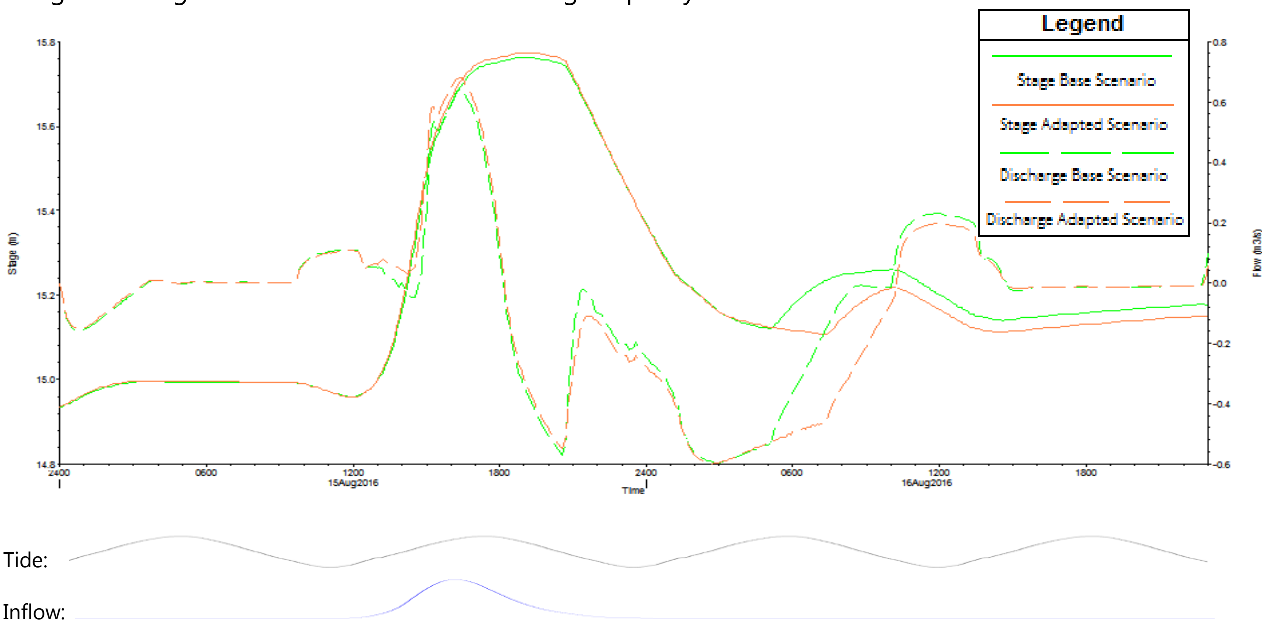


Figure 32 Model output on the effect of adding a new bridge to the primary channel South-Ruimveldt

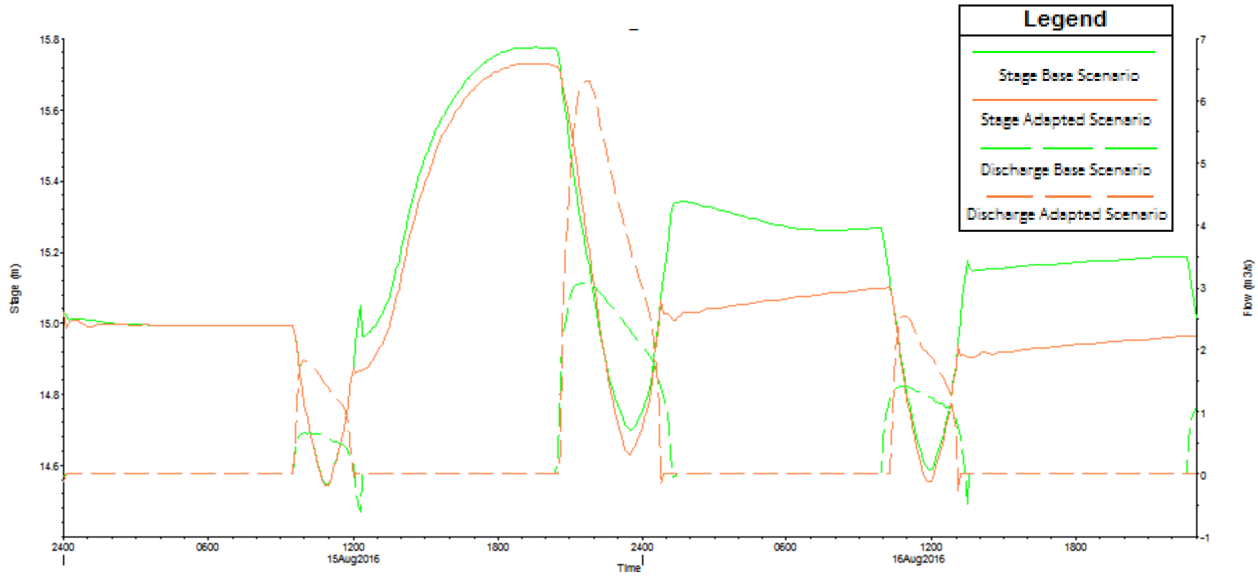
4.4.3 Cleaning the channel

Vegetation grows fast in the tropical climate of Georgetown. Growing vegetation, dumped garbage and other material increases the roughness of the channel bed and banks. In this section a scenario a proper maintained channel and a vegetated (base model) channel are compared. The Manning roughness values for a cleaned channel are based on earthen lined channels (channel) and short grass vegetation (banks).

Table 7 Assumed Manning roughness for normal and cleaned channels

Scenario	Left Bank	Channel	Right Bank
Vegetated (base model)	0.105	0.084	0.105
Cleaned	0.027	0.018	0.027

The influence of cleaning the channel is considerable. It is expressed below in a stage and discharge graph near the koker. It should be noted however that the roughness values for the cleaned channel are based on the basic values in the manual. Nevertheless cleaning the channel has a significant influence on both discharge and water levels. Not so much during the peak rainfall since the koker is closed during that period. But later discharges are much higher and stages much lower, hence the water can be discharged quicker and inundation will therefore be shorter.



Tide: 

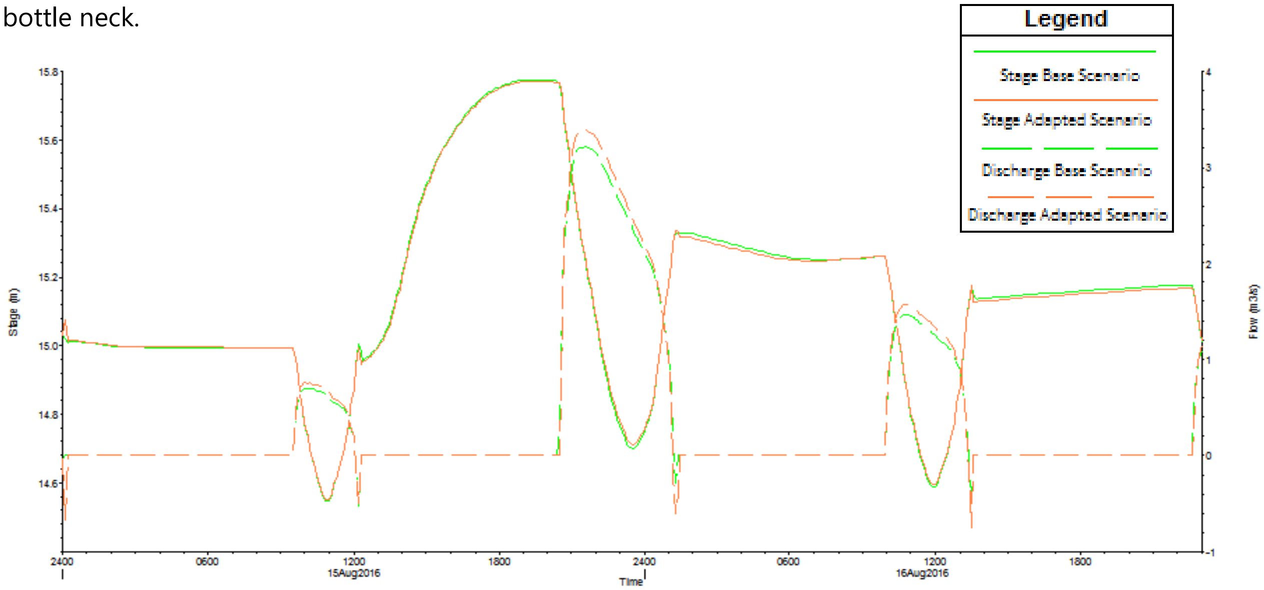
Inflow: 

Figure 33 Output of cleaned (orange) versus vegetated channels (green) on koker stage and discharge

4.4.4 Widening the primary channel

One of the proposed measures is increasing the discharge and storage capacity. This can be done by enlarging the cross sectional area. For this scenario the size of the cross sections in the first river reach (the first 700 metres of the channel) in transverse direction was increased by ten percent.

The widening of the first reach with ten percent results in slightly higher peak discharges (approximately five percent) and has no significant effect on the water levels. Hence the width of the channel is not the bottle neck.



Tide: 

Inflow: 

Figure 34 Influence of widening the channel on stages and discharges on cross section halfway

4.4.5 Adding a storage area

Increasing the storage capacity in the drainage system can be done by using a specified area near the primary channel for storage. This intervention was included in the model by placing a 19.000 square meters area on a non-used part of the South-Ruimveldt catchment.

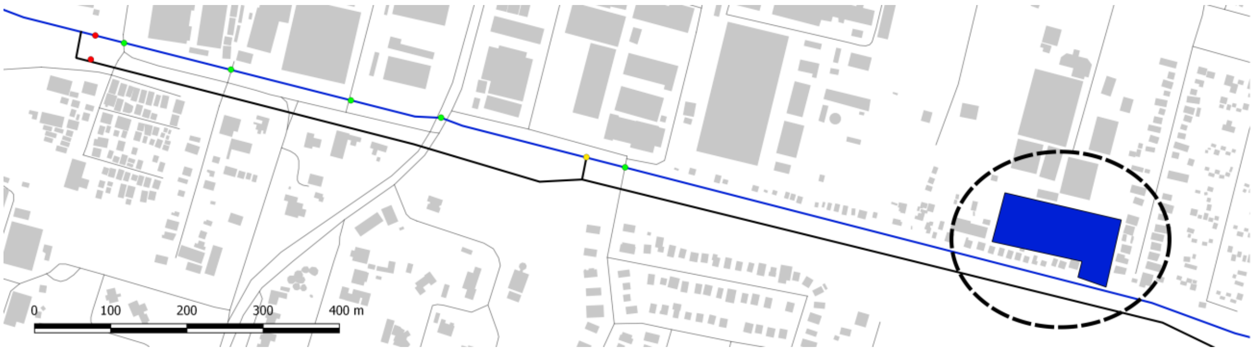


Figure 35 Location of the modelled storage area in the South-Ruimveldt catchment in Guyhog Gardens

The influence of the storage area is large and it results and can be best summarized as follows. In the following graph, a longitudinal cross section of the South-Ruimveldt channel is shown. The structures such as bridges and culverts are included and are shown as the vertical lines in the plot, with the height indicating the deck levels of the structures. The bed level is plotted and is the lower black line, which was only measured for the downstream area. The water levels in the current situation and in the adapted scenario are plotted as green lines and red lines respectively

First the storage area decreases the water levels along the channel. Secondly it stores a sufficient amount of water during the rainfall and discharge period while it releases it after this period. Thirdly the peak discharge lowers as well due to the storage in the first period. This cannot be viewed in the output in Figure 36 Influence on maximum water levels of an added storage area (along length of the channel)Figure 36 which presents the maximum water levels.

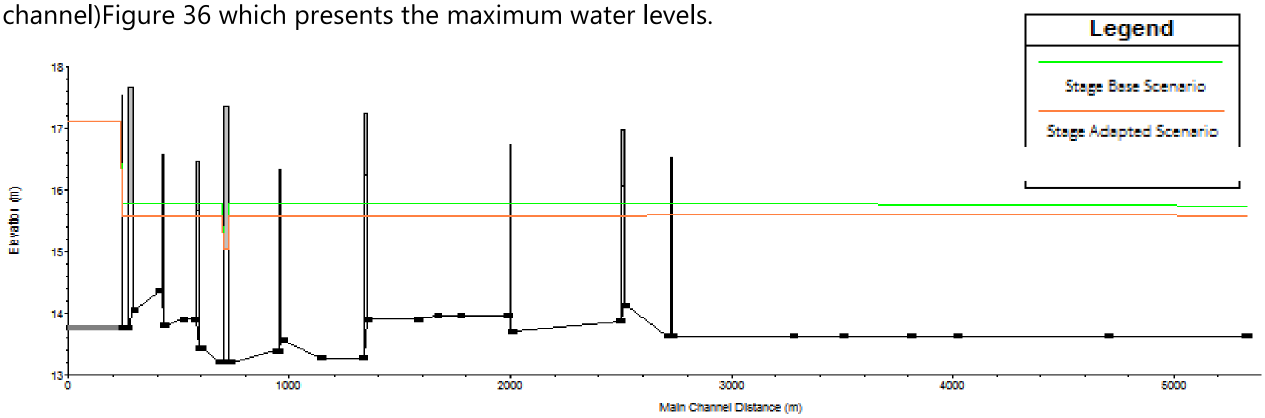


Figure 36 Influence on maximum water levels of an added storage area (along length of the channel)

4.4.6 Adding a pumping station at the koker

Sometimes a koker has an (emergency) pump next to it to discharge water at all times (for instance: Young Street). In the model a pumping station with the capacity of Young Street (1 m^3/s) was added to the koker as one of the modelled scenarios. The pump switches on at a water level at the koker of 15.2 meter and stops when the water level reaches 14.8 meters. The model sometimes has some small calculation errors when the water level is around 14.8 or 15.2 (Figure 37). These are the multiple following vertical lines at three quarters of the length of the graph.

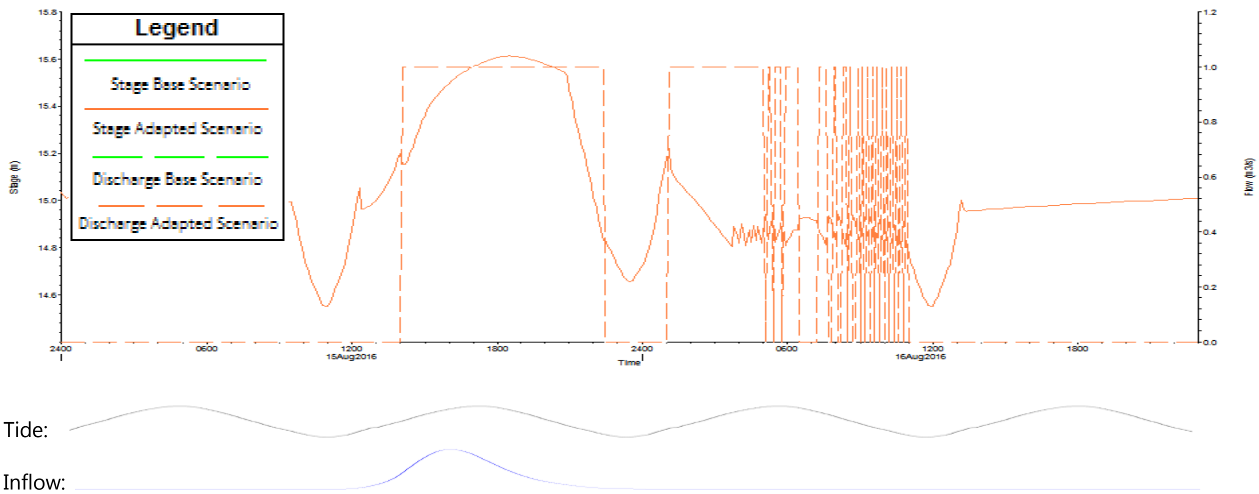


Figure 37 Model output on additional pump at the koker, discharges over runtime

The pumping station results in a considerable reduction of discharge time for the whole system. Besides that it allows for discharge during high tide which lowers the maximum water level.

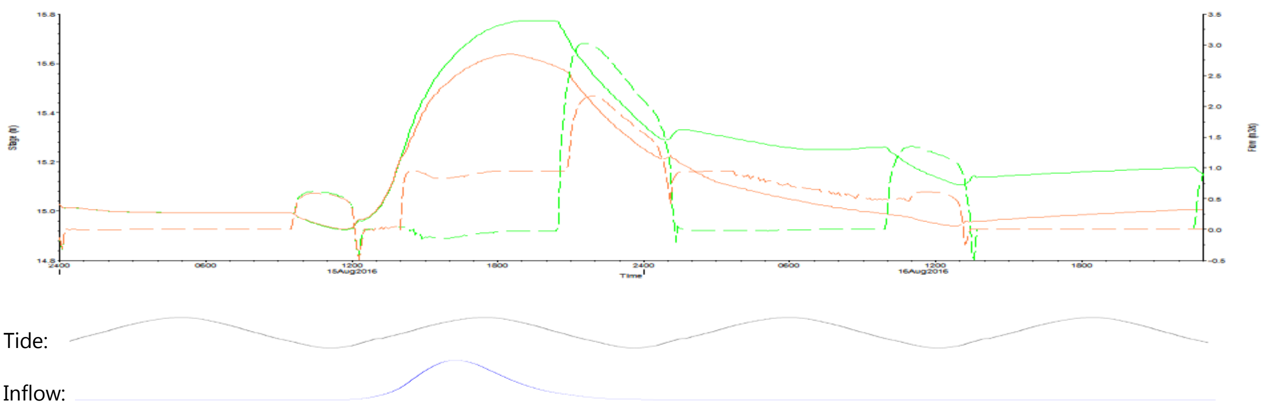


Figure 38 Influence of an added pump on discharge and water levels at the koker

4.4.7 Blocked culverts in local drainage system

In the base model scenario the inflow from the local drainage system is delayed by culverts. However, in certain circumstances culverts could be blocked. This could result in an even further delayed inflow into the system. The analysis of the local drainage system is used and a delayed inflow is put into the model.

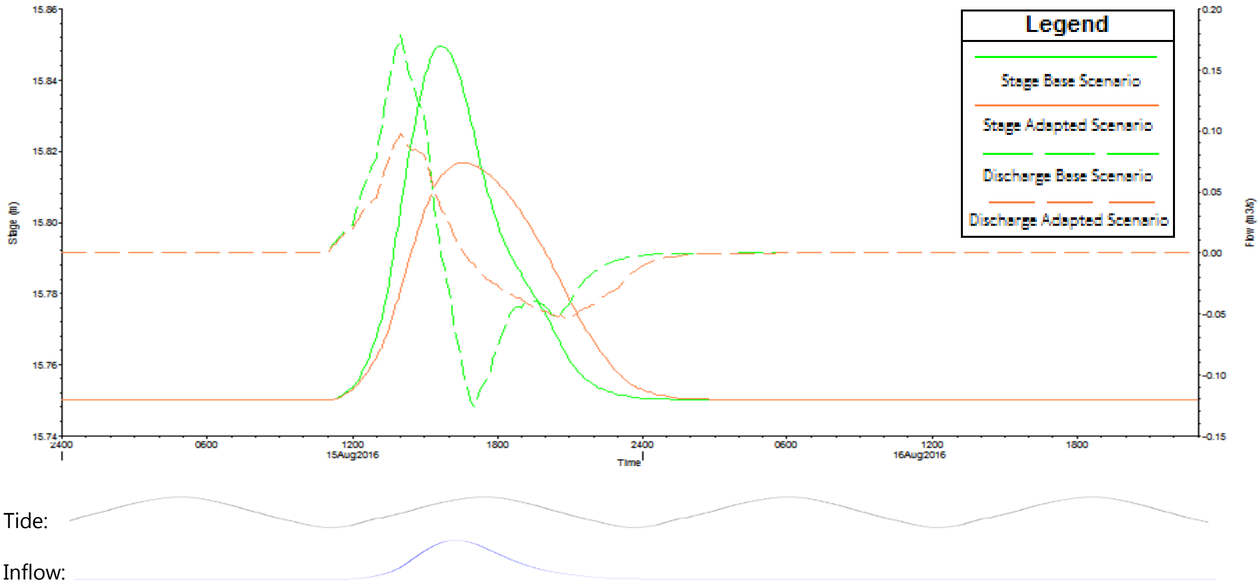


Figure 39 Differences between a working and malfunctioning local drainage system

The effect of delayed inflow is visible in the total system and it is noticeable that it averages out water levels and discharges. However at some locations peak water levels and discharge increase. This could be caused by the interaction between the primary channel and the (modelled) inflow storage areas (4.3.3). In Figure 39 this phenomenon is not visible.

4.4.8 Closing connection with Houston North

South-Ruimveldt primary channel is connected with the primary channel of catchment area Houston North to relief the Houston North channel from its malfunctioning koker (4.3.3). In this scenario the coupled channel scenario is compared to a scenario in which the South-Ruimveldt channel is decoupled from the Houston North channel.

The influence of the closure on the maximum water levels in South-Ruimveldt is not large. However, discharging and lowering of water levels along the Ruimveldt channel happens a lot faster.

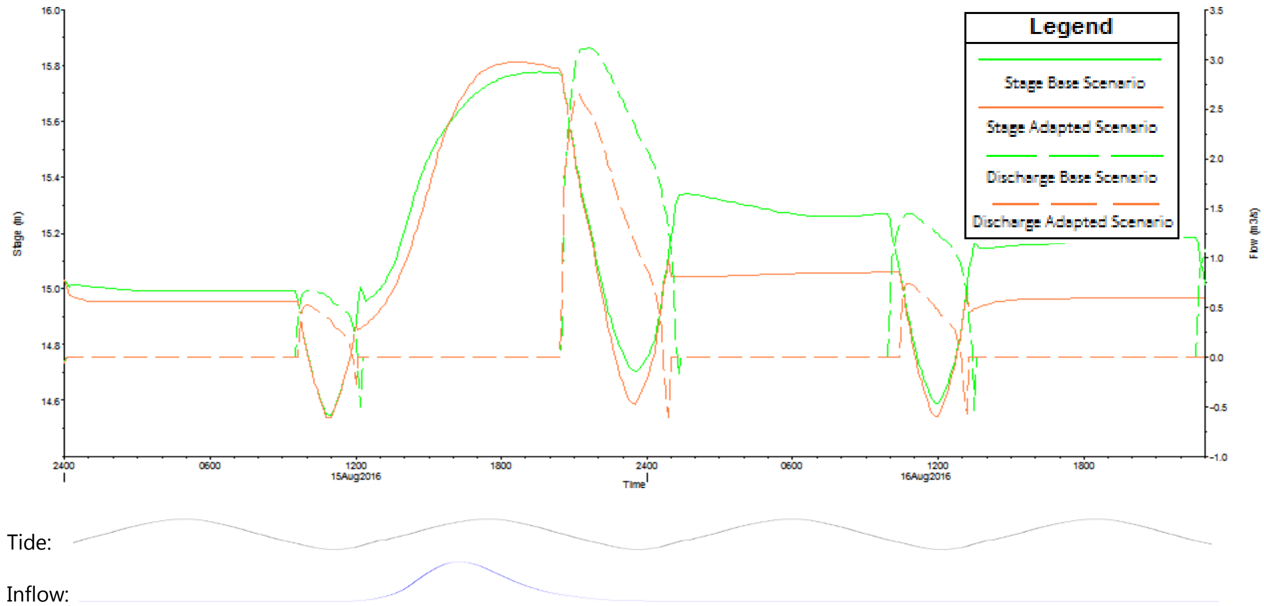


Figure 40 Comparison of koker stage and water level with and without Houston channel connected

Please do note that this is a drainage problem which should be examined in an integral process explained in chapter Knowledge-based decision making2.4. This is due to the fact that closing the connection will have consequences for the catchment area Houston North. Therefore the effect of the measure on the other catchments should be examined as well.

4.5 Remarks

The model gives reasonable insight into the system and the effect of different interventions. This is all based on the assumptions and measurements made. The recommendations for improvement of the modelling (capacity) will be given in chapter 9. However, the following remarks should be taken into account while using the model in its current state.

- Manning roughness values are equal for the total length of the channel. In reality vegetation will differ between cross sections.
- The rainfall is currently homogeneously distributed over the catchment area in the model. In reality the rainfall will have a distribution over space.
- The geometry upstream of the last bridge in South-Ruimveldt and of the Houston North channel is estimated. In reality this geometry differs and this has an effect on the current output.
- The elevation of the lateral structures (and therefore the inflow points in the primary channel) is estimated to be 0.5 meter below bank level. This was based on four measurements. In reality this elevation and the amount of culverts/connections differs per connection.
- The contraction and expansion coefficients where unaltered from their standard values. In reality they differ per bridge and they can be verified during high water discharges.

Of the eight modelled interventions four have a significant effect. Especially cleaning the channels, adding a pumping station to the koker, closing the connection of South-Ruimveldt with Houston North and adding a storage area seem to have effect on water levels or discharge (time).

However, it must be noted that all these interventions have their setbacks as well. These have to be taken into account during decision making.

- Cleaning might not always be possible due to inaccessibility and needs regular maintenance.
- Closing the connection between South-Ruimveldt and Houston North could lead to severe problems in the catchment area Houston North if the koker and drainage system is not repaired there.
- Constructing a pumping station requires high investments and continuous maintenance and attention of operators.
- Constructing a storage area uses a lot of potentially expensive space and needs maintenance as well.

To determine the effectiveness of the interventions the costs must be taken into account relative to their effects and setbacks.

5 OUTFALL STRUCTURES

In this chapter the research and work done on the kokers at the Demerara river will be explained. First an introduction will be given to the general structures (5.1) and the principles of maintenance (5.2). Afterwards the scope of the project (0) and the methodology used (5.4) are described. Finally the work done is applied on an example scenario (0) and some additional remarks are given (5.6).

5.1 Introduction

The outlet of the primary drainage channels is regulated by many outfall structures. These kokers open during low tide of the Demerara River and close during high tide. It is the last step in the complete drainage system, and therefore of high importance for preventing flooding.

The kokers are built in the early 1900s and are mostly made of concrete, excluding some brick kokers and steel elements, and can be found along the west side of Georgetown. In the north are two pumping stations located; Liliendaal and Kitty. These pumping stations are beyond the scope of this part of the project.

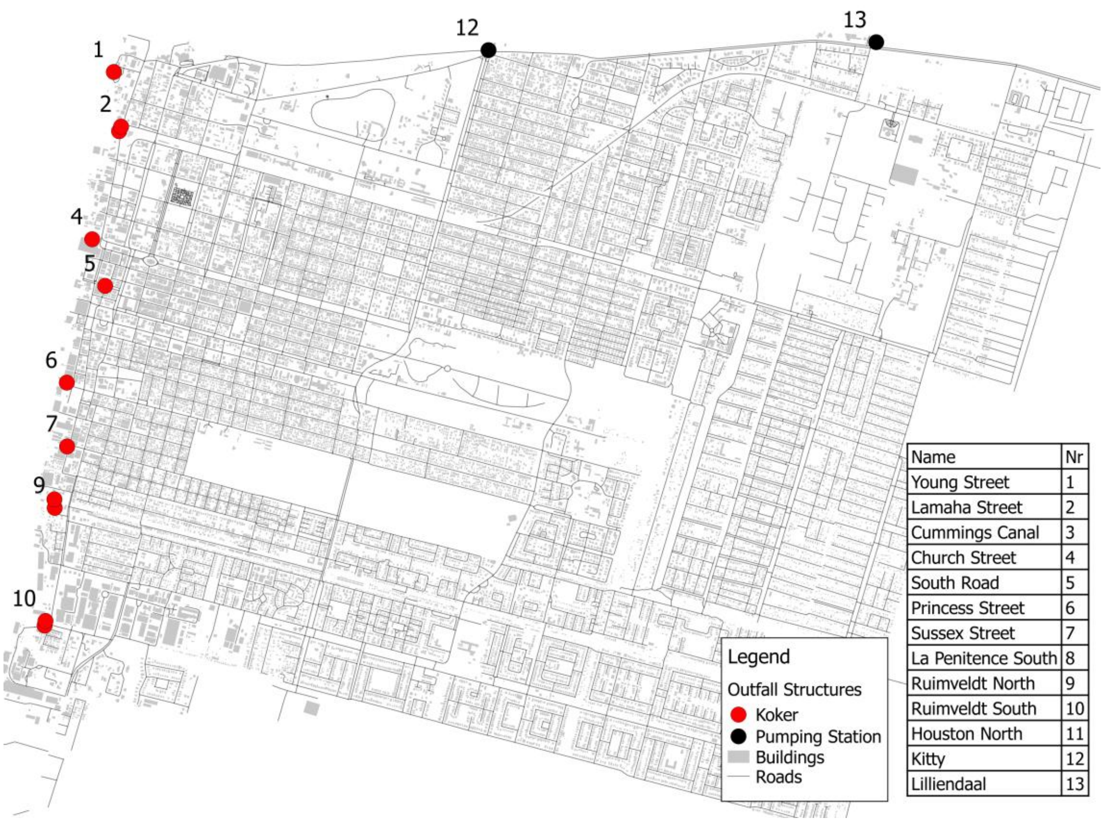


Figure 41 Overview of outfall structures in Georgetown, Guyana (Openstreetmap, 2016)

The kokers are managed by the MoPI and M&CC and each single koker is controlled by its own operator. These operators live beside the koker and are responsible for the opening and closing of the kokers. Throughout the country, engineers are checking if the operators are doing their job well, by visiting them regularly. At certain kokers emergency pumps are placed. These emergency pumps are used as a back-up system if the kokers fail (will not open) or do not have sufficient capacity. These emergency pumps are partly controlled by the NDIA and the M&CC.

5.2 Maintenance

Due to the long lifespan of the kokers maintenance is an important aspect in order to keep the system working. A definition of maintenance is the (regular) fixing of small mechanical, structural or other problems in order to prevent the structure from failure.

The goal of maintenance is to increase the life span of the structure which can be reached by fixing small problems with the elements of the structure. By performing maintenance the total cost of the structure can be decreased, the failure probability lowered and the life span can increase. By inspecting the structure on a regular base, future failure mechanisms can be detected. Otherwise the structure may suddenly fail and this could result in bigger costs, injuries or floods. In Georgetown the M&CC and the MoPI are responsible for the maintenance of the kokers.

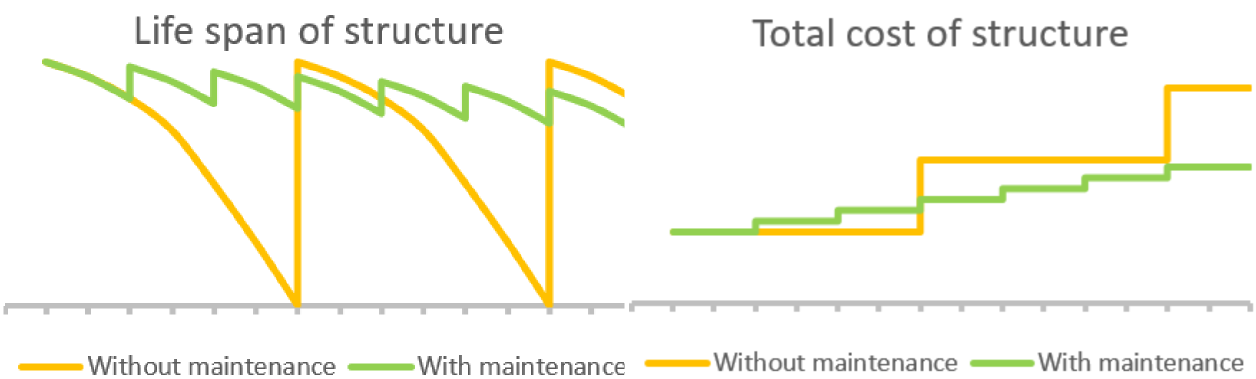


Figure 42 Graphical representations of the principle of maintenance (Thijmen Jaspers Focks, August 2016)

In the figure above a graphical representation of maintenance is presented. Every jump of the green line in represents small maintenance to the structure. If the element of a structure, which is most likely to fail, is replaced, the structural integrity as well as the total life span of the structure is increased. The right graph compares the total costs of a structure with or without maintenance. Every increment of the orange graph represents replacement of a large structural element due to failure.

There are two different types of maintenance: preventive and corrective (Life Cycle Engineers, 2010). With preventive maintenance the elements are repaired before it fails, whereas corrective maintenance repairs the elements after they failed. Corrective maintenance mostly is more expensive due to probability of more elements being damaged by one failing element. In Georgetown, Guyana signs of corrective maintenance are present at some outfall structures.

5.3 Scope

In this analysis research will be executed on the kokers according to a specific approach which is called structural asset management. This approach can be used to improve the quality of structures by performing regular inspections and based on these inspections decide whether or not to perform actions.

Figure 43 Process of structural asset management (Thijmen Jaspers Focks, August 2016)

This approach is discussed in the book Management of Deteriorating Concrete Structures by George Somerville. The approach is schematized as shown in Figure 44. This scheme shows the different steps to be taken in order to assess a structure in a proper manner and which steps are included in this project (indicated in red boxes) (Somerville, 2008).

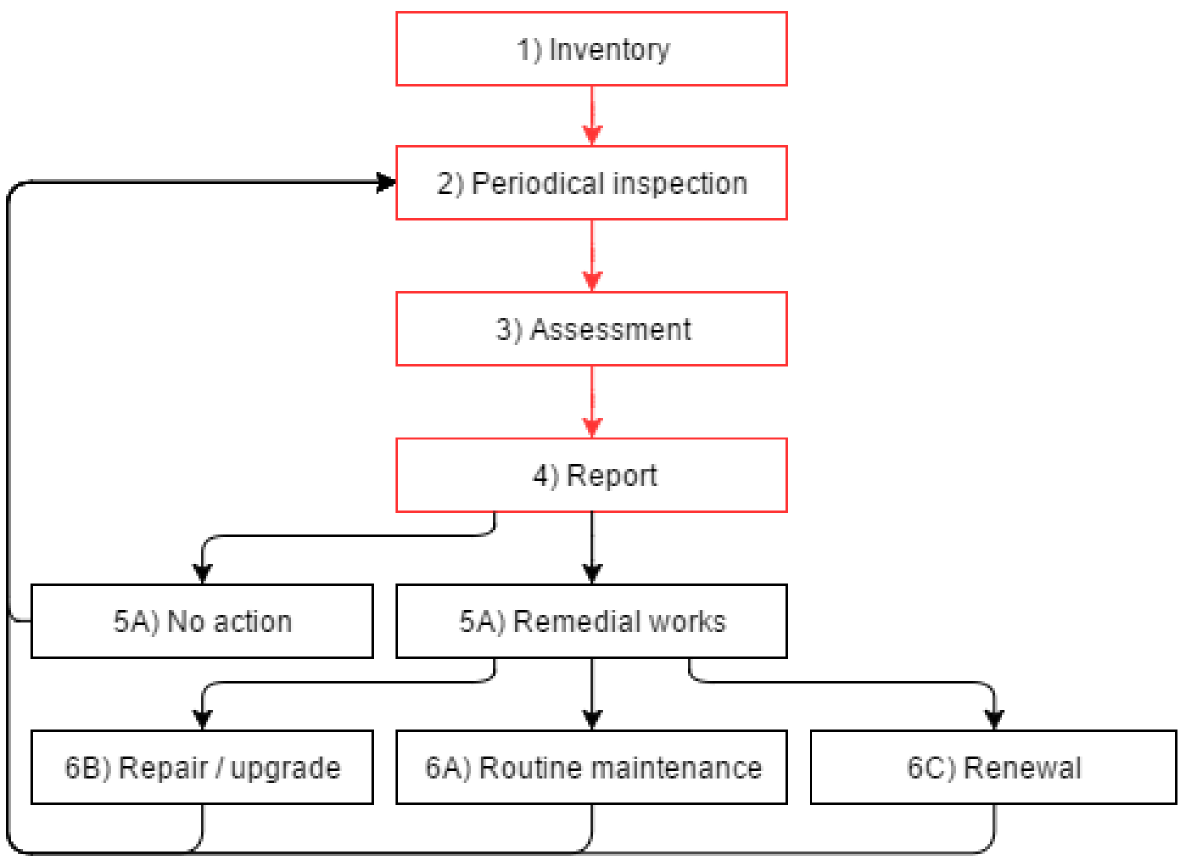


Figure 44 Structural asset management and included steps in koker research (Somerville, 2008)

The final product can be used for information based decision regarding maintenance. Steps 5 and 6 are actually the application of the assessment tool and are beyond the scope of this project.

During this project a structural asset management tool will be developed. This tool can be used to improve the use of preventive maintenance. The developed tool consists of an inspection form with manual, a data management system and an Excel document which provides an overview of the life span of a structure.

5.4 Methodology

In this chapter the total methodology which was used to make the structural asset management tool is explained. First explanation is given on the inventory and drafting of the tool (5.4.1). Afterwards the periodical inspection is described (5.4.2). Thereafter the assessment will be explained (5.4.3), followed by the way of reporting the assessment (5.4.4).

5.4.1 Inventory

The first step of the structural assessment is to gather as much data of the kokers as possible, such as the material, dimensions, the different components and the usage. This information is digitized by making fact sheets for each of the kokers. Besides that, theoretical information of probabilistic design and failure mechanisms of structures has been obtained as well.

Based on the theoretical information and gained data of the kokers, a failure tree was developed. This failure tree gives an overview of the failure mechanism of the kokers. For a few failure mechanisms, the probability of failure has been calculated, using the method of Monte Carlo. Combined with the grading of the different elements, discussed in paragraph 5.4.2, the probability of failure is calculated by entering the grading in the excel file. The result is a graph with the condition of the element over the years and a supporting manual as well.

5.4.2 Periodical inspection

In order to assess the structure in an objective way, an inspection form is developed. The inspection form consist of all the different element of a structure and room for grading (APPENDIX I). The inspection form is intended to be as objective as possible. A support manual should give some background information on how to use the inspection form and thereby assess a structure in an objective manner.

5.4.3 Assessment

The inspection form is based on a first inventory and therefore it should be calibrated. During the project the tool was tested on two of the kokers. Instead of writing a report about the assessment, the inspection form and manual were improved. This implies that after the assessment, a new inventory has been done. This feedback loop is essential for developing a proper tool.

5.4.4 Report

After improving and finalizing the inspection form and manual, the assessment tool is ready to be used. In order to make the tool transferable, an example koker will be assessed in the next paragraph. The reporting part of the assessment is up to the inspector.

5.5 Example koker

In this paragraph a made up example koker, called the 'Mainstreet koker', will be inspected and the possible maintenance strategies will be explained. This example assessment explains the steps taken in order to set up a maintenance system and can be used as a step by step manual for kokers in Georgetown as well elsewhere in Guyana. During this example the four steps explained in the scope explanation will be executed (0). It was chosen to use an example koker for two different reasons. The first reason is that, by using an example koker, all possible elements can be included. The second reason is that the example koker presents a general scenario, based on real kokers in Georgetown, which is applicable to every koker to avoid misconceptions with other kokers..

Mainstreet koker is constructed in the early 1900's and is located in an urban area and is connected to a sea. Meaning the koker can only drain water from the city during low tide. The entire superstructure is built with concrete and the door is made of hardwood.

5.5.1 Inventory

In the first step of the example all data on the koker is documented in the factsheets provided. The koker was visited and the dimensions and other facts about the koker were written down in the koker factsheet. These factsheets give an immediate overview of the koker and contain the information on the catchment area, the operator(s) and the maintenance history. This gives anybody the opportunity to understand what kind of koker they are dealing with.

1.1 General

Construction year	1931		Pump present	No	
Number of doors	1		Garbage filter	No	
Material	Concrete		Raising system	A	
Possible to maintain	The koker is easily accessible from both sides with all types of vehicles.				
Operator name	Steve Owinson				
Operator phone number	06 123 45 678				
Operator since	21-06-1994				
Size catchment area	102.8	ha	Width of sluice opening	2.80	m
Invert level of Sluice	13.29	mgD	Cross section below 15.54 mGD	6.30	m^2
Sluice depth below 15.54 mGD	2.25	m	Catchment/cross area	16.32	$\frac{ha}{m^2}$

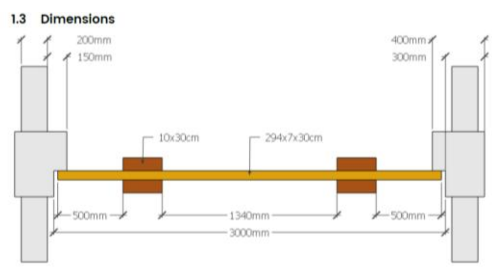


Figure 45 Elements of factsheet inventory (Thijmen Jaspers Focks, August 2016)

5.5.2 Failure mechanisms

The second step in the assessment was to acquire more information about the structural behaviour and capacity of the structure and relate this data to the probability of failure of the structure. This means Ultimate Limit State (ULS) has to be determined for the different mechanisms of failure. In this case failure represents a situation in which the koker is unable to control the flow of water between its two sides.

To determine the possible failure mechanisms, a failure tree was created. Such a diagram visualises the mechanisms by which the structure can fail and is based on the definition of failure for the koker. The failure tree consists of three different layers. The first layer, indicated with a capital letter, consists of the categories within a koker. The second layer, indicated with the letter of the category and an element number (for example: 'A1) Beam'). The last layer displays the possible failure mechanisms per element and is indicated by an additional number. This failure tree is part of the failure tree of the entire drainage system of Georgetown (2.5).

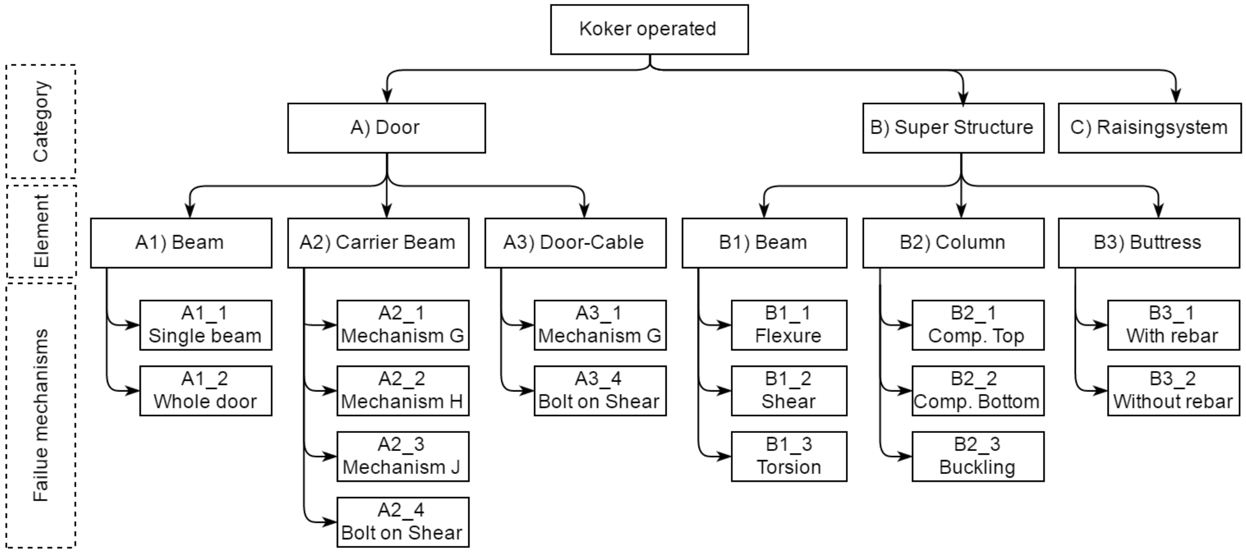


Figure 46 Partial failure tree koker Georgetown (Peter Vijn, August 2016)

In order to determine the failure mechanism with the highest probability of occurrence it was required to check all the possible failure mechanisms per element. These computations were automated in a developed Excel file. The total description of the computations and failure mechanisms can be found in the support document 'designer manual'.

The excel file uses basic probabilistic design to determine the probability of failure of a certain element of the koker. These types of calculations are based on the fact that material properties differ per batch. For example the strength of a first batch of concrete can be slightly different from the second batch, even though the same material is used. This implies that the deterministic value of the resistance of a beam is hard to determine. Instead a range of values is determined with a certain probability to it. The same applies for the load on an element. It was assumed that the used distributions are Gaussian distributions.

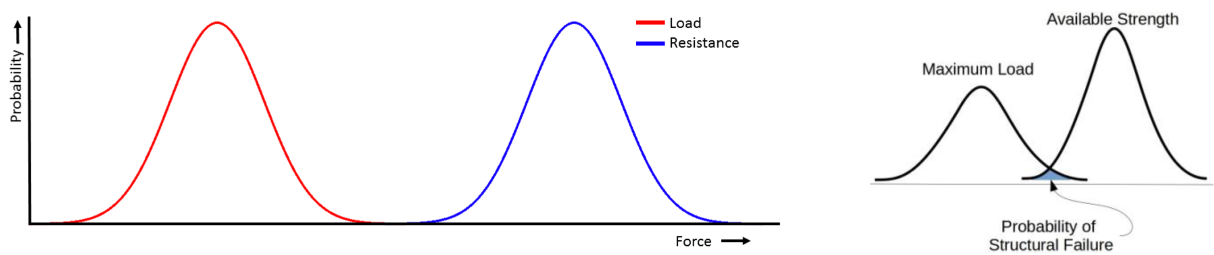


Figure 47 Example of distributed load and resistance (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2015)

When multiple distributed variables are combined (for example combining volume and density for weight) a new distribution is acquired. The parameters for this new distribution are determined by using Monte Carlo to simulate the one leading property. In this example the density is leading, which means the other distribution are assumed to be deterministic.

An example of the calculation for the failure of the total door is given. This was computed using the dimensions of the Mainstreet koker.

Load:	μ_L	$= M_{Ed,A2\_1}$
		$= \frac{1}{16} * \rho_{water} * g * (n_{horizontal\ beams} * h_{horizontal\ beam})^2 * l_{gap}^2$
		$= \frac{1}{16} * 1000 * 9.81 * (12 * 0.3)^2 * 1.34^2 = 14.3\ kN$
	σ_L	$= \frac{1}{16} * \rho_{water} * g * (n_{horizontal\ beams} * \sigma_{horizontal\ beam})^2 * l_{gap}^2$
		$= \frac{1}{16} * 1000 * 9.81 * (12 * 0.011)^2 * 1.34^2 = 0.15\ kN$
Resistance:	μ_R	$= M_{Rd,A2\_1}$
		$= \left(\frac{1}{6} w_{hbeam}^2 f_{t,w}\right) * n_{horizontal\ beams}$
		$= \left(\frac{1}{6} 0.07^2 * 42.000.000\right) * 12 = 411.6\ kN$
	σ_R	$= \left(\frac{1}{6} w_{hbeam}^2 \sigma_{f_{t,w}}\right) * n_{horizontal\ beams}$
		$= \left(\frac{1}{6} 0.07^2 * 2.000.000\right) * 12 = 3.6\ kN$

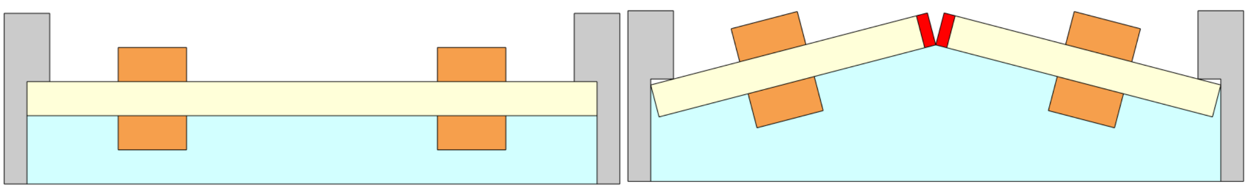


Figure 48 Leading failure mechanism of the koker door (A1_2) (Thijmen Jaspers Focks, August 2016)

The result of this analysis is a list with leading failure modes per element. For Mainstreet koker the leading failure mechanism for the element 'Beam A1' is mechanism 'A1_2) Whole door' (Figure 46). This mechanism has a certain load-resistance distribution**.** Based on this distribution a probability of failure of the door can be determined. As the same method is applied to all failure mechanisms, these probabilities can be compared and a general probability of failure for a koker can be determined. This probability can be compared to the value of other kokers in the area to determine the koker with the highest probability of failure. The total description of the computations and failure mechanisms can be found in the support document 'designer manual'.

5.5.3 Life cycle

The determined probabilities of failure are based on the initial state of the elements. This is due to the fact that the material properties used, are those of a structurally integer element. For example the bending moment resistance of a wooden beam decreases over a period of time. The reduction can be caused by multiple deterioration mechanisms, like the anisotropic shrinkage of wood.

During the lifecycle of an element, this results in a decrease in resistance. At the start of the lifetime ($t = t_0$) the beam is installed and has its full resistance. As soon as the beam is exposed to the environment it starts to deteriorate. In this case it is assumed that the deterioration is a linear process which ends at after some time ($t = t_2$). At this time the beam will have no resistance. On the left side of the graph the distributions of the load and the resistance at initial circumstances, like the distributions discussed around are shown. As the resistance decreases, these two distribution slide over each other. At a certain point in time,($t = t_1$), the distribution start to overlap and the probability of failure of the element starts to increase. From this moment on, the element should be inspected regularly in order to prevent failure.

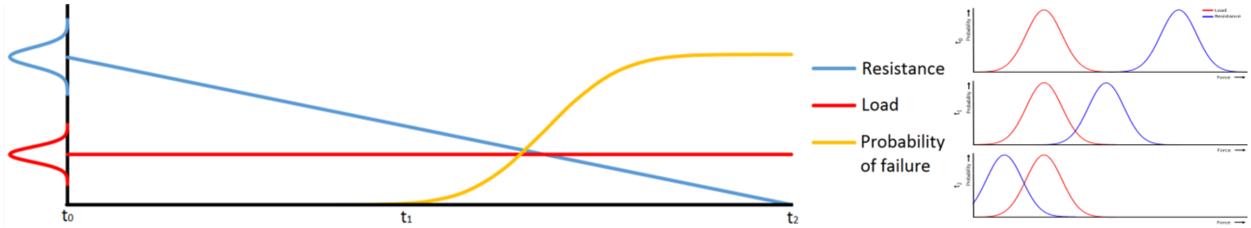


Figure 49 Lifetime of a structure with decreasing resistance over time (Jonkman, 2015)

5.5.4 Inspection

The next step in the assessment of the 'Mainstreet koker' is to determine the rate of deterioration for the different elements. This is done by performing an inspection. Since it is impossible to collect all the data required for an analysis as described before, this step requires a simplification. This is achieved by the student team by creating a pre-defined system with a scale of grades from one to ten. The objectiveness of this analysis is very important in order to achieve comparable results for different kokers. An example of such a grading scale is included below.

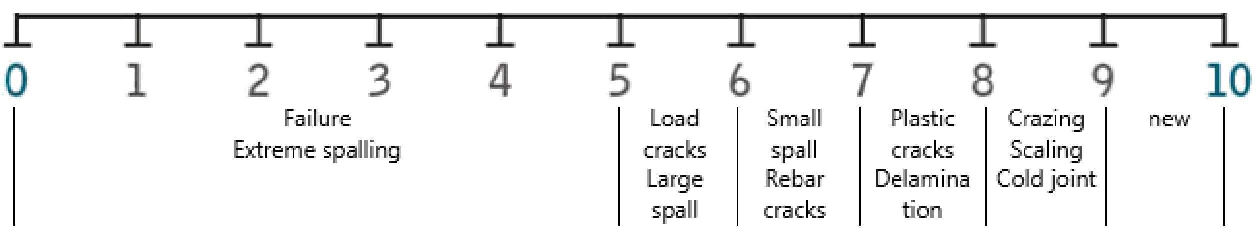


Figure 50 Grading system for a beam on inspection form (Thijmen Jaspers Focks, August 2016)

The terminology used in the grading system is explained in the manual of the inspection form. For each type of deterioration a photo and a textual explanation is provided. This is done in an attempt to maximise the objectiveness of the grades an inspector gives. An example of this additional information is shown below.

Rebar cracks – This type of cracks is caused by the expansion of the corroding reinforcement bars. The small cracks are parallel to the reinforcement bars on the interior of the beam. The cracks can be either parallel to the element, caused by the deterioration of the longitudinal reinforcement, of perpendicular to the element, which is caused by the transverse reinforcement. This will eventually lead to spalling.

Figure 51 Example of deterioration example for objective grading on inspection sheets (Somerville, 2008)

With this grading scale it is possible to perform a visual inspection on a koker without the need for special equipment. An example of such an inspection on the imaginary 'Mainstreet koker' can be found in APPENDIX I. This form includes the grade per element and provides space for comments. For example the columns are graded with an eight, which indicates some signs of deterioration, but not severe. The beam got the grade seven, which indicates some rebar cracks. The shapes of the cracks is parallel to the shape of the reinforcement bars on the interior and indicate deterioration and expansion of these bars, which leads to loss of steel cross area and therefore loss of strength.

Figure 52 Example of a rebar crack worth an 8 on the inspection form (Peter Vijn, August 2016)

5.5.5 Assessment

As the grades per element are determined during inspection, it is possible to create a graph which visualizes load and resistance over time. The distribution of the resistance at the moment of inspection can be derived and the probability of failure per element is obtained. Over time a graph can be created when probabilities of failure are documented over a longer time span.

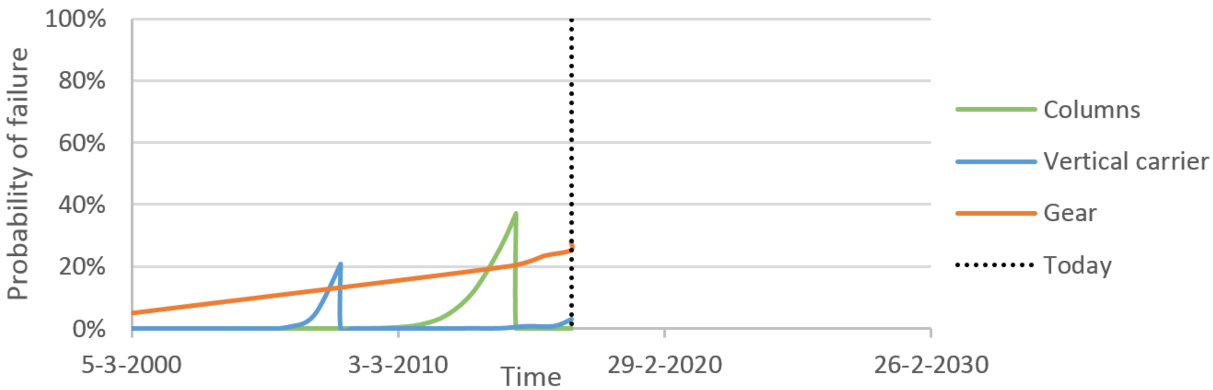


Figure 53 Load and resistance of imaginary 'Mainstreet koker' elements over time (Peter Vijn, August 2016)

The shown lines provide information about rate of deterioration of an element. For example the orange line which represents the gear is slowly increasing, indicating a graduate rate of deterioration since the first inspection in 2000. The line for representing the columns (green) has a spike around 2014, indicating a larger probability of failure. This probability is decreased by some sort of maintenance or replacement.

Another aspect to take into account is the steepness of the curves. While most of the inspected time the failure probability of the vertical carrier beam is close to zero, it increases to a steep line in a period of months. The graph (at a specific time moment) does not show any indication of a future increase of the probability of failure. However this indication could be spotted when considering the grades given to several elements. The sudden increase of failure probability is caused by the overlapping of the load distribution and the resistance distribution when a grade approaches the value 5.

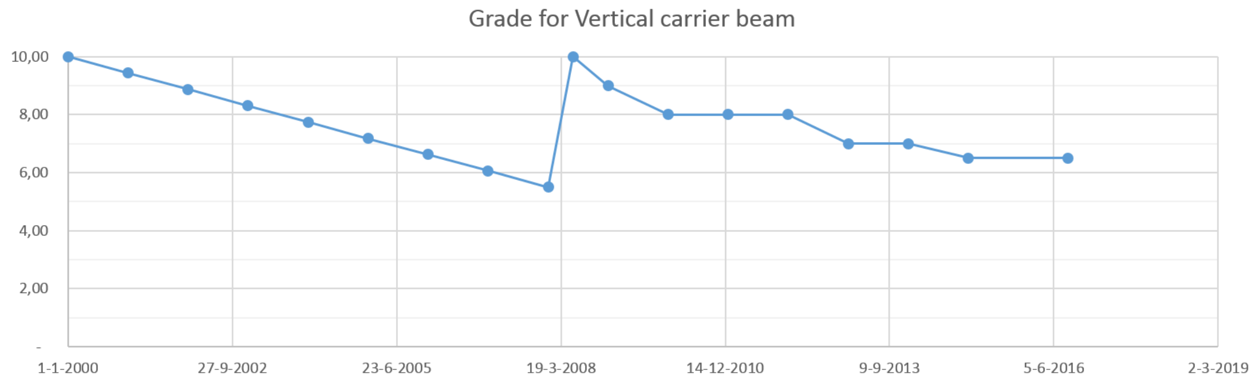


Figure 54 Grades during inspection of a vertical carrier beam over time (Thijmen Jaspers Focks, August 2016)

This system of translating the given grades to a probability of failure is automated in a developed Excel file, which also provides the figures shown in this paragraph.

5.5.6 Maintenance

Separate graphs of the grading of elements do not provide the user of the tool with any knowledge based decision on which elements is most fit for maintenance. Based on the past, this graph indicates that it would be best to increase the grade of the gear. In practise it might be better to improve the quality of another element, based on a prediction of future deterioration.

Predicting the most relevant element for maintenance is accomplished by adding an expected lifetime to the different elements based on experience in the past. For example a vertical carrier beam can have a life span of ten years, while a concrete column has longer lifespan expectancy. When this assumption is combined with the current grade of an element a total failure probability graph can be created which takes life expectancy into account.

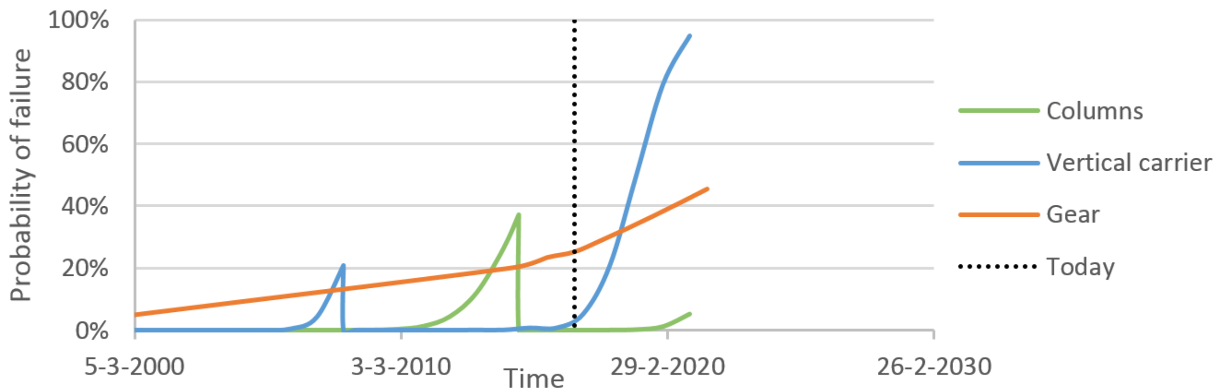


Figure 55 Failure probabilities of different elements over time (Peter Vijn, August 2016)

This graph contains two types of information, divided by the black dotted line. The black line indicates the present. The prediction of the probabilities shows an increase of the failure probability of the vertical carrier in the near future. This indicates the urge for maintenance in order to prevent the beam from failing.

This way of predicting future developments based on the past has its limitations. It cannot be used to determine the exact moment of failure of elements, but it can help to relate the state of the different elements to their lifespan and allows a better comparison of elements regarding the decision to perform maintenance on certain elements.

5.6 Remarks

This paragraph provides a short summary of remarks made regarding the use of the provided method for structural asset management and the developed excel files.

- The goal of the applied method was to develop a tool which can help to quantify the urge of maintenance and therefore be helpful in the decision making on a limited budget.
- The predictions made by the excel file are not exact. They are added to provide a more complete view regarding the possibilities of maintenance. This should be kept in mind during the usage of the excel files.
- The in the inspection form included section regarding the foundation was added to put a focus on signs of seepage. The signs of seepage may not always be visible as the seepage can pass the koker underneath the foundation.
- Besides the visual inspection of the structural components, it is good practise to keep in touch with the koker operator, as this person is the most likely to notice certain types of foundation failure.
- Only the concrete and timber elements were assessed during this project. The steel elements as well as the foundation and bolts of the kokers were beyond the scope. Therefore it is useful to take a better look at these elements.

6 ADDITIONAL OBSERVATIONS

During the project a considerable amount of fieldwork and data gathering has been done. This resulted in an extensive list of observations on the system as a whole as well as some remarkable local observations. The most significant of these observations are shortly documented in this chapter.

6.1 Policy and governance

The maintenance and functioning of the drainage system is only one side of the coin. Behind the technical considerations policy and governance are very important for a good working drainage system. Currently the responsibilities for management of the system in Georgetown and Guyana differ. Also the practices in management of the system changed over the years.

Table 8 Indication of responsibilities on the drainage system for different stakeholders

Organization	Responsibility	
M&CC	The Major and City Council (M&CC) is responsible for all engineering and maintenance works, including the local drainage, the kokers, channels and pumps. Only the koker of Houston North is under responsibility of Ministry of Public Infrastructure.	City of Georgetown
MoPI	Ministry of Public Infrastructure (MoPI) is responsible for the sea defense and the Houston North koker. Furthermore it helps M&CC and NDIA in case of emergencies related to drainage.	City of Georgetown
NDIA	Responsible for drainage and irrigation, helps occasionally in case of drainage issues in the city of Georgetown.	Outside of Georgetown
Private sector	Responsible for drainage system in private estates, for instance the owner of the sugar cane fields.	Private properties
Inhabitants	Inhabitants do not have a responsibility, but may clean their own drains in front of houses. They may also block the drains in the surrounding of their homes due to littering, or due to squatters.	City of Georgetown

The M&CC is responsible for the major part of the urban drainage system. They have only few engineers available for this important task that operate under the lead city engineer. Outside of Georgetown, NDIA manages both the public drainage channels and outfall structures. This happens in close collaboration with the regional authorities.

Works (for instance: bridges, cables and pipes) executed in and around the drainage system often need approval from different agencies. However, several situations were spotted where the drainage system is affected negatively by these works from other authorities or institutions.

Figure 56 Drainage system blocked by other works (Joost Remmers, August 2016)

Besides works done by private or public institutions some Guyanese inhabitants built their houses illegally on the banks of the reach. This has an influence on the conveyance due to high water discharge but mainly poses problems during the maintenance in which these so called 'squatters' might hinder the accessibility of the channel for equipment.

The procedure of discharging changed over the years. In the past discharging happened when the canals were properly filled. In this case the discharge rates are higher than using the current procedure. Currently discharging happens whenever possible. The main motive for this transition is the fact that the situation upstream in the drainage system is unknown to the operators of the outfall structures. To avoid a risk of flooding discharging happens whenever possible.

6.2 Technical remarks

In the several chapters on the local drainage system, primary channels and outfall structure already some technical remarks were made which were of major importance to the analysis. Besides of that some elements require some additional context.

6.2.1 Litter

An obvious but important observation is the presence of a severe amount of litter in both primary channels and the local drainage system. This litter differs from small packaging, to organic waste but also large items like refrigerators. In some cases this hinders the conveyance and could contribute to additional serious problems like diseases and stench. Besides of this, there are no possibilities to remove the litter systematically, like trash racks within the drainage system.

Figure 57 Refrigerator in front of culvert of East Bank Public Road bridge (Joost Remmers, August 2016)

6.2.2 Siltation

Siltation occurs near outfall structures and primary channels. Especially near the kokers and behind these structures a severe amount of sedimentation is present. Earlier it was said that the way of discharging changed from large amounts at one time to continuous discharge whenever possible. Earlier a procedure called 'flushing' was used in which the primary channel is filled as full as possible and when full the koker was opened and sediment was flushed out of the system. This does not happen anymore and this could be one of the causes of siltation.

In chapter 4 it was noted that interventions were made in the past on the discharge of the Houston North channel. Two trenches were dug between the Houston North and South-Ruimveldt primary channel. Besides that the outfall channel of the catchment Houston North koker was silted up and therefore a structure with two culverts was made to connect the Houston North channel to the outfall of South-Ruimveldt channel on the Demerara. These culverts have again silted up over the past year. During field observations it was noticed that there are signs of seepage near the culverts.

Figure 58 Silted up adjusted koker of Houston North (Joost Remmers, August 2016)

6.2.3 Local drainage system

In the local drainage system some discrepancies can be found between the drainage system and other (public) infrastructure. Roads and wards are sometimes present below the bottom level of the drains. At some locations this can lead to problematic situations as for instance at Blue Mountain Road. This street has the nickname 'Water Street' because at mild rainfall this street immediately floods.

6.3 Data management

One of the main recommendations of the DRR report of 2015 was to improve the methods of data management in Georgetown. Currently the data is not centralized at one location and is divided over several organizations. This limits the use of this data and the development of knowledge on the system. The datasets gathered and developed during the execution of this project will be transferred to the University of Guyana. They will start gathering, managing and distributing the data on the drainage system.

6.3.1 Mapping the system

The catchment areas as they are stated in the current documents are not in accordance with the reality. For instance: several connections of South-Ruimveldt with other catchment areas are not documented and seem to be unknown by the responsible engineers as well. Also, there is a connection between the South-Ruimveldt and the La Penitence catchment areas, near the crossroad of Mandela Avenue and Arapaima Street. To make profound decisions on possible interventions on the system, the responsible engineers should have a complete insight and full knowledge of how the system looks like.

Different maps are available at this moment but none are fully suitable for getting an understanding of the drainage system. Especially the connections between primary channels and the location of culverts connecting the local drainage system to the primary channels are not documented on these maps. Better mapping of the system is essential for effective interventions, making hydraulic models or understanding of the system.

6.4 Long term strategy

To obtain a durable drainage system, a long term strategy for the drainage system should be produced, with specific attention for Operations & Maintenance. Several important processes should be taken into account that can influence the drainage system. These are for instance the subsidence of Georgetown, the development and growth of the city and changes in rainfall patterns due to climate change. In this strategy the desired situation of the drainage system can be stated. The ideal method of drainage should be researched, so that future projects within the system do not stand alone, but are part of a strategy with a vision.

The ideal drainage method for the long-term is not necessarily the same as it is now. It is not sure whether drainage under gravity can be used. When looking at the current situation, even in dry periods, the water levels in the wards are still relatively high, more or less the same as the roads. It could happen that in the future pumping is the only option to lower the water levels. When this is the case, it is important to take measures as early as possible.

7 RESULTS

It is essential to notice that the DRR reviewed the whole system and recommended not to consider new, large scale, expensive infrastructure but instead advised to take a large number of small steps that will increase eth knowledge and collective ownership of the drainage infrastructure. The current students-project is intended to be one of those steps forward in developing knowledge on the system. This has a consequence that deliberately conclusions on the whole system will hardly be made and both results and recommendations focus mainly on improving the knowledge.

7.1 Local drainage system

During the project an extensive analysis has been made of the local drainage system (from rainfall to discharge into the primary channels). Summarized, this analysis delivered the following results:

1. A user friendly hydrological analysis has been made which helps determine converting rainfall parameters into discharges into the local discharge system for different land uses and return rates.

2. A detailed field observation and a spreadsheet model were made and help gaining more insight into the dynamics of the system. This can be used to assess the effectiveness of local drainage systems.

3. It is found that the local discharge areas dewater in a delayed manner. Therefore the most upstream sections of the areas are more prone to flooding. Measures in the local drainage system are essential for providing flood safety.

4. From table 6 in paragraph 3.5.6 it can be concluded that dredging of local drainage system seems to be the most effective measure to reduce inundation. It should be noted that this scenario does not take the groundwater level into account.

7.2 Primary drainage channels

In short the analysis on primary channels, mainly executed using the hydraulic model HEC-RAS, delivered the following results:

1. A hydraulic model was developed in which unsteady flow computations on the South-Ruimveldt primary channel can be executed. Together with this model a hydrological analysis and a manual for model making in Georgetown was delivered.

2. The hydraulic model is suitable for modelling interventions in the system of which eight were executed during this project as an example.

3. With a calibration campaign it was shown that a sufficiently accurate model of reality can be made of a single catchment area in HEC-RAS.

4. It was proven that boundary conditions can be simplified without losing their characteristics and that this is possible within the modelling capabilities of HEC-RAS.

5. Of the eight modelled interventions four have a significant effect; cleaning the channels, adding a pumping station to the koker, closing the connection of South-Ruimveldt with Houston North and adding a storage area seem to have effect on water levels or discharge (time).

7.3 Outfall structures

Developing a method for structural asset management for the kokers of Georgetown along the Demerara river delivered the following results:

1. A method for structural asset management which provides the user a tool to keep track of the rate of deterioration of (different elements of) the kokers. It provides information to make decisions based on computations. Support manuals and tools for Guyanese stakeholders are included.

2. Datasets were made containing locations, dimensions, photos and inspection forms on all individual kokers in Georgetown. Methods to expand these datasets as factsheets and inspection forms are included.

8 CONCLUSIONS

In this chapter the conclusions that can be drawn for this research will be carefully explained. First of all it must be noted that this report was drawn as a start for knowledge based decision making on the drainage system in Guyana.

8.1 General conclusions

Some of the general conclusions which can be drawn from the work and time in Guyana are given in this paragraph.

1. Engagement of the Georgetown community is large. However, in practice this is not always visible in the action of individuals. The recommendations of the DRR report on governance and the social side effects of the drainage system are more relevant than ever.

2. Pollution of the local drainage system and primary channels should be either prevented or the effects of it should be mitigated.

3. The management of the drainage system in Georgetown is difficult due to the different stakeholders and the knowledge that is available is not used effectively.

8.2 Local drainage system

The model made on the local drainage system was made and several example scenarios were executed. Based on these scenarios and the process of model making some conclusions can be drawn.

1. The storage based principle model was used effectively to prove the importance of the local drainage system. A malfunctioning local drainage system in itself can cause flooding in neighbourhoods.

2. Increasing the local storage capacity could maximize the improvement of discharge times of the local drainage system. Please do note that the feasibility of execution and the groundwater levels have not been considered at this stage.

3. Cleaning the culverts is the second most effective measure but it is less costly and has less spatial impact then improving the local drainage system as a whole.

8.3 Primary drainage channels

From the development of the hydraulic model and the example scenarios which were analysed in paragraph 4.4 some preliminary conclusions can be made on the primary drainage channels. However, it must be noted that the model is not calibrated for high water scenarios.

1. HEC-RAS can be used for the visualization of the effect of different interventions in the drainage system. A clear model structure which can be implemented in other areas is available for an hydraulic model on a primary channel in other catchment areas.

2. It is not possible to model the floating litter in the channels and examine the effect of it on the drainage system. Besides that the fine sediments are hard to examine as well within the current capabilities of the model and the persons responsible. This does not mean that these effects are not relevant.

3. Closing the connection of the South-Ruimveldt channel and the Houston North channel could lead to a considerable decrease in water levels and discharge times. However, the effects on Houston North have not been considered in this case.

4. Cleaning the primary channel is a very effective measure to increase the discharge and decrease the water levels over time.

5. Adding a pumping station at the koker is effective on increasing the discharge capacity.

Please do note that it is necessary to complete the integral design loop for all these measures. This includes the cost aspect and the reliability of different measures.

8.4 Outfall structures

From the analysis on the structural assessment tool for the outfall structures several conclusions can be drawn.

1. The current level of performed maintenance is adequate, resulting in heavily deteriorated structures.

2. The performed maintenance of the system is mostly reaction-based instead of prevention based.

3. A well defined approach on whether to apply maintenance or not is missing, as well as a proper documentation of the history and the current state of structures.

4. The current deterioration rate of the structures is not monitored properly (e.g. due to the lack of inspections).

5. The available means (budget wise and staff wise) are insufficient to perform better maintenance.

9 RECOMMENDATIONS

After the extensive time that was spent in Georgetown for the execution of this project several recommendations were formulated. These focus mainly on the improvement of knowledge on the system and improving the capacity of knowledge based decision making. First some general recommendations will be given which apply to the whole drainage system (9.1). Afterwards more in depth advices are formulated on the local drainage system (9.2), primary drainage channels (9.3) and outfall structures (9.4).

9.1 General recommendations

Some of the recommendations can be applied on all the three elements of the project. Therefore they are added separately below.

1. Upgrade modelling capacity in order to use model outcomes for knowledge based decision making to come up with the best measure to improve the drainage system.

2. Use the capacity at the University of Guyana to start building models of the city of Georgetown. Different catchment areas can easily be modelled during a graduation topic, and can subsequently be used in a larger model.

3. Assure that authorities take into account the effect on drainage whilst deciding on interventions in and around the drainage system to prevent negative effects of these interventions on the conveyance and discharge capacity.

4. Develop new mapping and information systems of Georgetown's catchment areas. Make a sound analysis of interconnected areas between catchment areas. Identify culverts, siphons and channels between areas to get a complete overview. Map all these observations in one new general map of Georgetown and one map per catchment area.

5. Gather more information on the distribution of rainfall over space by using rainfall measurement stations spread around the city. The distribution of rainfall intensity over space can then be analysed and used in the modelling.

6. Most of the additional observations could be a direct consequence of a lack of awareness. This has already been treated extensively in the report written after the DRR mission of 2015 and was not a major subject of this project. However, it should be noted that whatever measure is taken, awareness of people for their drainage system is essential for successful implementation.

9.2 Local drainage system

A spreadsheet model was made of a specific catchment area in the local drainage system. Multiple improvements of the system where investigated using the model. The recommendations are listed below.

1. The spreadsheet model should be calibrated, since the current model is only verified by visual inspections. The calibration implies measuring water levels and outflow inside the areas over time, and compare it with the outcomes of the model. It is recommended to do this calibration in relatively 'clean' systems, since the blocked systems may be difficult to calibrate.

2. Since the current model assumes a flat system with no gradient, the influence of the natural slope of the wards is not taken into account. This should be done by measuring the surface elevations of the wards and implement it into the model.

3. The model should be improved by taking into account the overflow of banks in order to make the inundation predictions more accurately. The present model only works within the volume of the local channels. Once the water level reaches the maximum depth of the channels, the water level will keep on rising as it was contained by the initial channels.

4. Accurate rainfall data and storm cases are necessary. In this report a single design storm was determined. However it could be that other types of rain showers pose a higher load on the local drainage system. Therefore the rainfall analysis has to be evaluated with relevant stakeholders in order to find the most leading cases for local drainage systems.

5. Damages of floods need to be determined, both in terms of the economic and the social aspects. It is recommended that an additional socio-economical study is performed to investigate which damages are actually present.

6. The analysis with the developed model of catchment area 7 of the South-Ruimveldt area should be finished on short term. This can be done by including the costs for each measure into the model, and evaluate which measure to execute, taking into account costs, effectiveness and feasibility.

7. The analysis with the developed model should be extended to other zones of South-Ruimveldt. The following steps should be taken:

- Make a full system analysis of each area to be included, including geometry (heights) and failure mechanisms
- Determine the possible measures to improve drainage
- Check the costs, feasibility and effectiveness of the possible measures with the calibrated spreadsheet model

9.3 Primary drainage channels

The analysis on the primary drainage channels mainly consisted of the development of a modelling approach for a catchment area in Georgetown. During the analysis the following recommendations were developed. They consist of recommendations for improving the example model of South-Ruimveldt (9.3.10) and of recommendations for implementing hydraulic modelling in other catchment areas (9.3.2).

9.3.1 Improve the existing model of South-Ruimveldt

The model that was made on the catchment area South-Ruimveldt and consists of all steps needed to make a model which can be used to estimate the effect of different interventions. However, for further use or more accurate estimations the following recommendations are advised:

1. Expand geometric data of the most upstream part of the South-Ruimveldt channel. Furthermore, the geometric data (heights) of the connections between secondary and primary channels should be measured.

2. Implement the Liliendaal catchment area in the model. Currently, a basic assumption is made on the influence of the Liliendaal pump station on the South-Ruimveldt channel. This can be made more reliable by implementing the Liliendaal catchment area.

3. More calibration steps should be performed in order to make a more reliable model Especially calibration with measurements made during high discharge (4.3.4).

4. Execute sensitivity analyses after the model has been expanded. This can help verifying the strength of the basic assumptions that were made initially (4.3.3).

9.3.2 Implementation of model in other catchment areas

The same approach that has been used in the catchment South-Ruimveldt can be used for other catchment areas as well. The following recommendations are advised:

1. Set a clear goal for the model before the actual modelling starts. The making of a model is not a goal on itself. The goal can be to understand the system and find the weak spots. It can also be used to estimate the effect of possible interventions in the system.

2. Build the model in a structured manner. The first set up should be simple (steady flow) to get an idea of the system. After that a more complex model can be build (unsteady flow) and can be calibrated. More information can be found in the provided modelling manual with this report.

3. The user should be able to criticize the results. Exact data from the hydraulic models output might not be reliable and therefore the model might be more useful in comparing the effect of different interventions.

In APPENDIX H a full plan was made for implementing the modelling approach in different catchment areas. In combination with the provided manual document this appendix can be used for the development of hydraulic models in other areas.

9.4 Outfall structures

During the project a structural assessment tool was made for the kokers in Georgetown along the Demerara river. Effective improvements on the structural design of the kokers should be the result of the use of the assessment tool. A couple of recommendations are given for improvement of this tool:

1. Gather data on the total lifespan of koker elements and implement these in the tool to improve the prediction of remaining lifespan.

2. Expand the tool by including missing elements like the wood works leading to the wing walls, steel beams and brick work elements.

3. Analyse the cost of replacements and maintenance and implement them in the current method to investigate the most cost effective improvements.

4. Consider the design of protective elements and cleaning of the koker door. This could result in an extended lifespan of the koker elements. This can be examined with the structural assessment tool.

5. Consider the tactically scheduling of maintenance in periods where the probability of failure is lowest (for instance during dry season). Take this into account while using the structural assessment tool.

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APPENDIX A. ADMINISTRATIVE NOTES

In this appendix all stakeholders, contributors and data sources are mentioned for further reference to the reader.

Table 9 Contact information of student team Delft University of Technology

Name:	Responsible for:	E-mail
Joost Remmers	Primary channels + final report + process	joostremmers@gmail.com
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Jos Muller	System analysis + Local drainage system	jrmmuller@live.nl
Thijmen Jaspers Focks	Outfall structures + manuals	thijmensjf@hotmail.com
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Martijn van Wijngaarden	Primary channels + manuals	mjpvanwijngaarden@gmail.com
General contact information: www.bouwmij.org info@bouwmij.org		

Table 10 Contact information of partners from the Netherlands

Organization	Project Role	Contact Information
CDR International (Coasts, Deltas and Rivers)	Sponsor	Dirk Heijboer d.heijboer@cdr-international.nl Eelco Bijl e.bijl@cdr-international.nl
Delft University of Technology	Supervisor Multidisciplinary Project	prof. dr. ir. Winterwerp
		ir. Pasterkamp
		Yolanda de las Heras
RVO – DRR Team	Initiator of the project	Arjan Braamskamp
		Fokke Westebring
		Rob Steijn
Dutch Embassy Paramaribo, Suriname	Initiator of the project	Ernst Noorman
		Maurice Pourchez
		Heine Lageveen
Dutch Honorary Consul, Georgetown, Guyana	Emergency contact	Ben ter Welle

Table 11 Contact information on partners from Guyana

Organization	Contact Information	Project role
National Task Force Secretariat	Dr. Sewnauth Punalall	Initiator and Supervisor
	Major General Joseph Singh	Chairman and supervisor
	Lennox Lee	General contact
National Drainage and Irrigation Authority	Christine Mohammed Douglas	Protocol Officer
	Frederick Flatts	Supervisor
	Dave Hicks	Primary Contact (Engineer)
	Jeremy Douglas	Primary Contact (Engineer)
	Rudolph Persaud	Engineer
Ministry of Public Infrastructure	Jorge Linton	Primary Contact (Engineer)
	Colin Gittens	Sea Defence Engineer

Mayor and City Council	Colvern Venture	Primary Contact (City Engineer)
	Kenson Boston	Engineer
University of Guyana	Maxwell Jackson	Primary Contact (Lecturer)
	Colin Quintyn	Student
	Nicole Eastman	Student
SRKM	Gregory Williams	Consultant
Ministry of Natural Resources	Haimwant Persaud	GIS Expert
HydroMet	Dr. Garvin Cummings	Primary Contact (Director)
	Komalchand Dhiram	Engineer

Table 12 Attendance list of knowledge transfer days

Organization	Name	E-mail adres
NDIA	Jermy Douglas	jermeyldouglas@gmail.com
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APPENDIX B. ADDITIONAL MATERIAL

Besides this final report some additional documents, datasets and files were made to clarify and explain some of the steps undertaken. The final reports, manuals, datasets and developed models and will be handed over to the University of Guyana who will keep it under supervision.

B.1. Local drainage system

The local drainage system analysis (including the hydrological analysis) also provided some tools and datasets which can be used for further investigations.

Table 13 Additional documents with the analysis of the local drainage system

Documents	Tools	Datasets
Final report	Excel tool for local flood risk	Rainfall data (HydroMet)
		GIS maps
		Satellite pictures

B.2. Primary channels

The primary channels analysis and especially the HEC-RAS modelling required some extra documents and datasets. Especially the manual for model making in Georgetown is an important document for further use.

Table 14 Additional documents with the analysis on primary drainage channels

Documents	Tools	Datasets
Final report	Discharge tool	GIS Maps
HEC-RAS v4.1 Application manual: Georgetown Drainage Channels	HEC-RAS folder structure	Base model structure
		Tidal data
		LIDAR data
		Eight models with interventions

B.3. Kokers

The analysis of the kokers delivered a structural asset management tool which can be used for prioritizing maintenance. The following documents, tools and datasets are delivered together with this final report:

Table 15 Additional documents with the koker analysis

Documents	Tools	Datasets
Final report	Designer excel tool	Photographs
Manual for design tool	Maintainer excel tool	Dimensions per koker
Manual for maintenance tool		Filled factsheets
Manual for inspection forms		Filled inspection forms
Template inspection form		Map koker locations

B.4. Knowledge transfer days

On the 8th and 9th of September 2016 two knowledge transfer days were held. This was done as an addition to this report and the final presentation (given on September 7th). The attendee list of this program can be found in appendix A. Below the program of the knowledge transfer days can be found. Example models, tutorials and presentations were used to give a kick-start to the implementation of the executed work in Guyana. Below the program of these two days is given to give the reader more insight in the content of these days.

Table 16 Schedule for knowledge transfer days on September 8th and 9th, 2016

Nr.	Time	Day	Title	Description
1	09:00 - 12:00	Sept 8th	Introduction to HEC-RAS	In this session we will discuss the working of the computer program HEC-RAS with all the used components. All the attendees are supposed to be able to build a simple model at the end of this morning.
2	13:00 – 16:00	Sept 8th	Application of the hydraulic model	In this session we will discuss the boundary conditions, assumptions, difficulties and the interpreting of the model when it is being used for a drainage channel in Georgetown.
3	09:00 – 12:00	Sept 9th	Design process and local drainage system	In this session we will discuss the design process that could be used for the drainage system, together with an applicable method of a flood risk analysis. Besides of that, the analysis of the local drainage system will be mapped out.
4	13:00 – 16:00	Sept 9th	Structural asset management tool	In this session the background theory and the setup of the structural asset management tool for the outfall sluices will be discussed. Besides, an example application of the tool will be done.

APPENDIX C. CONTENT DRR REPORT

Below parts of the executive summary of the report, which was written after the mission executed by the Dutch Risk Reduction team, are presented. It includes their main recommendations and if the item before the recommendation is a ✓ it indicates that the recommendation was included in this project.

C.1. Elements of summary report

The Government of Guyana has requested the Government of the Netherlands to advice on their drainage situation, both for Georgetown and the low-lying agricultural coastlands. The official request from the Guyanese Ministry of Public Infrastructure was sent to the Netherlands Embassy in Suriname on 03-08-2015. In this letter, it was also requested to comment on the coastal defence strategy, but it was decided during the preparatory telecoms and the kick-off meeting to focus on the drainage problems. It was decided by the Dutch Government to follow-up the request by means of a scoping DRR – Team mission addressing the flood risk management in the northern coastline of Guyana, and Georgetown in particular.

The objective of the mission was to specify what can be done to better operate and manage the drainage system of Georgetown and the low-lying coastal areas.

Considering the economic situation of Guyana and the relatively mild character of the flooding events under normal conditions, it is not recommended to consider new large scale, expensive infrastructure. Instead, it is advised to take a large number of small steps over a period of several years that will increase the knowledge and the collective ownership of the drainage infrastructure among local experts, Guyanese governments, and the people of Guyana. By increasing trust, cooperation and local expertise Guyana can become a South-American example of effective and efficient water management

This report provides concrete suggestions to make the Guyana approach towards water management in general and drainage in particular more integrated and more proactive. The suggestions cover a wide palette of topics and include:

1. Upgrade Modelling Capability
   - Make a long-term project plan to gradually develop the hydraulic drainage model for Georgetown, with the design requirements mentioned in Section 3.2.
   - Set up a simple spreadsheet type of network model for the entire drainage system of Georgetown and use it to better understand the flow of water. Use this understanding to support project proposals (for example increasing the pumping capacity of the most northern outfall koker along the Demerara River).
   - ✓ Start selecting two or three engineers with a passion for computers and modelling and train them on the subject of hydraulic modelling.

2. Improve flood resiliency of people
   - Develop a communication plan with the aim to increase the understanding of the people about what it means to live with water (in terms of potentials and challenges) and execute this plan. It has to be clear that the flood risk will never be reduced to zero. Consider to use a shared symbol, for example the water lily.
   - Make a flood hazard map of Georgetown and use it to explain to the people why it is important to build their properties (houses and businesses) flood-proof.
   - Prepare a simple explanation (for example, a Youtube video) on how the drainage system works, why water needs space, and why it is important to keep the drainage system free from constructions and solid waste.

3. Upgrade small-scale floating dredging capabilities
   - Specify the requirements for small scale floating dredgers for the city of Georgetown and justify the investment based on a cost/benefit calculation. Decide on whether it should be a public or a private entity to run the "City Dredging Operations".
   - Purchase dedicated equipment and start operations. Evaluate the performance on a regular basis.

4. Develop and apply rational risk approach
   - ✓ Prepare a first set of flood hazard maps for a region yet to be chosen (for example one isolated catchment area in Georgetown). Next steps are to prepare flood hazard maps for other areas as well, including rural areas.
   - Set up the framework for analysis for the sea defence risk assessment using the Rational Risk Approach briefly described in Section 3.5. The items mentioned under 'national debate' in Section 4.1 should be part of this activity.

5. Pilot "Living with Water"
   - Develop a pilot "Living with Water" in which all elements of an integrated long-term and holistic "Drainage System Management" are specified and made applicable to Guyanese situations. One pilot location could be chosen in consultation with GuySuCo (low-lying coastal area with planned or unplanned urban development on formerly rural lands). Involve different governmental agencies to develop structural ways of cooperation;
   - Idem, but now for an existing highly urbanized catchment area in Georgetown.

6. Asset Management
   - ✓ Consider the suggestions given in the Table in Section 3.7 on Asset Management.

7. Data Management
   - ✓ Start collecting all available data on the drainage system (Georgetown and elsewhere), digitise, and apply gap analysis to see what misses. Start collecting and digitising these missing data. This includes data on locations of canals, kokers and pumps, their dimensions, capacities, flow velocities, bed composition, embankment composition, etc).
   - ✓ Start collecting all relevant hydro-meteorological data that is required for a risk assessment (of the drainage system as well as the sea defence system – see Section 3.5). Use a pre-set format for such data collection and store it in a national central data base. Apply gap-analysis to see which data is missing.
   - ✓ Use geo-informatics to collect data on land use, long-term shoreline dynamics (mudbanks), and flood events. Store these data in a fixed format in the central database.
   - ✓ Start analysing the data in a consistent manner and contributing to better understanding of the flood risks. LIDAR data in combination with land use data can be used to prepare flood hazard maps. Long-term rainfall data (GuySuco) can be used to determine the frequency of occurrences of extreme rainfall events, which serves as input for the risk assessment.

8. Technical short-term improvements
   - ✓ Consider the technical upgrade options listed in Table 3.2;
   - Consider improving the hydraulic efficiency by streamlining corners of drainage canals

APPENDIX D. TOPOGRAPHICAL DATA

One of the major difficulties in working on the drainage system of Georgetown is retrieving proper maps topographical data on the system. Several sources where used during the process and the student team mostly processed the data themselves using open source GIS software (QGIS Development Team, 2016).

D.1. Excel map of Georgetown

The M&CC is in possession of a map made in excel showing drains and canals in great detail. It gives insight in how the system is set up throughout the city but also in parts of the catchment areas. However, during fieldwork the accuracy and relevance of the map was checked and it does not resemblance the system as it is functioning currently.

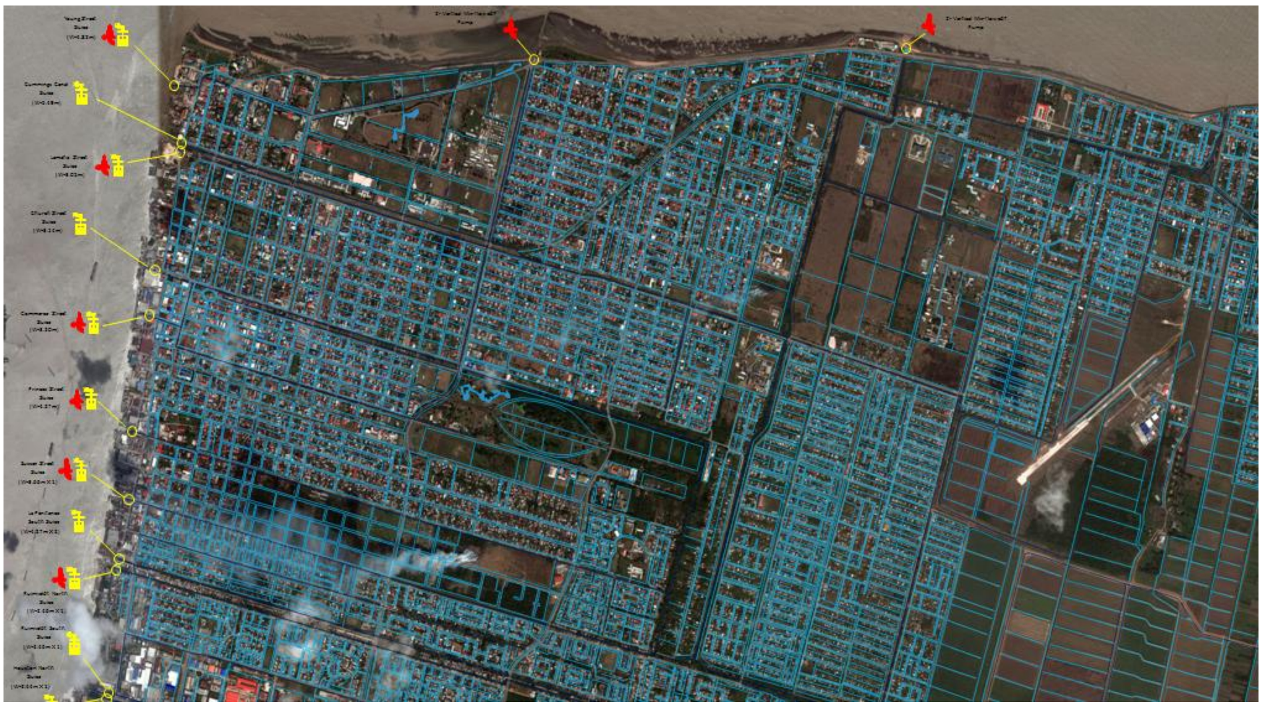


Figure 59 M&CC Excel map of Georgetown

D.2. LIDAR Data

In 2002 a firm was hired to make LIDAR elevation and surface data using an aeroplane above the region of Georgetown (Icaros Geosystems B.V., 2010). The data is available but barely used by the authorities. During the project a depth elevation map has been retrieved from this data. Using this data the drainage areas and natural drainage lines which were the runoff and discharge analysis were determined.

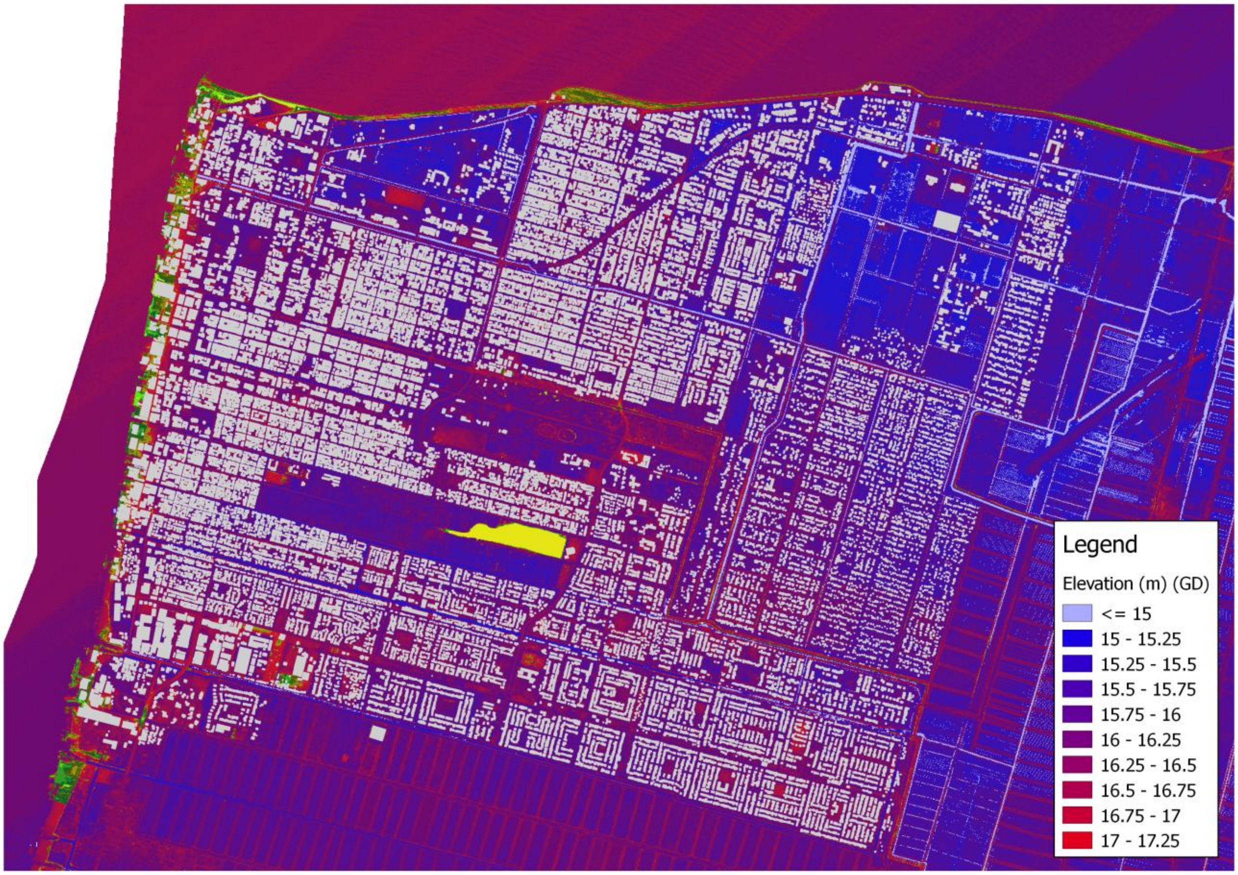


Figure 60 Depth elevation map of Georgetown (m) with respect to GD (Icaros Geosystems B.V., 2010)

The material that has been used to define the discharge areas, natural drainage lines and inflow points for the hydraulic model also finds its origin in the LIDAR data. Raw GIS files were retrieved from the Ministry of Natural Resources and used to define the discharge areas and natural drainage lines. These contour lines can be created using basic plug-ins in QGIS for different resolutions. Three levels of detail are available (349k, 40k and 20k). Both the 349k and 40k map were used for defining the discharge areas. The natural drainage lines are based on the land profile and its corresponding discharge lines with the lowest elevation (Persaud, 2012).

D.3. Openstreetmap

Map layers were exported from Openstreetmap to make both maps of the drainage channels and retrieve information on land cover for the runoff analysis (Openstreetmap, 2016). These layers were edited and extended with data for spatial analysis in GIS (QGIS Development Team, 2016).

Figure 61 Openstreetmap map with used layers for runoff coefficient (Openstreetmap, 2016)

D.4. Older data

Especially the Georgetown Water and Sewerage Master plan (Halcrow, 1994) gives some valuable information on the location, names and gathered topographical information on the catchment areas. These maps are used by the M&CC, NDIA and MOPI during work in the city.

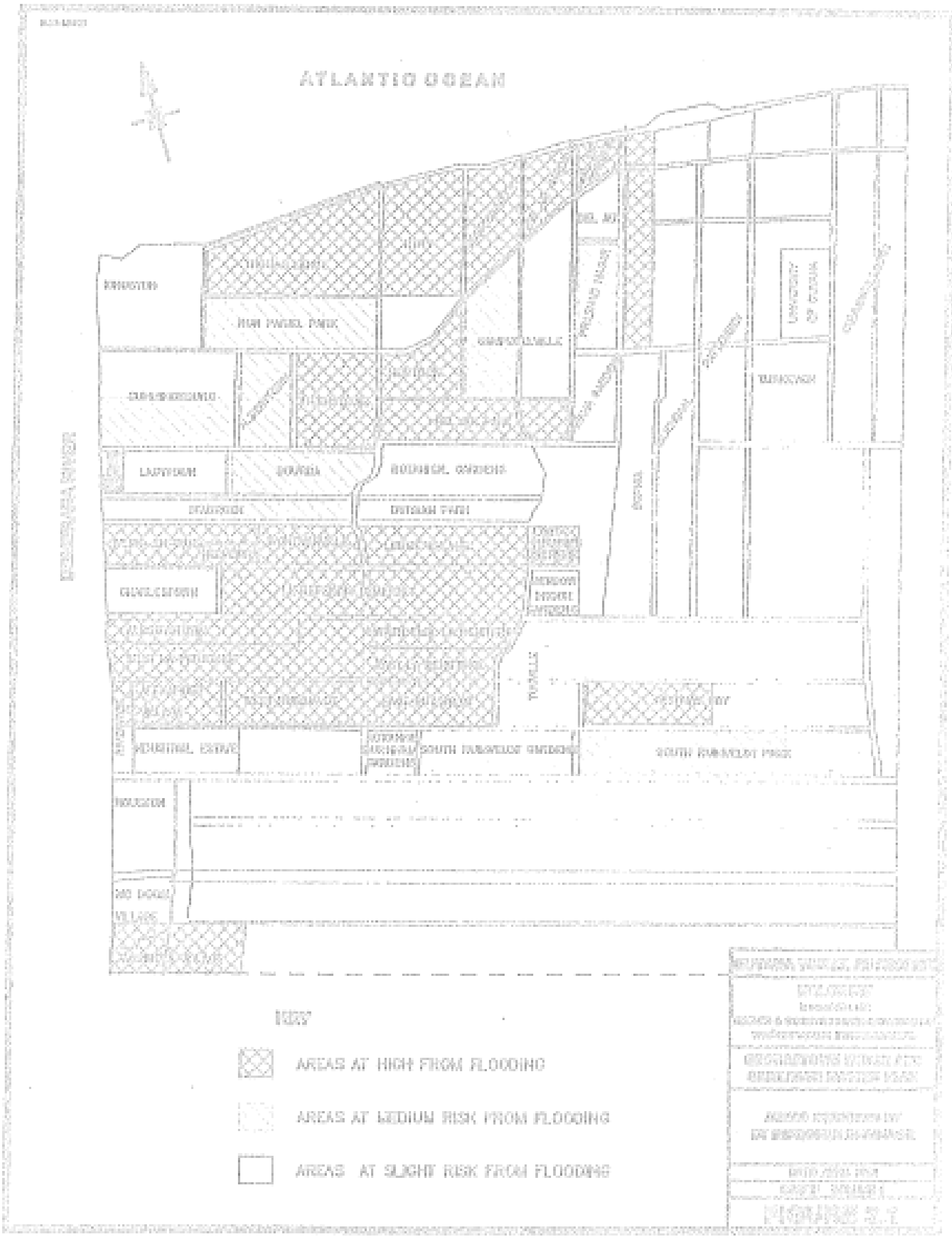


Figure 62 Flood risk map from 1994 (Halcrow, 1994)

APPENDIX E. DESIGN/RISK APPROACH

On request of the Guyanese stakeholders more clarification is given in this report on a sound design and risk approach which can be used for knowledge-based decision making. In this appendix some examples and basic structures are given. First the integral design loop is discussed (E.1). Secondly the flood risk analysis is described (E.2). Thirdly the failure tree which was developed (also part of the flood risk analysis) is given (E.3). Finally a method for economic optimization is described (E.4).

E.1. Integral design loop

Below an overview of how the three project parts (local drainage system, primary channels and outfall structures) can be implemented into the integral design loop (2.4).

Table 17 Incorporation project elements in integral design loop

Step	Included	Local drainage system	Primary channel	Kokers
Scope		Zone 7 in catchment area South-Ruimveldt	The South-Ruimveldt channel, next to Caneview Avenue	All kokers in Georgetown along the river (applicable on kokers elsewhere)
Analysis: Determine scope, causes and characteris tics of the system	Included in the report	The first two steps of a Flood Risk Analysis are made: system assessment and hydraulic boundary conditions. System assessment focuses on the geometry and the state of the system. Both will be presented in chapter 3. A qualitative system analysis (failure tree) is made, which is part of chapter 2.	A system assessment is made, which was mainly to find out the geometry of the system. Also, the hydraulic boundary conditions, such as inflow from the local drainage system and tide data, were included. Both will be presented in chapter 4. The qualitative system analysis (failure tree) in chapter 2 also included the primary channel.	First, the dimensions and the materials of the kokers are determined. Afterwards, the resistance of the kokers is calculated. Inspection sheets are made based on grading of elements. Finally, a way to compare the results and to make a choice which koker to maintain first is presented.
	Not included in the report	A quantitative system analysis, consequence estimation and the determination of the flood risk are not included in the report.	-	-
Synthesis: Generate possible measures to solve the problem	Included in the report	Based on the failure mechanisms, some possible measures are determined.	Based on the failure mechanisms, some possible measures are determined.	-
	Not included in the report	-	-	Possible measures to improve the kokers are not included in this report.
Simulation: Find out the effects of the measures on the system	Included in the report	The effects of the measures on the system are tested with an Excel spreadsheet. This spreadsheet will be delivered.	The effects of the measures on the system are tested with the model HEC-RAS. This model will be delivered.	-
	Not included in the report	-	-	The synthesis is not made, so the successive steps cannot be made.
Evaluation: Compare the measures	Included in the report	The results are presented next to the present situation and each other.	The results are presented next to the present situation and each other.	The evaluation step from the analysis can also be used in the final evaluation.
	Not included in the report	There is no judgement on which measure to take. A tool to do this will be presented.	There is no judgement on which measure to take. A tool to do this will be presented.	The simulation is not made, so the successive steps cannot be made.
Decision		This has to do with policy and is not included in the report.		

E.2. Flood risk

Flood risk or the risk of flooding is an indication which areas are the most endangered by flooding combined with the highest consequences. It gives an insight which areas are more at risk when flooding occurs and where the current flood defences do not offer an acceptable level of protection.

To provide sufficient protection, large investments often have to be made. It is therefore beneficial that these are as efficient as possible. Important is the consideration which areas have the highest change of flooding and where the damages are the most severe. This insight can be gained by determination of the flood risk of these vulnerable areas.

A risk is expressed in a change or probability of an event occurring and a corresponding consequence. In case of flood risk, the event that leads to flooding is the failing of a certain flood defences and the consequence is the damage that occur due to the flooding (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2015).

The probability of failure is expressed in the amount of time it is expected this failure will occur. For instance it is expected that a current flood defence will fail in a period of 50 years. The probability of failure is than 1/50 = 0.02

The second part is the consequence. This is the effect or damage that occurs due to the event. This damage can be expressed in monetary or non-monetary values and can be divided in direct and indirect damages. For instance the structural damage of buildings or infrastructure due to the failure of a sea defence in a storm event can be expressed in direct monetary values. In addition, the inundation of an area could also causes less economic activity and therefore less export than before the flood event, causing an impact on the overall economy of the area. This is regarded as an indirect consequence. Besides the monetary damage the affected people also could suffer from social consequences such as the nuisance of their house flooded or not feeling safe or even the loss of life. These effects are regarded as the non-monetary damages and can also serve as an argument to increase the flood protection. To express non-monetary damage into a quantity proves to be a difficult task and additional readings have to be consulted.

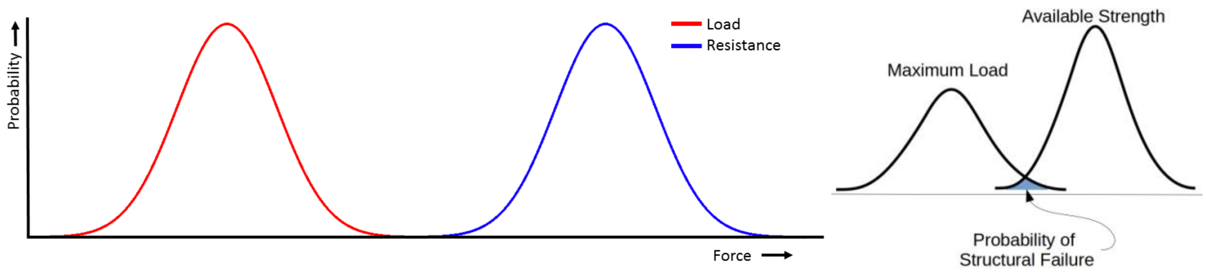


Figure 63 Load and resistance distribution and corresponding probability of failure

By multiplying these two quantities, the combined effect of change and consequences is expressed in one value. Risk therefore is expressed in a certain value per time. Below, it is expressed in a formula:

$$\text{Risk of flooding } [\text{GYD/year}] = \text{Probability of flooding } [1/\text{year}] \times \text{ Consequences } [\text{GYD}]$$

In order to find these probabilities per area and the corresponding consequences a flood risk analysis has to be performed. To perform a flood risk analysis characteristics of the system have to be known, such as the loads on and the resistance of the system, the mechanisms that could lead to a failure event and the probability of this failure event. Also the consequences have to be determined.

Table 18 Framework for flood risk analysis (Jonkman & Schweckendiek, Flood Defences, 2015)

	Step	Description
1	System assessment	To start the determining the risk of flooding of a system, the system has to be schematized. This means locating all the relevant flood defences that are present including their dependency on each other. For instance a section of rip-rap sea defence is connected to a concrete seawall. If one of these structures fails, due to their connection the both will be regarded as failed, since the area behind the connected structures is now flooded. Therefore the state of each component has to be examined.
2	Hydraulic boundary conditions	To determine the probability of failure of the system, the loads that are present on the system have to be determined. This requires the evaluation how relevant, how large and how often hydraulic loads occur. From this situation a design storm could be determined with a certain return rate. The system will be further examined based on this design storm.
3	Qualitative system analysis	The system, as schematized in step 1, is divided into sections in a rational way, such that the same characteristics hold per section. The possible failure modes per section are determined and this is visualised with a failure tree. An important factor is which failure modes can occur independent of other processes or which failure modes are dependent on other failure modes.
4	Quantitative system analysis	To determine the failure probability, the failure modes from step three will be computed. This requires a probabilistic assessment on the loads and the resistances of the sections. This means the determination of the probability when a load exceeds the resistance of a flood defence. When the loads exceed the resistance of a section of flood defences, the defence can fail.
5	Consequence estimation	Estimate the damages that happen when the system fails. These can be monetary or non-monetary and direct or indirect. The result is an indication of the damages in an area that is affected by the failing of the earlier determined sections. This could be visualized by a consequence map.
6	Flood risk	Once the consequences and the failure probability are known for each failure mode, the total flood risk of an area of interest can be determined. By combining the probability of flooding per area and the corresponding consequences per area, the risk of flooding can be also be specified per area and a distinction in risk per area can be made. It is than possible to determine if the current level of protection per area is sufficient or needs to be improved.

Once the risk of flooding is known, action can be taken in order to reduce the risk, either by reducing the probability on flood events or by reducing the consequences that occur due to flooding. The question is whether the current level of risk is acceptable, or if reduction is needed. This decision whether to lower the risk is dependent on policy.

E.3. Failure tree

In this appendix the complete failure tree of the drainage system is shown. It basically consists of three parts, namely failure of the primary channel, failure of the local drainage system and failure of flow from the road or gardens into the local drainage system. Each part is subdivided into two parts, namely insufficient outflow and insufficient storage, which are subdivided in several parts. Also, the three parts have an extra condition; this means that water enters the system from another source than an upstream channel or rain.

There are two symbols in the system: the AND-gate and the OR-gate. 'And' means that all underlying events should take place to let the upper event occur. 'Or' means that one of the underlying events should take place to let the upper event occur.

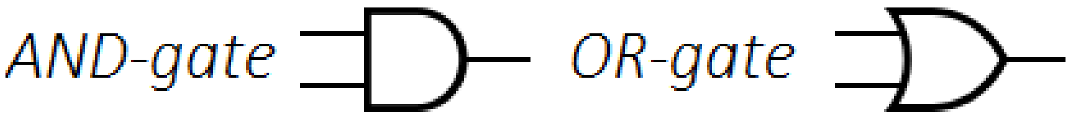


Figure 64 Symbols used in the failure tree (and- and or- gate) (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2015)

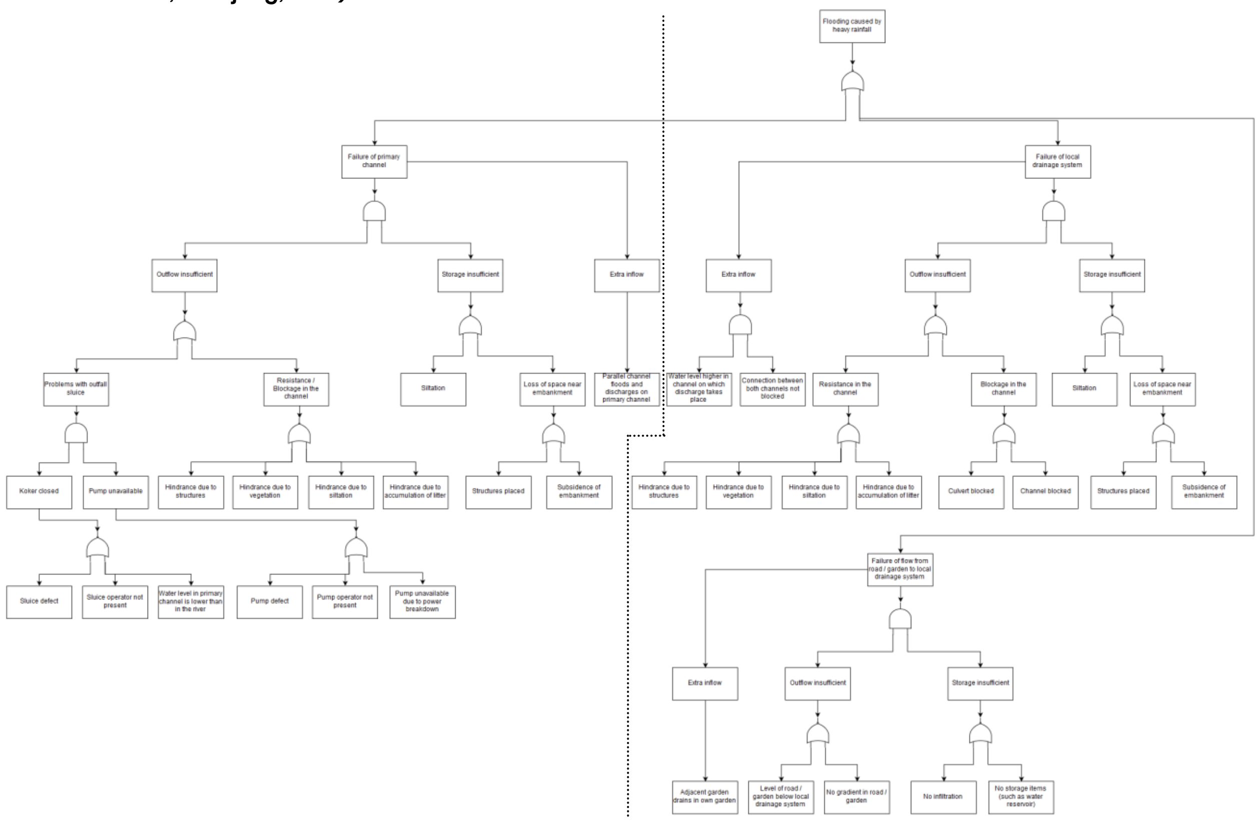


Figure 65 Total failure tree drainage system of Georgetown

Below two enlarged images of the full failure tree can be found. The division can be found in the figure of the total failure tree. They are added in this manner to assure the visibility of the elements.

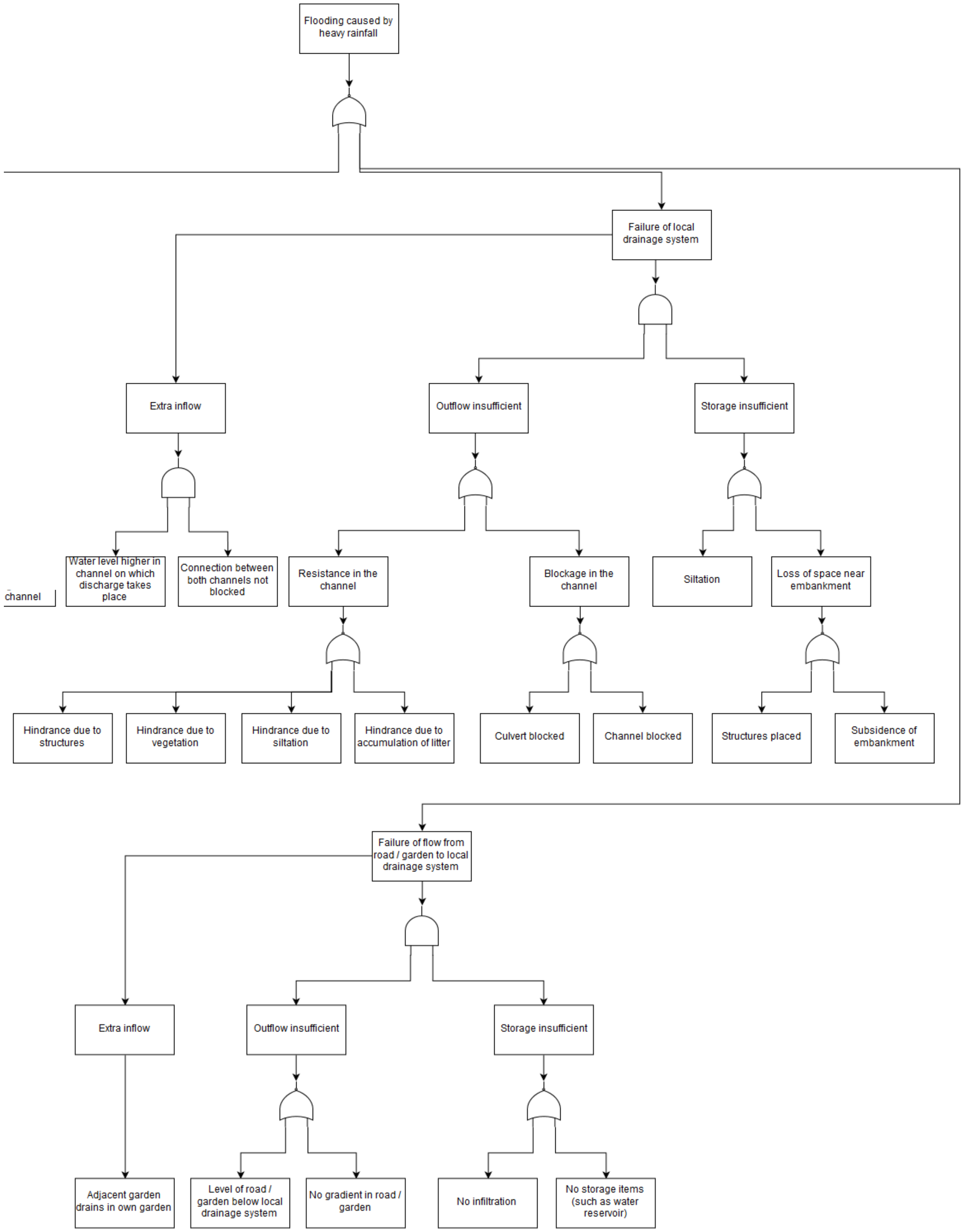


Figure 66 Part 2 of the failure tree for the drainage system of Georgetown

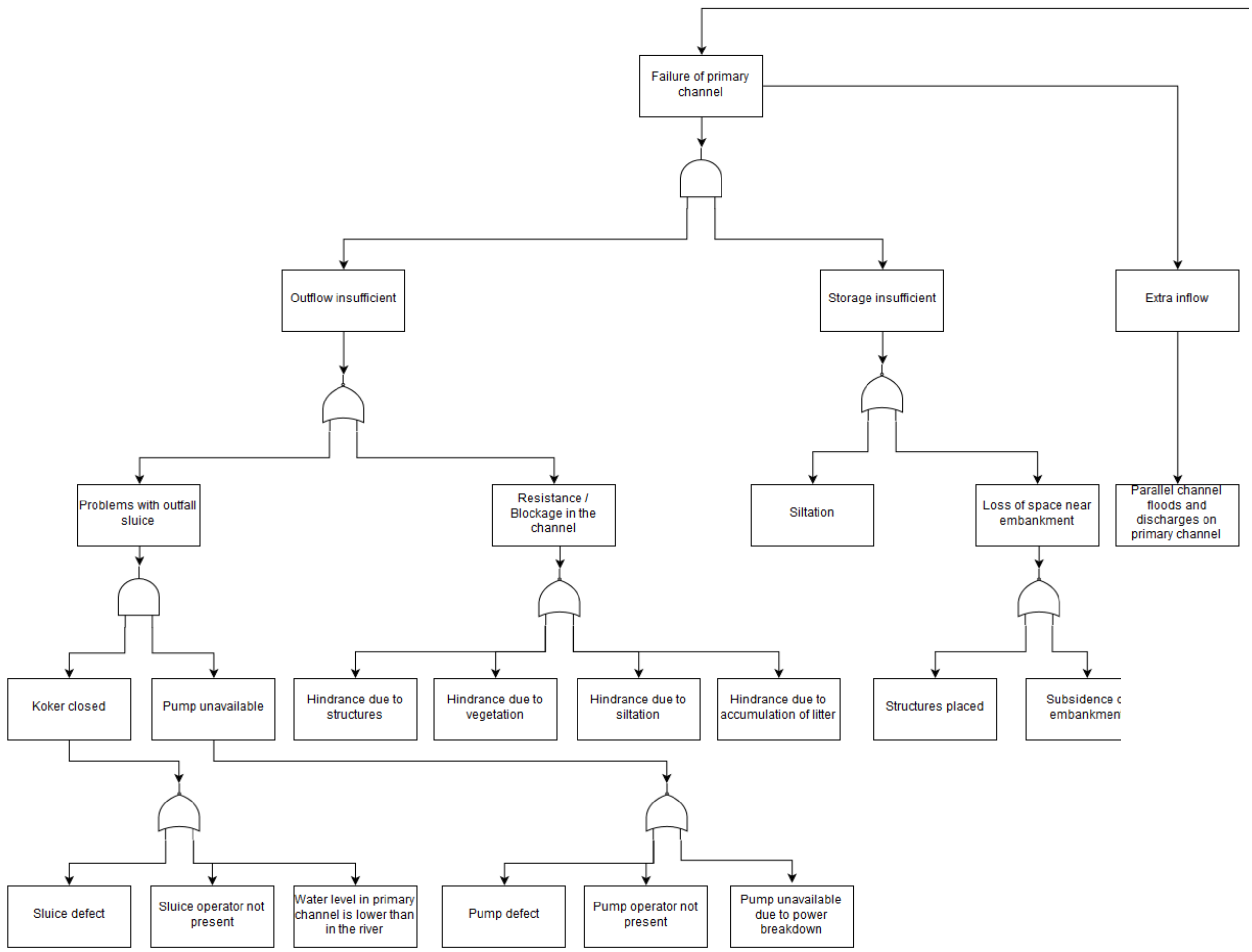


Figure 67 Part 1 of the failure tree for the drainage system of Georgetown

E.4. Economic optimization

Often determining the most effective solution is a difficult step, especially when the investment for risk reduction is high. It is therefore preferred to make use of economical optimization. This method considers the risk expressed in a monetary value and the cost of different investments.

The risk is given in a certain probability of a failure event combined with its corresponding damage. This gives a monetary value that is at stake per year. The investment for a risk reduction measure is therefore also given in a monetary value per year. This could be acquired by dividing the total cost of an investment with the lifetime of the measure, taking into account possible inflation. Additional readings on this subject are advised.

By combining these two quantities a monetary value for this measurement can be retrieved. It is then possible to compare the measures with each other based on their effective risk reduction and their costs (Jonkman & Schweckendiek, Flood Defences, 2015).

$$C_{tot} = I + (P_f \times D)$$

With:

C_{tot}	Annual total costs for a measure	[GYD/year]
I	Annual investment for the implementation of a measure	[GYD/year]
P_f	Probability of failure of a measure	[1/year]
D	Damage of structure if failing	[GYD]

Next to this, the measure itself should also be cost efficient. This is determined by comparing the investment in the measure, and the risk reduction and damage costs. The investments should be lower than the risk reduction.

$$\mathrm{I} < \left(P_{f,0} - P_{f,new}\right) * D$$

With:

I	Investment for the implementation of a measure	[GYD/year]
$P_{f,0}$	Initial probability of failure	[1/year]
$P_{f,new}$	New probability of failure after measure	[1/year]
D	Damage of structure if failing	[GYD]

For example a reduction has to be applied on a certain system. The expected damages due to a design storm are around 30 million GYD. The present probability of failure of the system is determined around 15%. There are 2 possible measures. The investment of a certain imaginary measure A is quite high (1 million GYD), but the risk reduction of this measure is also very high ($P_{f,new}$ = 0.02). On the other hand, the investment of measure B is low (0.5 million GYD), but does not have a large reduction on the current level of risk ($P_{f,new}$ = 0.05). In this case it is important that an economical assessment is made in order to compare these two measures. Sometimes, the reduction of risk and thus the consequences does not weigh up to investment that has to be made. In that case it is most economical to actually do nothing (Jonkman & Schweckendiek, Flood Defences, 2015).

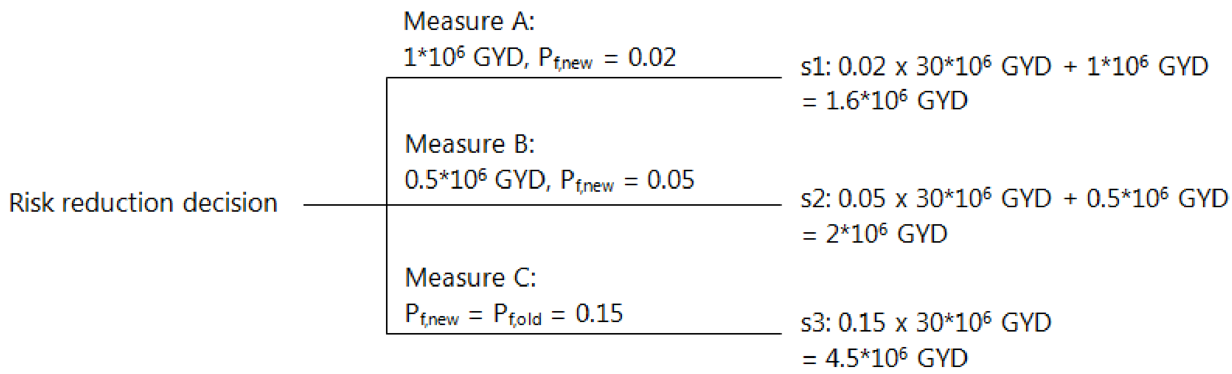


Figure 68 Example of economic optimization (three scenarios) (Jos Muller, August 2016)

Comparing the measures on their cost-effectiveness, it can be seen that for both scenario A and B the investments are lower than the risk reduction.

- Measure A: 1.0*10^6 < (0.15-0.02) * 30 *10^6 = 3.9 * 10^6
- Measure B: 0.5*10^6 < (0.15-0.05) * 30 * 10^6 = 3.0 * 10^6

After comparing the results of the assessment it is clear to see that in this situation measure A is the most economical. This measure is the most cost effective and has the lowest annual costs.

The scope of the report is solely on the investigation of the impact of risk reduction measures. In order to give insight in the consequences and the cost of the investment data on damage is needed from the region of Georgetown itself.

APPENDIX F. HYDROLOGICAL ANALYSIS

The model and calculations require a design storm with a daily and hourly rainfall intensity as it would occur in Georgetown. In this section the derivation of this design storm is explained.

F.1. Rainfall analysis

For this analysis a dataset of daily rainfall intensity from 1987 until 2016 was made available. Apart from the daily rainfall, HydroMet has started a program at 1 January 2016 until present in which the hourly rainfall is measured (HydroMet, 2016).

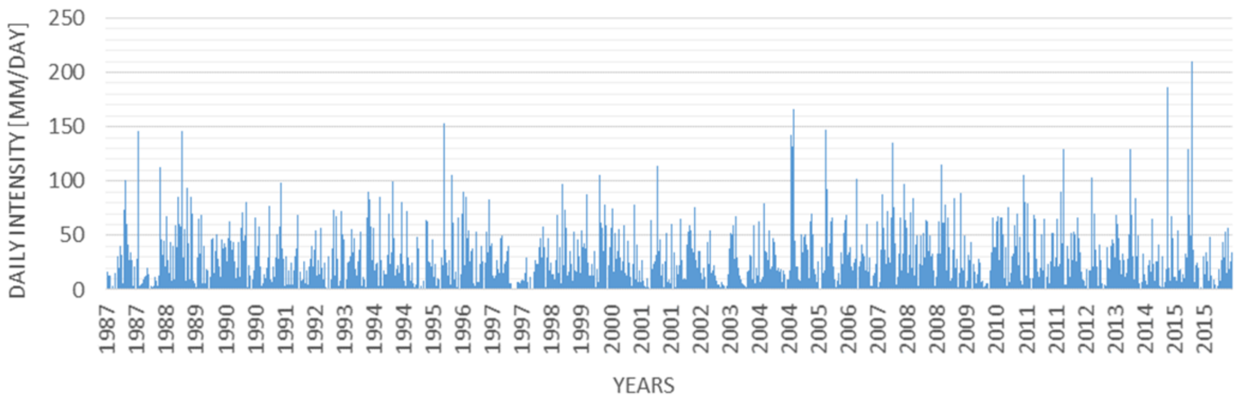


Figure 69 Extreme daily rainfall intensity profile (1987-2016) (HydroMet, 2016)

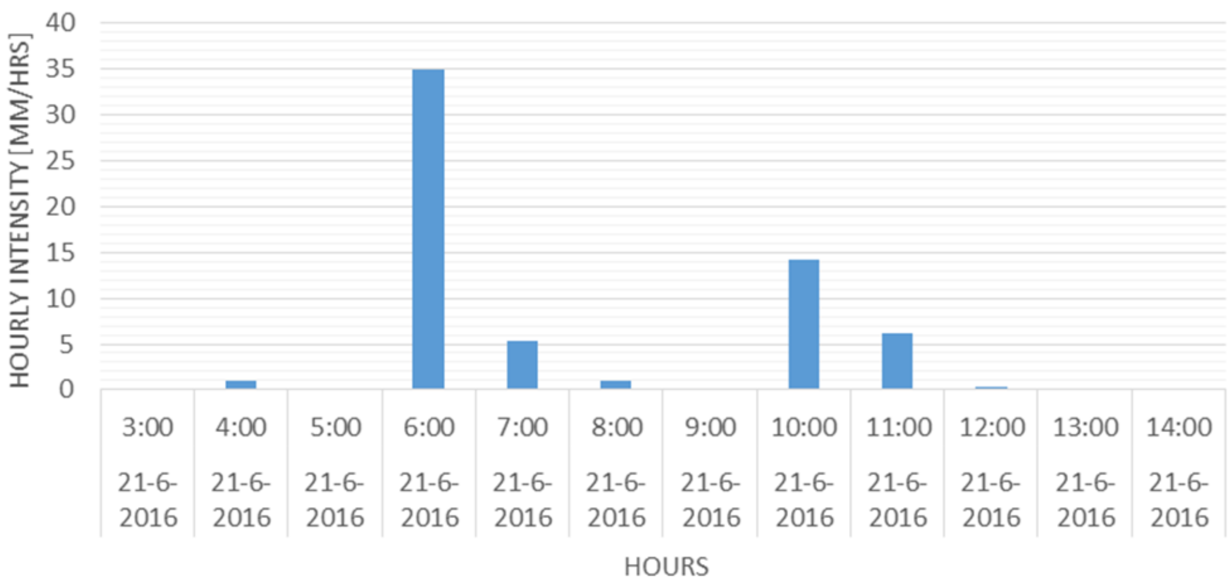


Figure 70 Hourly rainfall intensity profile example (June 21st, 2016) (HydroMet, 2016)

F.2. Frequency analysis daily rainfall

To asses this, a frequency analysis of the annual extreme rainfall is preformed which gives insight in the occurrence of storms dependent on their intensity. By collecting all the extreme daily rainfall per year and ranking them in order of their maximum intensity, the probability of occurrence was calculated with a Weibull distribution (Savenije, 2014).

$$P(X) = \frac{r}{n+1}$$

P(X)	= Probability that event X occurs
X	= annual extreme rainfall event
r	= rank of the event
n	= total number of sampled events

With the probability of occurrence for all the storm intensities known, the corresponding return period can be calculated.

$$T = \frac{1}{P(X)}$$

This is the indication of the amount of years in which the expected intensity occurs, given the current record of annual extreme rainfall. However this does not mean that this kind of storm will only occur once in the T years. For instance, the probability of the occurrence of a storm with a return period of once in the 50 years is:

$$P(X \geq x) = 1 - \left(1 - \left({}^{1}\!/_{50}\right)\right)^{10} = 0.18$$

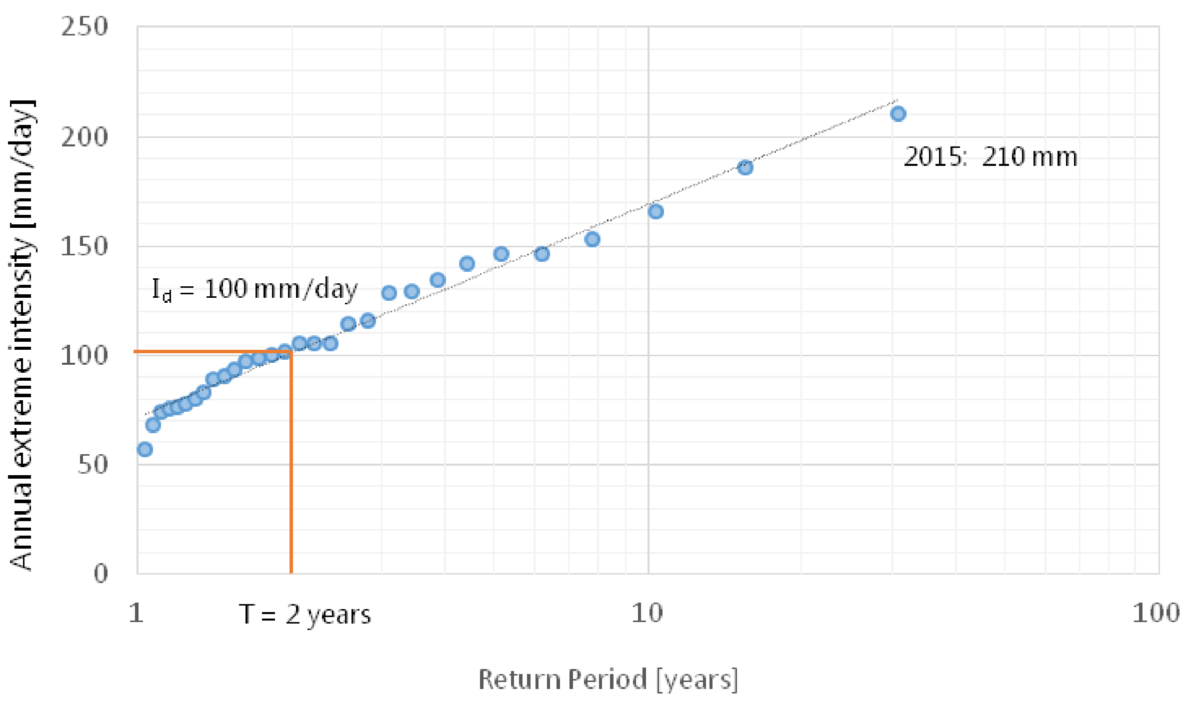


Figure 71 Return period of daily rainfall intensity (HydroMet, 2016) (Jos Muller, August 2016)

F.3. Hourly rainfall intensity

At the time of writing this document a dataset of 8 months of hourly rainfall intensities was available (HydroMet, 2016). This is sufficient to give a first insight in the average hourly rain profile. In order to get this average profile, the data was sorted to the biggest storms that occurred from January until August 2016. These storms were projected with their maximum intensity on the same moment. From this scatter an average profile was receded. The hourly rainfall intensity is expressed in the volume relative to the total volume that has fallen.

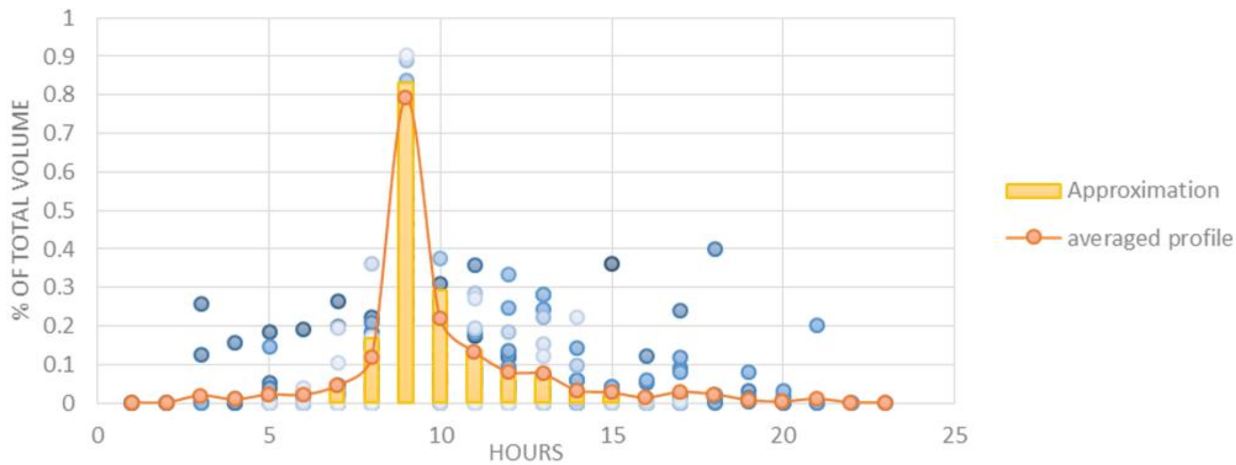


Figure 72 Hourly rainfall intensity analysis (Jos Muller, August 2016)

Since this profile is a broad average obtained from many storms, an approximation of this profile is derived in order to create a user-friendly profile which can be used for runoff and discharge computations.

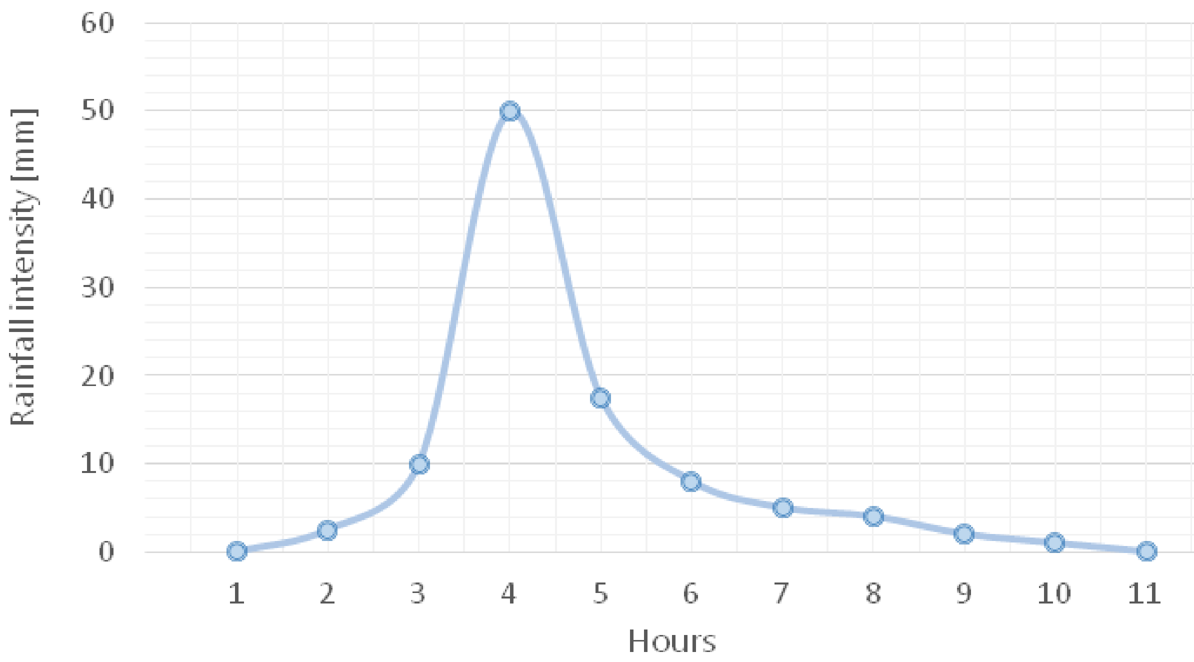


Figure 73 Averaged hourly rainfall intensity profile (Jos Muller, August 2016)

F.4. Runoff analysis

In paragraph 3.2.2 the runoff analysis was explained. This was retrieved using an excel and different runoff coefficients for different land uses. For the example area (7) the runoff determination is added in the table below.

Table 19 Example of runoff coefficient computation for small local drainage area

Characteristics	Value/text			
Nr	7			
Name	South Ruimveldt Park 3			
Surface area (m^2)	190000			
Surface of the earth	**Surface area**	**Percentage**	**Runoff coefficient***	**Relative proportion**
Roofs/Buildings	31419	17	0.95	0.1615
Asphalt/Concrete Roads	33180	17	0.9	0.153
Stone Pavement	70260.4	37	0.85	0.3145
Bare Surfaces	30111.6	16	0.2	0.032
Parks, Strips of Land	25029	13	0.1	0.013
Total (%)	100			**0.674**

F.5. Remarks

There is only one meteorological station in Georgetown and its nearby area and that is the HydroMet station in the Botanical Gardens. Because no further data is available on distribution over area and the size of the area under consideration is maximum 4 km wide it is assumed that the rainfall falls evenly distributed over the whole area. However, this is a rough assumption and when more data is available and used on the distribution of a storm over the area it would increase the accuracy of the analysis.

It is noted that in the future a more precise metrological analysis of the rainfall has to be performed. One of the subjects which have to be more examined is the duration of a storm. In this case a single average duration is chosen for the design storm. However shorter storms with higher intensities or longer storms with longer constant loading on the drainage system could also occur with their own respective return period. These different kinds of storms present a different kind of loading on the drainage system and could be of interest in the future.

Also the hourly rainfall profile is derived from 8 months of rain showers data. Therefore it is hard to detect a trend in the change of this profile over the years. However during the interviews, the team received info of a trend in which the daily rain intensity becomes lower, but has a higher intensity in the first hours of the storm, giving it a higher impact on the storage inside the areas of the city.

APPENDIX G. LOCAL DRAINAGE SYSTEM

In this appendix more elaboration will be given on the local drainage system analysis. First more background information is given on what such as system looks like (G.1). Secondly more technical background is given as an addition to the model that was developed (G.2).

G.1. Addition to area description

In chapter 3 some examples of the states of channels in the local drainage system are stated. In this appendix, some more example illustrations are given. Also, an overview of their location within the system is given.

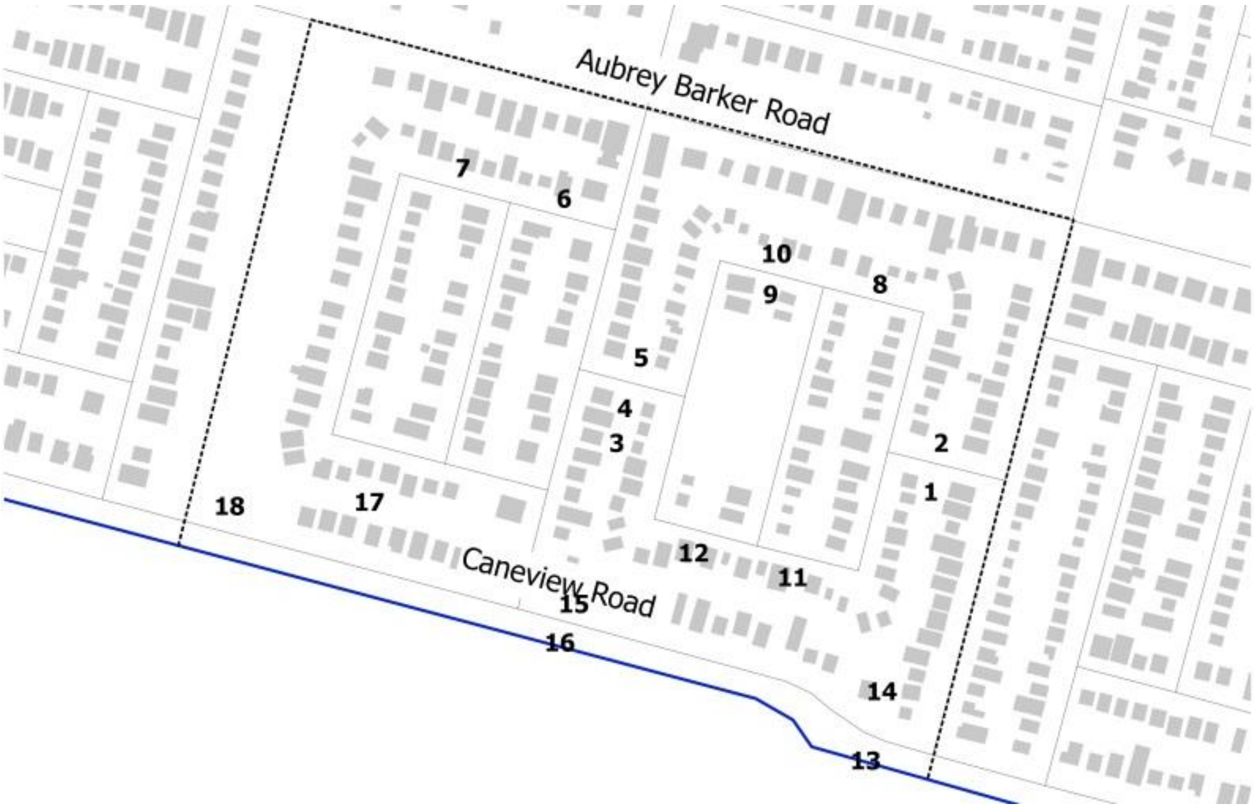


Figure 74 Overview of photographs in the area description appendix with their corresponding locations (Openstreetmap, 2016)

Figure 75 Illustrations on local drainage system (1) (Ruben van Montfort, August 2016)

1. There is only limited vegetation present. However, there is siltation on the bottom.
2. There is no vegetation present. However, the flow of water is limited by litter and siltation.
3. A secondary channel with limited vegetation present.
4. Close to the culvert, there is no vegetation or litter present. However, there is siltation.
5. In this picture, one can clearly see the difference between channels with wooden walls and without wooden walls.
6. This channel is entirely filled with vegetation. Conveyance and storage can hardly take place.

Figure 76 Illustrations on local drainage system (2) (Ruben van Montfort, August 2016)

7. On one side of the culvert, there is a large amount of vegetation present. Flow is hindered.
8. This is a channel with a lot of vegetation in it. Flow is hindered.
9. This is a channel without vegetation. However, there is a layer of sediment present.
10. Next to this culvert, there is a lot of vegetation present.
11. This is a channel without vegetation.
12. This is a channel without vegetation.

Figure 77 Illustrations on local drainage system (3) (Ruben van Montfort, August 2016)

13. The part from the culvert to the primary channel is fully grown here. Flow is hindered.
14. This is a channel with a lot of sediment at the bottom. Also, vegetation is present.
15. This is a channel where maintenance has taken place.
16. This part between the culvert and the primary channel has some vegetation in it.
17. This is a good example of the difference between maintenance and lack of maintenance. In one part of the channel, conveyance can take place. In the other part, this is limited.
18. This tertiary channel is fully filled with grown vegetation.

G.2. Theoretical background model

To analyze the local drainage system (3) an excel model was made. The model is based on basic hydraulic principles. These are explained in this part of the appendix. Also more clarification is given on the assumptions that were made to develop the model.

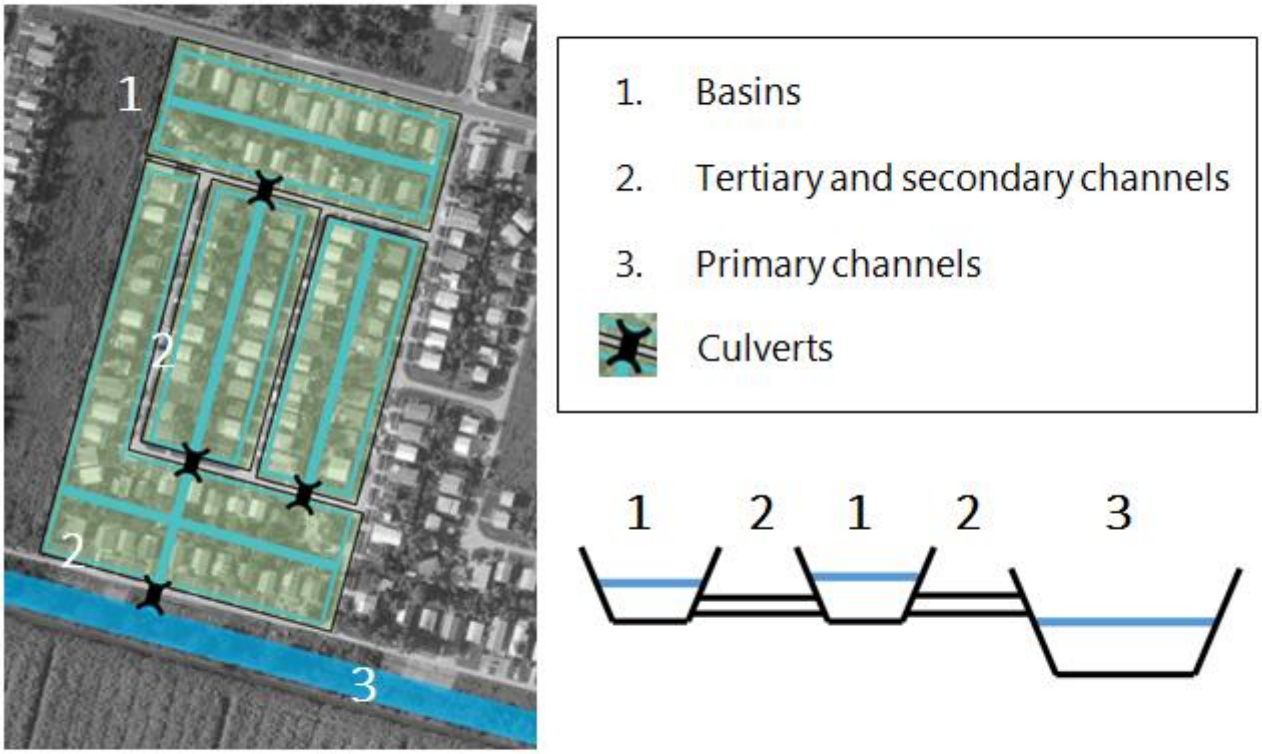


Figure 78 Modelling subdivision of local drainage system (Jos Muller, August 2016)

The underlying method of determining the water level inside the ward and the rate at which the ward runs empty is the storage principle. In this approach the change of volume of water inside a containment is determined by the inflow and the outflow during a specific time step:

The storage principle is based on the change of volume inside a containment. The water level differs in time when the outflow is not equal to the inflow (Savenije, 2014).

$$\frac{dS}{dt} = Q_{in} - Q_{out}$$

dS/dt	Change of the volume of water during a time step t	m^3/s
Q_{in}	Inflow	m^3/s
Q_{out}	Outflow	m^3/s

The inflow in the case of a ward in South-Ruimveldt is precipitation. When the rain starts to fall, it will flow on the roofs and gardens and flows from there into the local drains. This can be calculated by making use of a runoff coefficient, which takes the effect of direct runoff by roofs and intrusion by gardens into account (3.2.2). In some downstream basins, the inflow is also decided by the outflow of some upstream basins.

The outflow occurs through the culverts. It is assumed that the water bodies inside the basins have a very low flow velocity and can therefore be assumed static. When the water level inside the upstream basin is higher than the downstream basin, the culvert has a theoretical discharge. This is calculated via the energy equation, taking into account in- and outflow reduction and wall friction:

The discharge through the culvert depends on several processes. Firstly the head difference (ΔH) is the driving force of the discharge. Besides of that tow friction factors are included: one for wall friction (c_f) and one for channel roughness (c) (Battjes, 2002).

$$Q = A * \sqrt{2g * \Delta H / c}$$

Q	Outflow discharge	m^3/s
A	Outflow cross section	m^2
ΔH	Energy level difference between two basins (=water level)	m
c	Friction coefficient	(-)
ξ	In- (0.5) and outlet losses factor (1.0)	(-)
L	Length of the outflow connection	m
D	Equivalent outflow diameter	m
c_f	Wall friction factor	(-)
n	Manning's coefficient	$s/m^{1/3}$
R	Hydraulic radius	m

$$c = \Sigma\, \xi + 8 c_f \frac{L}{D} \qquad c_f = \frac{g}{\frac{1}{n} * R^{1/6}}$$

The friction factors for the culverts serve for two purposes in this model. The first (c) is the actual friction that occurs around the inlet and outlet of the culvert itself. The second, the wall friction factor (c_f), is used to describe the situation in the drains. Since the flow in reality does not only occur in the culvert but also in the local channels, the state of the local channels influences this flow. For instance if the culvert is clean, but a dense vegetation is present in front of the culvert, the flow is lagged as well.

Figure 79 Friction and flow obstruction in and around culverts (Jos Muller, August 2016)

As mentioned earlier the storage is the volume of water that is present inside the local drains around the houses. By measuring the width, depth and the length of these channels, the total effective volume of the storage can be computed. When the inflow and outflow is known, the change in volume during a specific time step can be calculated and added or subtracted from the previous storage in order to find the new storage and corresponding water level.

The model computes its output again every time step. During computation it uses parameters which have an influence on each other. Therefore the computational time step was chosen as 5 minutes to prevent large computational faults due to interdependencies.

APPENDIX H. MODEL CLARIFICATION

In this appendix more detailed information is given on the developed example model on the hydraulics of the primary channel South-Ruimveldt. First the total structure of the model will be explained (H.1) and clarification is given on the files in a HEC-RAS model (H.2). Afterwards all geometric and hydraulic input that was made is described (H.3). With this input data the model can be run. This procedure is described in (0). Finally some extra remarks are given on the software package HEC-RAS (0).

H.1. Model structure

A framework was developed for the total model structure. This can be distributed in one folder with a size of approximately 5 megabytes. The folder consists of three elements: the hydrological analysis, base model structure and model scenarios.

The first folder, 'hydrological analysis', consists of an Excel file which represents the full hydrological analysis. In this file you can retrieve discharge data for different discharge areas. It computes this in the same manner as described in the hydrological analysis chapter in this report. It is possible to compute the discharges for different return rates.

In this overview sheet you have to fill in the manual input to compute the Steady Flow and Unsteady Flow Hydraulic Data for the model:

Nr	Name	Surface Area (km^2)	Runoff Coefficient	Inflow Points
1	Banks Estate	0.18	0.74	3
2	Guyhog Gardens	0.36	0.67	4
3	Roxanne Burnham Gardens 2	0.29	0.66	2
4	Roxanne Burnham Gardens 1	0.24	0.59	1
5	South Ruimveldt Gardens 2	0.28	0.65	1
6	South Ruimveldt Gardens 1	0.34	0.68	2
7	South Ruimveldt Park 3	0.19	0.67	1
8	South Ruimveldt Park 2	0.33	0.74	1
9	South Ruimveldt Park 1	0.12	0.72	1
10	North Ruimveldt 1	0.21	0.6895	2
11	North Ruimveldt 2	0.16	0.737	2
12	North Ruimveldt 3	0.09	0.718	1
13	Festival City 1	0.24	0.6975	1
14	Festival City 2	0.22	0.6495	1
15	Festival City 3	0.11	0.688	1
16	Meadowbank	0.51	0.704	1
17	Houston Estate	0.95	0.6	1

Give the area number of the area you want the input for both Steady Flow and Unsteady Flow data:

Area Nr Data Needed	Area Name	Wanted Return Period
1	Banks Estate	1

Below you can find the steady state discharges for the area nr. you filled in above:

Discharge Total (m^3/s)	Discharge Inflow Point (m^3/s)
1.33	0.44

Below you can find the undsteady flowdischarges for the area nr. you filled in above:

Hrs	Total (m^3/s)	Point (m^3/s)
1	0	0
2	0.1	0.03
3	0.2	0.07
4	0.6	0.2
5	0.7	0.23
6	0.5	0.17
7	0.3	0.1
8	0.1	0.03
9	0.1	0.03
10	0	0
11	0	0

Figure 80 Main menu interface in Hydrological Analysis sheet (Joost Remmers, August 2016)

The second folder, 'base model structure', consists of three subfolders. In the first subfolder, 'model files', the total hydraulic model in HEC-RAS can be found. In this HEC-RAS model the final adjustments to the model can be made when it needs to be improved. In the second subfolder, 'model run', the base model can be run to view the basic output results. The last subfolder, 'calibration', a model is added in which calibration measurements can be implemented and adjustments can be made for calibration. When satisfied on these adjustments they can be applied on the base model in the first subfolder 'model files'.

The third folder, 'model scenarios', consists (currently) of eight subfolders and can be expanded. In this folder all interventions can be modelled and executed. Per intervention one subfolder can be created which contains of the same two folders: 'model files' and 'model run'. In the subfolder 'model files' geometry or unsteady flow files can be added or adjusted to create a scenario for the intervention. Also new plan files can be made to make sure the model can be executed. In the 'model run' folder the scenario can be executed and output can be viewed.

To every folder a readme file is added which contains basic information on the content of the folder and of the files in the folder.

H.2. HEC-RAS structure

Every project contains of several files which together form the total HEC-RAS model. The base model structure which can be found in the total folder contains all needed files which are needed to run the model. The following files are present:

Extension	Name	Description
.prj	Project	Collection files linking all other files to each other. Only one file per project.
.g	Geometry	Geometry file containing the full geometry. Multiple files possible.
.u	Unsteady flow	File containing boundary conditions and input. Multiple files possible.
.p	Plan	Plan files linking geometry with unsteady flow files. 1 per scenario.

Figure 81 Extensions of all files in base model structure

It is wise to assure that only the files above are present in the 'model files' folder to keep a clear view on your project. When the model needs to be executed all files can be copied to the 'model run' folder and here the model can be executed and the output can be viewed.

H.3. Input geometric data

In this chapter the total input of geometric data will be explained. The geometric data consists of the following elements:

1. River reaches and river stations

The total model contains of two rivers with multiple reaches. South-Ruimveldt is divided in reach 1 and reach 2 (below and above the junction with Houston North). Houston North is modelled as a river reach as well. The river stations along this river reach containing cross sections, bridges and lateral structures are named after the distance to the most downstream cross section which is the connection with the Demerara river.

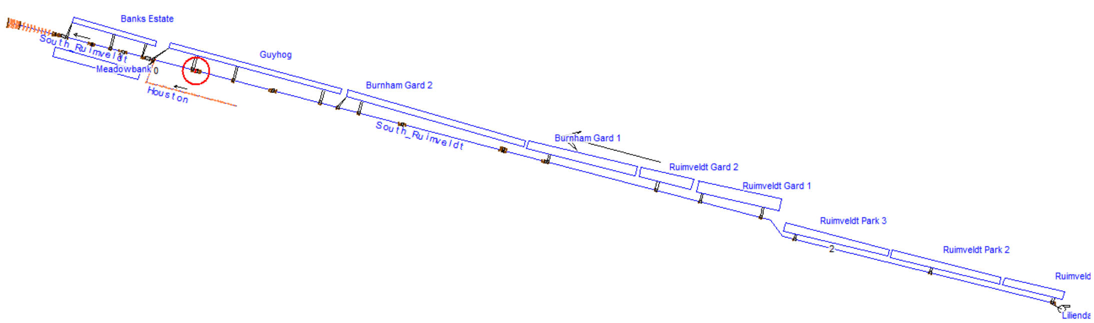


Figure 82 Total overview geometric data with river reaches with all schematized elements (Joost Remmers, August 2016)

2. Cross sections

The cross sections are implemented from the geometric measurements. They consist of downstream reach lengths (easy to calculate using the names of the river stations), Manning's roughness values and main bank channel stations. The centre of the axis system is always at the most outer measured point on the left side of the cross section. Contraction and expansion coefficients are chosen to be standard as 0.1 and 0.3.

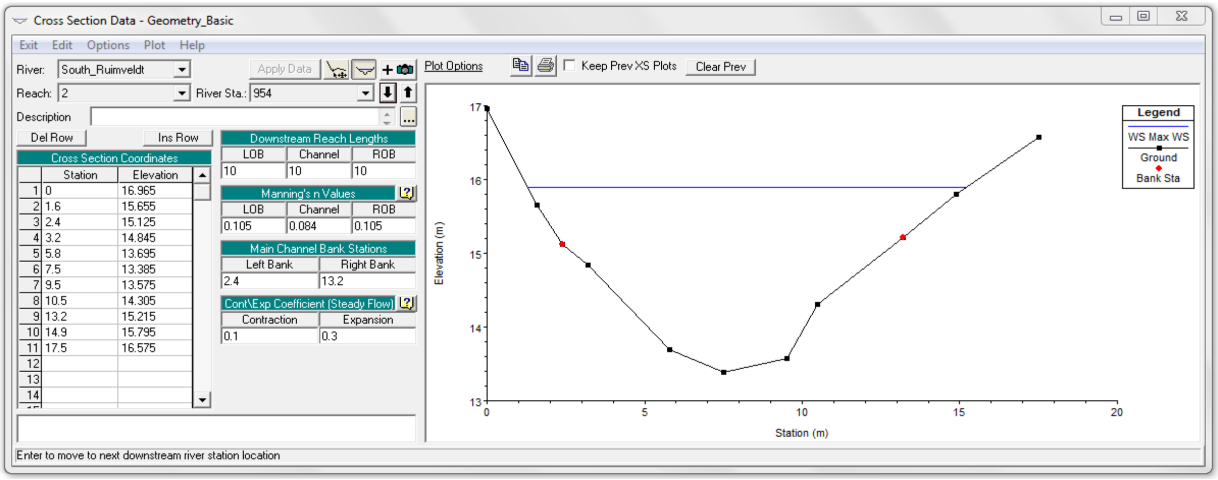


Figure 83 Cross section input geometric data window (Joost Remmers, August 2016)

3. Bridges and culverts

The dimensions of the bridges are measured along with the cross sectional data. The names of the bridges are produced in the same way as for the cross sections. The required input dimensions consist of four different elements of which the final three are optional: deck, pier, sloping abutment and culvert.

For all of the elements, the dimensions of the downstream and the upstream side of the structure should be implemented. HEC-RAS interpolates the dimensions in between these boundaries. HEC-RAS builds the bridge within the two chosen bounding cross sections. In case of a culvert, the deck should cover the complete cross section.

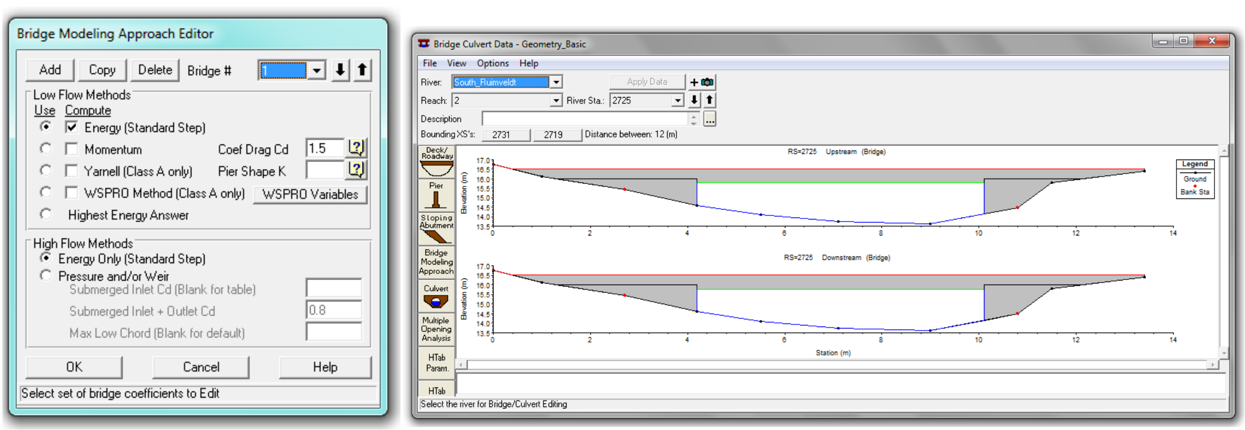


Figure 84 Bridge input geometric data window (Joost Remmers, August 2016)

Besides of the structure dimensions, four additional cross sections should be added: two downstream of the bridge and two upstream of the bridge. In the example model they are placed on both sides at 2 and 12 meters from the end of the bridge.

In the modelling approach, only the standard step is used: the energy method. The other methods give similar results.

4. Storage areas and lateral structures

These elements are the most uncommon and are used to model the effect of the local drainage system on the primary channel. The storage areas themselves are the estimated surface area of the local drainage system and their elevation is estimated to be half a meter below the level of the most outer point of the corresponding cross section in the river reach. The discharge computed in the local drainage system analysis flows into these modelled storage areas and via a lateral structure into the primary channel. This goes via a lateral structure for which the option linear routing is used. In these option the coefficients the elevation of the spillway crest is at the same level as the storage area. Hindrance of flow from the storage area to the primary channel is modelled to be as low as possible (with a coefficient of 0.99) and hindrance of flow from primary channel to storage area is modelled to be as high as possible (with a coefficient of 0.10). In this way the model accounts for the blockage of inflow at high primary channel water levels.

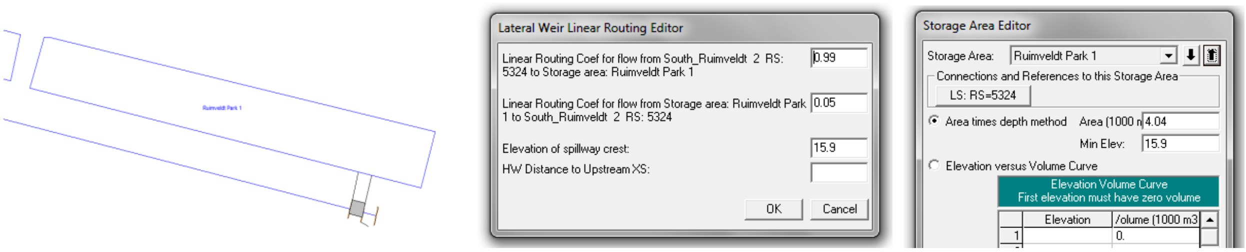


Figure 85 Storage area and lateral structure input geometric data window (Joost Remmers, August 2016)

5. Inline structures (outfall koker)

An inline structure is built in one cross section (just upstream of the gate itself). The geometry of an inline structure consists of two elements. The first element is a weir which should cover the whole cross section. The second element is a gate which should include data on its width, height, sill level and sideways placement in the cross section.

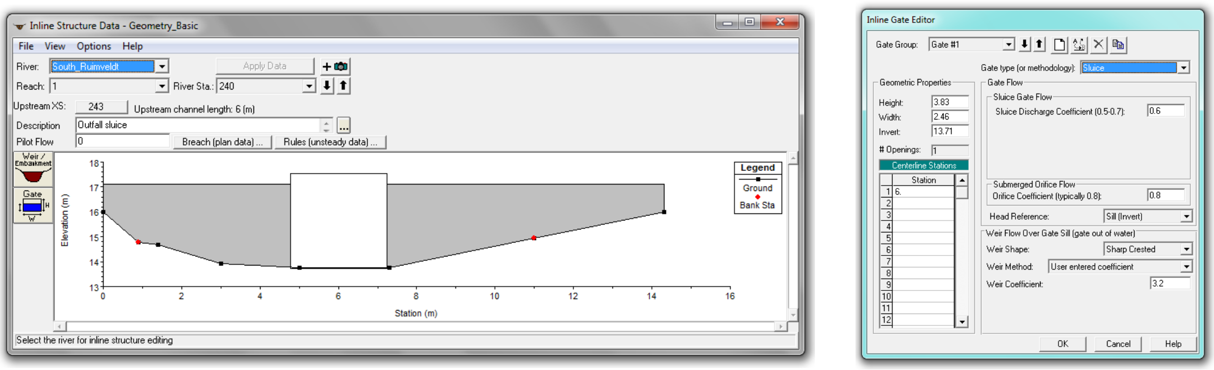


Figure 86 Inline structure (gate) input geometric data window (Joost Remmers, August 2016)

Different types of gates could be chosen. In this case the lifting gate is chosen (koker in HEC-RAS) with a standard discharge coefficient of 0.6. The opening regime of the gate is described in the steady/unsteady flow data editors.

6. Pumping stations

Pump stations are added to certain cross sections. It will contract water from this cross section to another element (storage area or cross section) or out of the system. In the case of the Liliendaal pump station, the most upstream cross section of the model is chosen. HEC-RAS uses the water level in this cross section to determine whether the pump should start up or shut off. There is a difference of 0.5 meter between these water levels, to make sure the pump is not constantly turning on and off again every couple of minutes. The water level at which the pump starts pumping is determined by a simplified model of Liliendaal, to investigate the influence of the backwater curve at the very upstream end of the channel.

The imaginary pump has a capacity of 1 m^3/s. This is the remaining capacity of the Liliendaal pumping station when it also has to drain the Liliendaal catchment area with a rainfall of 100 mm/day. With an estimated area of 10000000 m^2 and a direct runoff coefficient of 0.6 the averaged required capacity of the Liliendaal pumping station to drain the Liliendaal catchment area is:

$$\frac{0.1 * 10000000 * 0.6}{24 * 3600} = 7\ m^3/s$$

With an estimated total capacity of the two pumps of 8 m^3/s, the remaining capacity for the South-Ruimveldt catchment area is 1 m^3/s.

H.4. Input hydraulic data

In this chapter the total input of hydraulic data in HEC-RAS will be described. All the boundary and initial conditions will be defined. These are needed as input before the model can start computations.

1. Boundary conditions

The rainfall is added as a boundary condition to the storage areas. For every storage area the corresponding runoff has been analysed and the delay in runoff has been taken into account in the analysis of the local drainage system. For different return periods the rainfall has been analysed and each profile consists of a shower with a duration of nine hours. The rainfall is added to the system after twelve hours, which roughly corresponds to the closing time of the South-Ruimveldt koker. This is done to ensure that a worst case scenario is considered, i.e. the rainfall starts when the koker has just closed.

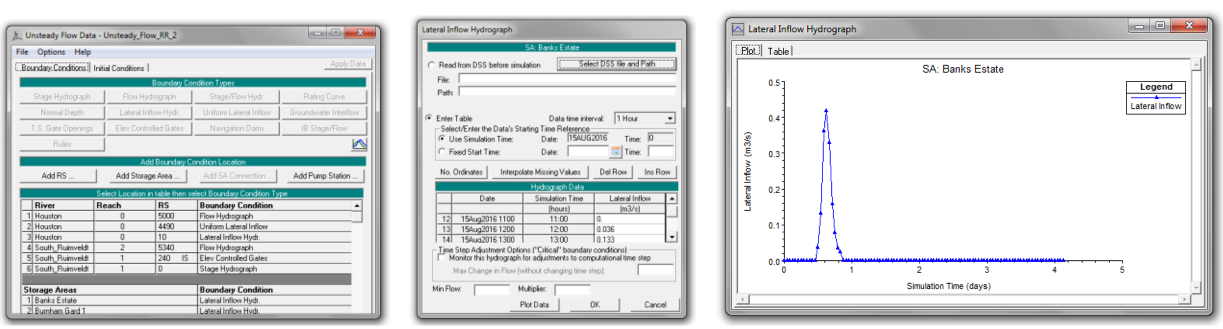


Figure 87 Input of hydraulic inflow into the system (Joost Remmers, August 2016)

At the koker a boundary condition is applied which is called ‘elevation controlled gates’. The gate is elevation controlled and opens at a water level difference between river side and channel side of 0.001 meter. It closes again at a difference of 0 meter. The gate is initially opened at a height of 1.5 meter. The gate opening will be 1.5 meter in the next opening cycle as well.

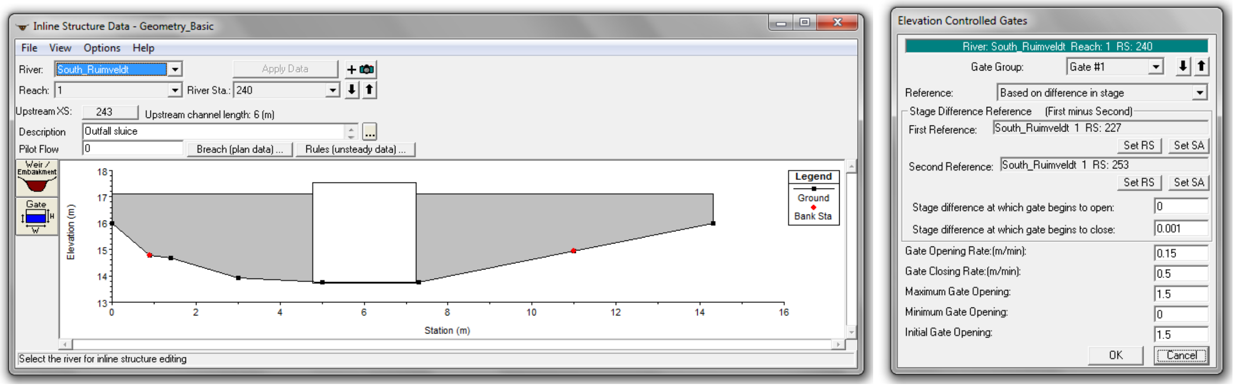


Figure 88 Boundary condition of an elevation controlled gate in HEC-RAS (Joost Remmers, August 2016)

At the final end of the channel (behind the koker) the boundary condition ‘stage hydrograph’ is applied to account for the tidal elevations downstream. Here the tide can be entered over time and by this way a water level is fixed at the end of the channel. Together with the water level inside of the channel it determines whether the koker opens or not.

2. Initial conditions

As initial conditions water levels have been used. For all storage areas, it has been accomplished that no water is initially present. Therefore the water level is equal to the minimal elevation of the storage area. The water levels in the river reaches of Houston and South-Ruimveldt are both equal to the initial tidal elevation. For the Houston and South-Ruimveldt river reaches a zero flow condition has been applied initially.

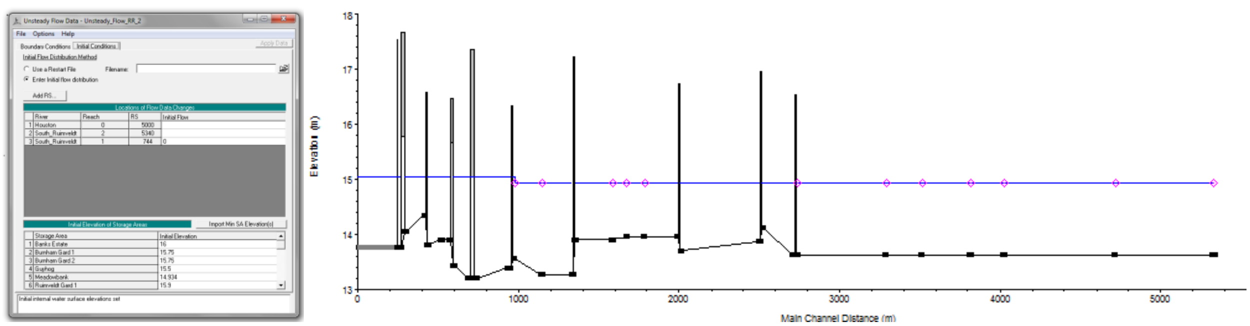


Figure 89 Initial conditions in HEC-RAS (Joost Remmers, August 2016)

H.5. Output

Calculations can be made by clicking the steady or unsteady flow calculation button. For each computational scenario a plan needs to be made which consists of a combination of a geometry and flow file. Also computational steps and detail of the output needs to be specified. If it is desired to make multiple computations in a single click and compare the outcomes, a plan of each computation has to be made by clicking the unsteady flow calculation button and saving the plan. From the main menu, in the run drop down, multiple plans, the plans can be selected and will be calculated in a single run. This was done to compare all the selected measures with the original (current) situation.

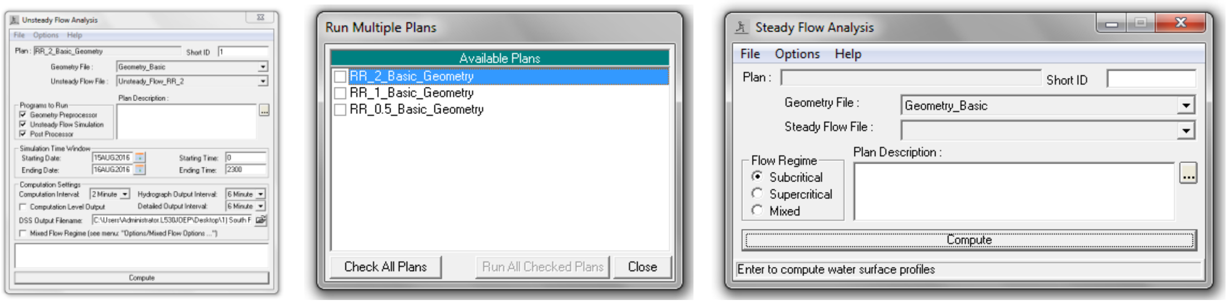


Figure 90 Running (multiple) plans in unsteady or steady flow in HEC-RAS (Joost Remmers, August 2016)

Output can either be viewed in a longitudinal cross section as a function of time, or in a stage or flow hydrograph at a single point. Both have been used to view the output of different measures.

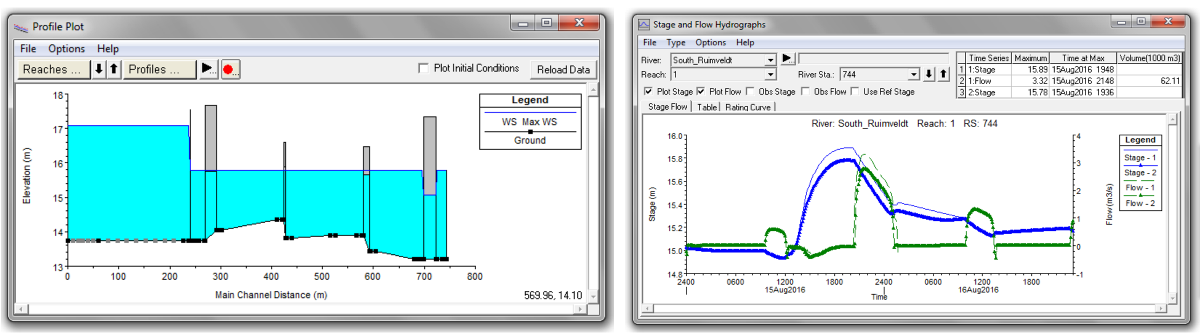


Figure 91 Viewing output in HEC-RAS (Joost Remmers, August 2016)

H.6. Remarks on HEC-RAS

HEC-RAS solves the physical laws of conservation of mass (continuity) and momentum, which are mathematically expressed in a set of partial differential equations. Energy losses from friction and structures are included in Manning's roughness coefficient and loss coefficients. An implicit numerical calculation scheme is used to solve the system of equations, and therefore account has to be taken of numerical issues discussed below.

In different conditions the calculation scheme might be stable or unstable depending on the calculation parameters. However, the implemented structures and geometric data might also result in drastic flow changes and consequently in instability.

Therefore it is always necessary to check whether the calculation produces largely varying outcomes for:

- Different time steps (entered in the unsteady flow computation window)
- Different distance intervals in between cross sections

Any unexplainable outcomes or fast changing flow conditions should be checked by decreasing the time step or reducing the distance between cross sections. If no significant changes can be observed, the outcomes can be considered stable. HEC-RAS will stop its run when the model becomes unstable and will give warnings when signs of instability seem apparent.

H.7. Improving the model

In the example model there are still some uncertain aspects. To improve the model and make it ready for decision making based on quantitative results, the following steps should be taken:

1. Expand geometric data of the most upstream part of the South-Ruimveldt channel

Only basic measurements are done. With proper measurements, the accuracy of the model can be improved. Use UG measurement techniques and the possibility of student theses for improving and expanding the knowledge on geometric data.

2. Implement Liliendaal catchment area

A basic assumption is made on the influence of the Liliendaal pump station on the South-Ruimveldt channel. This can be made more reliable by implementing the Liliendaal catchment area. However, a note has to be stated that this might be very time-consuming, because of the size of the catchment area.

3. Perform calibration steps

More calibration steps should be performed in order to make a more reliable model. Especially during high discharge.

After the above steps are executed, a comparison can be made between the initial model and the expanded model. In this way a sensibility analysis of the basic assumptions done in the initial model can be performed. This knowledge can be used when modelling other catchment areas (which might also be connected with Liliendaal).

H.8. Expanding the model

Besides of the South-Ruimveldt area, HEC-RAS can be applied to other areas as well. The same method might be used for other catchment areas within Georgetown. Most of them are urban and comparable to South-Ruimveldt. HEC-RAS can be used for rural areas outside of Georgetown like sugarcane fields as well. Modelling of the main channel can be done in a similar manner as for the South-Ruimveldt model. But the delay and storage in the local drainage systems should be approached in a different way, because of the different type of land use.

Before modelling other areas, some important subsequent steps have to be taken. The model should be made from 'global to detail'. The steps 2-6 given below can be followed a few times after each other. Each cycle the model can be updated and can be made more detailed, until satisfying results are acquired.

1. Improve modelling vision

This means that the designated engineer should be able to acquire certain goals with the use of a model in an efficient way. It is not always useful to make a perfect representation of the reality in the model, because this is very time-consuming and in some situations even impossible. Besides of that, it does not always improve the results. The user should thus be able to make assumptions in the model and evaluate these assumptions on their reliability. Besides of that, it is important to start with a global model and make it more detailed within a couple of cycles where the complete procedure is followed.

2. Set the goal of the model

The making of a model is not a goal on itself. It is always used for something bigger. The goal can have two faces. Firstly, it can be used to understand the system and finding the weak spots. It can also be used to research possible measures in the system. It is important to know which kind of measures are considered and which data is needed to make a good assumption of these measures. Besides, a good insight in the considered area is vital: where is it closed off, which connections do exist, how is the koker operated, etcetera?

3. Acquiring data

With the above steps taken in mind, the required geometrical and hydraulic data can be acquired.

4. Building the model

After the data is acquired, the model can be build. The first set up should be simple (steady flow) to get an idea of the system. After that a more complex model can be build (unsteady flow). Eventually the model should be calibrated. More information about building a model can be found in the application manual.

5. Performing analyses

With the model running, an analysis can be performed. Measures can be implemented and the results can be produced.

6. Draw conclusions

A part of the 'modelling thinking' is drawing the right conclusion out of the results of the model. The exact values given by the model should not be regarded as a fact without questioning it. The user should be able to criticize its own results. Because exact results might not be reliable, it is more convenient to compare different results with each other, for example the situation of doing nothing with the implementation of a measure. In this way the effectiveness of measures can be judged relative to each other.

APPENDIX I. OUTFALL STRUCTURES

I.1. Example factsheet

To gather data in a structured manner factsheets were developed of all the kokers. Below an example of such a factsheet is given for the imaginary 'Mainstreet koker'. It consists of general information, maintenance history, dimensions and sketches or photographs.

MAINSTREET KOKER

1.1 General

Construction year	1931	**Pump present**	No
Number of doors	1	**Garbage filter**	No
Material	Concrete	**Raising system**	A

Possible to maintain	The koker is easily accessible from both sides with all types of vehicles.

Operator name	Steve Owinson
Operator phone number	06 123 45 678
Operator since	21-06-1994

Size catchment area	102.8	ha	**Width of sluice opening**	2.80	m
Invert level of Sluice	13.29	mgD	**Cross section below 15.54 mGD**	6.30	m^2
Sluice depth below 15.54 mGD	2.25	m	**Catchment/cross area**	16.32	$\frac{ha}{m^2}$

1.2 Maintenance

DATE	*MAINTENANCE*
08-2014	Column 1 and 2 have been repaired with a new layer of concrete (20-50 mm thick)
09-2011	The body of the raising system has been replaced
06-2008	The door has been replaced completely
BEFORE	No history found

1.3 Dimensions

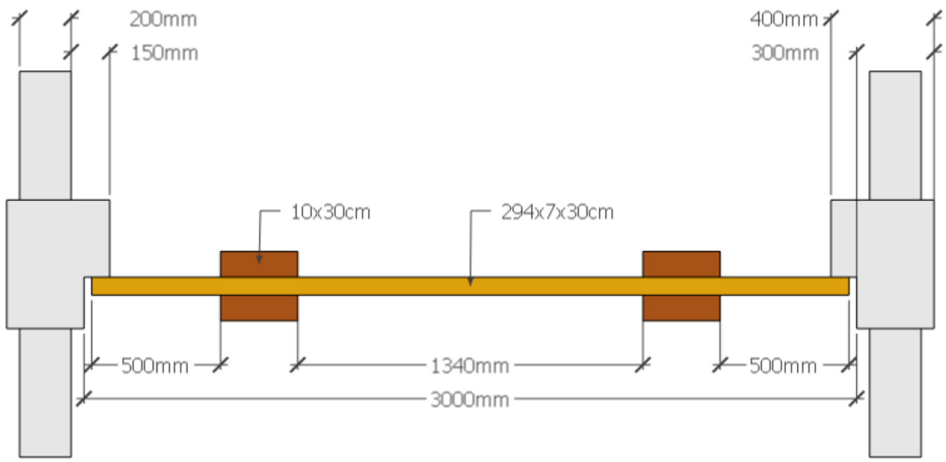
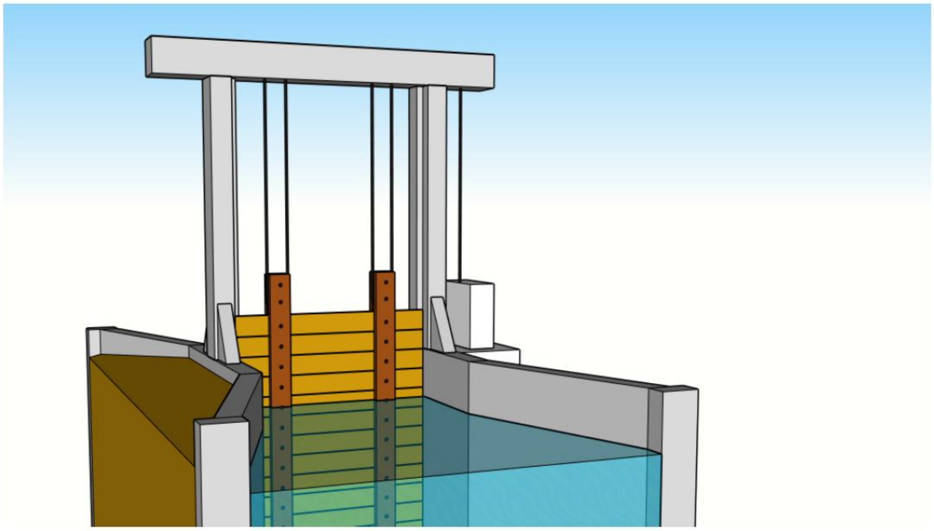


Figure 92 Example factsheet (part 1) (Thijmen Jaspers Focks, August 2016)

1.4 Photo's



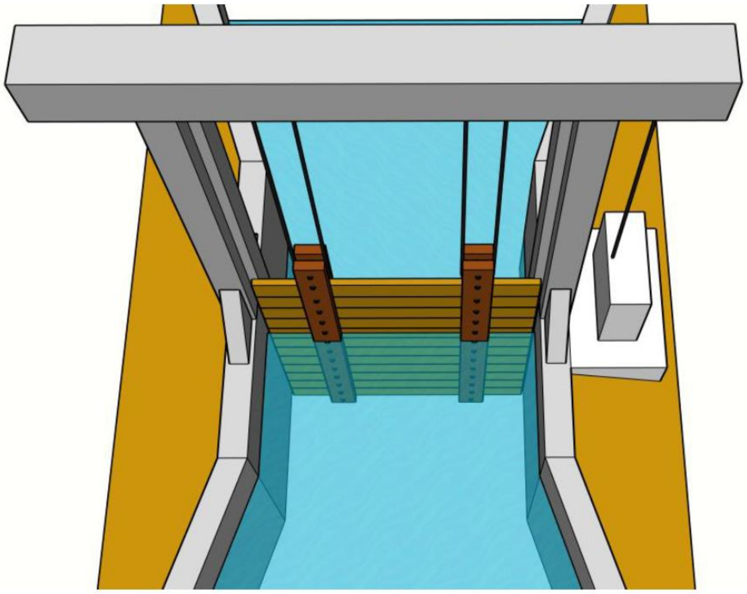


Figure 93 Example factsheet (part 2) (Thijmen Jaspers Focks, August 2016)

I.2. Inspection form

Part of the structural asset management tool is the made inspection form which can be used to assess a koker in the field. An example of a processed inspection form for the imaginary 'Mainstreet koker' is given below.

KOKER INSPECTION

Name inspector George Manfield

Date: 30/08/2016 12:30

Location of Structure Mainstreet

Type: Outfall structure

Raising system Type A

Emergency doors Present on location

1 Superstructure

Present	Part	Mat.	1	2	3	4	5	6	7	8	9	10	11	12
Yes	Column	C	8	8										
Yes	Buttress	C	8	9	8	7								
Yes	Beam	C	7											
Yes	Pulley	S	8	7	8									
No	Pump													
No	Roof													

Comment:

2 Door

Present	Part	Mat.	1	2	3	4	5	6	7	8	9	10	11	12
Yes	Vertical carrier beam	W	6	7										
No	Vertical beam													
Yes	Horizontal beam	W	7	8	8	8	8	8	9	8	8	8		
No	Horizontal beam (extra)													
No	Fastening pieces													
Yes	Bolts	S	8	8										

Connecting door-columns: Enough grease was applied

Comment: Only 2 bolts were inspected, next time inspect more

Figure 94 Inspection form example (page 1) (Thijmen Jaspers Focks, August 2016)

3 Raising system

System A	System B	System C	Part	Mat.	1	2	3	4	5	6	7	8	9	10	11	12
	■	■	Body	S	8											
	■		Gear	S	6	7	7	6								
	■		Axle	S	7	7										
			Cable / Chain	S	8											
			Counterweight													
	■	■	Braking system	S	7											
	■		Cable coil	S	7											
	■		Removable crank	S	9											
■		■	Steelcable													
■		■	Hoist													
■		■	Hook													
			Attachment component	S	8											

Comment: The entire system is starting to deteriorate, a lot of rust and lost material

4 Foundation

Present	Part	Mat.	1	2	3	4	5	6	7	8	9	10	11	12
	Inlet Conditions	C	7	7										
	Outlet conditions	C	8	7										
	U/S wing walls	C	7	7										
	D/S wing walls	C	8	8										

Signs of Seapage: Nowhere, seems to be fine

Comment:

5 Operator

Name operator	Phone numbre	Times opening/day
Steve Owinson	06 123 45 678	2

Comment operator: The koker works fine, but the raisingsystem starts to fail.
Also the beam of the door (Carrier beam) lost material

Figure 95 Inspection form example (page 2) (Thijmen Jaspers Focks, August 2016)

I.3. Assessment of failure mechanisms

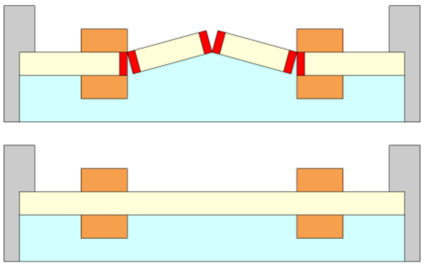
Below several of the doors failure mechanisms are discussed with their corresponding loads (Ed) and resistance (Rd). The full description of these computations can be found in the 'designer manual' which was made as a knowledge transfer document for this project (Somerville, 2008).

The first failure modes that are assessed are the failure modes of the door of the koker. These consist of the failure of a single beam within the door (A1_1) and failure of the whole door (A1_2). Both failure mechanisms are based on bending.

A1_1 Door – Single beam on bending

$$M_{Ed} = 3 * \left(\frac{1}{4}\rho_{water} * g * h_{\max depth} * h_{hbeam} * l_{gap}^2\right)$$

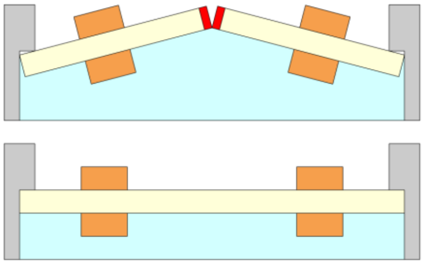
$$M_{Rd} = 3 * \left(\frac{1}{6}w_{hbeam}^2 * f_{t,w}\right)$$



A1_2 Door – Whole door

$$\mathrm{M_{Ed}} = \frac{1}{16} * \rho_{\mathrm{water}} * \mathrm{g} * (\mathrm{n_{horizontal\ beams}} * \mathrm{h_{horizontal\ beam}})^2 * \mathrm{l_{gap}^2}$$

$$\mathrm{M_{Rd}} = \frac{1}{6}\mathrm{w_{hbeam}^2} * \mathrm{f_{t,w}} * \mathrm{n_{horizontal\ beams}}$$

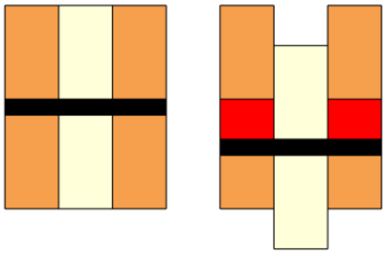


The carrier beams of the door also need assessment. From these elements the side beams can fail (A2_1) and the centre beams can fail (A2_2). Also the bolt joints can fail on bending (A2_3) and shear (A2_4).

A2_1 Carrier beam – Side beam failure

$$F_{Ed} = \frac{m_{hbeams} + m_{hbeams,extra} + m_{vbeams,extra} + m_{vbeams}}{n_{vbeams} * n_{bolts}}$$

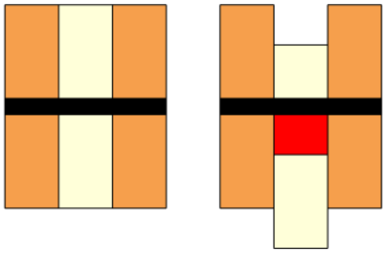
$$F_{Rd} = f_{h,1,k} t_1 d$$



A2_2 Carrier beam – Centre beam failure

$$F_{Ed} = \frac{m_{hbeams} + m_{hbeams,extra} + m_{vbeams,extra} + m_{vbeams}}{n_{vbeams} * n_{bolts}}$$

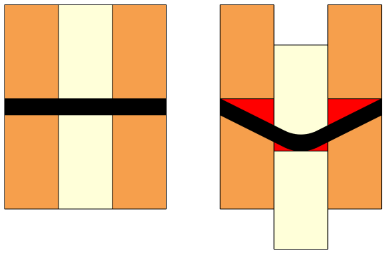
$$F_{Rd} = f_{h,1,k} t_1 d$$



A2_3 Carrier beam – Bolt failure on bending

$$F_{Ed} = \frac{m_{hbeams}+m_{hbeams,extra}+m_{vbeams,extra}+m_{vbeams}}{n_{vbeams}*n_{bolts}}$$

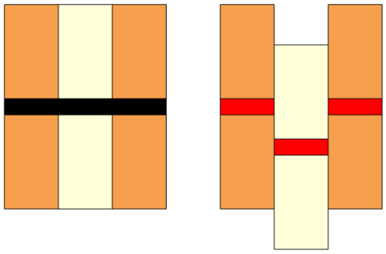
$$F_{Rd} = 1{,}05\frac{f_{h,1,k}t_1 d}{2+\beta}\left(\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}dt_1^2}}-\beta\right), \beta = \frac{f_{h,1,k}}{f_{h,2,k}}$$



A2_4 Carrier beam – Bolt failure on shear

$$F_{Ed} = \frac{m_{hbeams}+m_{hbeams,extra}+m_{vbeams,extra}+m_{vbeams}}{n_{vbeams}*n_{bolts}}$$

$$F_{Rd} = 1{,}05\frac{f_{h,1,k}t_1 d}{2+\beta}\left(\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}dt_1^2}}-\beta\right), \beta = \frac{f_{h,1,k}}{f_{h,2,k}}$$



Besides the door the superstructure can fail as well. The first element discussed is the beam which can fail due to a bending moment (B1_1), shear force (B1_2) and torsion (B1_3).

B1_1 Superstructure beam – Bending moment/flexure

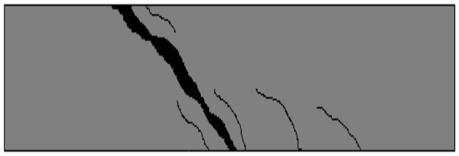
$$M_{Ed} = \sum_{1}^{\frac{n_{pulleys}}{2}} a_{p,n} * \frac{G_{door}}{n_{pulleys}}$$

$$M_{Rd} = F_s * z, \quad F_s = f_{yk} * \frac{1}{4} * \pi * \emptyset^2$$

B1_2 Superstructure beam – Shear force

$$V_{Ed} = \sum_{1}^{\frac{n_{pulleys}}{2}} \frac{G_{door}}{n_{pulleys}}$$

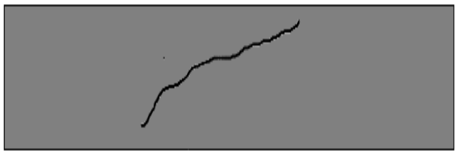
$$V_{Rd} = \frac{f_{ck} * b * z * v_1}{1.5 * (\tan(\theta) + \cot(\theta))}$$



B1_3 Superstructure beam – Torsion

$$T_{Ed} = \sum_{1}^{\frac{n_{pulleys}}{2}} e_{p,n} * \frac{G_{door}}{n_{pulleys}}$$

$$T_{Rd} = \frac{1.33*v_1*f_{ck}*t*A_k}{\tan(\theta)+\cot(\theta)}, \quad t = \frac{A_{beam}}{2*(b_{beam}+h_{beam})}, \quad A_k = (b_{beam}-t)*(h_{beam}-t)$$

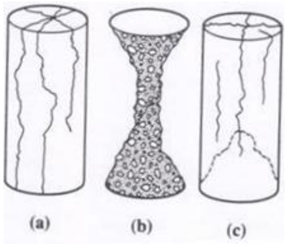


Besides the beam of the superstructure the column can fail as well. This can happen due to compression on the top (B2_1) and bottom (B2_2) but also due to buckling (B2_3).

B2_1 Collumn – Compression on top

$$V_{Ed} = \frac{1}{2} * (G_{door} + G_{beam})$$

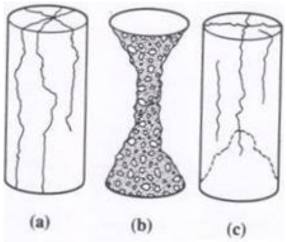
$$V_{Rd} = f_{ck} * A_{column}$$



B2_2 Collumn – Compression on bottom

$$V_{Ed} = G_{column} + \frac{1}{2} * (G_{door} + G_{beam})$$

$$V_{Rd} = f_{ck} * A_{column}$$



B2_3 Collumn – Buckling

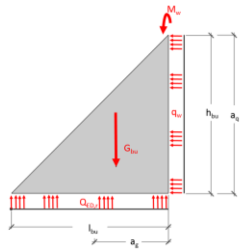
$$N_{Rd} = \frac{\pi^2 * E_{cm} * I_y}{h_{column}^2}$$

Last but not least the buttress of the structure can fail. Failure differs for structures with reinforcement (B3_1) and without reinforcement (B3_2).

B3_1 Buttress – Without reinforcement

$$Q_{Ed,r} = \frac{M_w + q_w * h_{bu} * a_q + \frac{1}{3} * G_{bu} * l_{bu}}{l_{bu} * a_g}$$

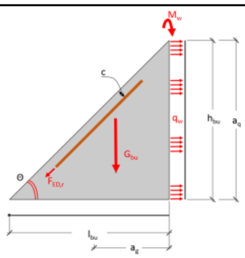
$$Q_{Rd,r} = f_{ck} * w_{bu}$$



B3_2 Buttress – With reinforcement

$$F_{Ed,r} = \frac{M_w - q_w * h_{bu} * a_q + \frac{1}{3} * G_{bu} * l_{bu}}{(l_{bu} - c) * \cos{(\theta_{ba})}}$$

$$F_{Rd,r} = A_s * f_{y,k}$$



APPENDIX J. MULTIDISCIPLINARY PROJECT

Earlier in this report (1.3) it was stated that this project is part of the masters curriculum of five of the seven group members. Two other group members (Siebe Dorrepaal and Martijn van Wijngaarden) do not participate in the corresponding course and therefore they had different responsibilities assigned. Because grading of the project needs to be possible for the supervisors, all group members had to register their activities during the project. Below you can find the responsible persons per analysis. Preparation work, report writing and all (oral) presentations were executed by the five students who did this project as a part of their master studies.

Table 20 Task division of project chapters/components main report

Chapter	Subcomponent	Responsible team member(s)
Introduction	Cover, preface, abstract, introduction	Joost Remmers
System analysis	Location	Joost Remmers
	Functioning	Jos Muller, Ruben van Montfort
	Historical developments	Ruben van Montfort
	Knowledge-based decision making	Ruben van Montfort
	Failure tree	Ruben van Montfort
	Final report (and check)	Joost Remmers (Thijmen Jaspers Focks)
Local drainage system	Hydrological analysis: Rainfall	Jos Muller
	Hydrological analysis: Runoff	Joost Remmers
	Area description	Jos Muller, Ruben van Montfort
	Model and example scenarios	Jos Muller
	Final report (and check)	Jos Muller, Ruben van Montfort and Joost Remmers (Thijmen Jaspers Focks)
Primary drainage system	Scope	Joost Remmers
	Geometric data: fieldwork	Siebe Dorrepaal, Martijn van Wijngaarden and Joost Remmers
	Geometric data: processing	Siebe Dorrepaal, Martijn van Wijngaarden
	Hydraulic input	Joost Remmers
	Boundary conditions	Siebe Dorrepaal, Martijn van Wijngaarden
	Assumptions	Joost Remmers
	Calibration (and check)	Joost Remmers (Siebe Dorrepaal)
	Accuracy	Joost Remmers
	Example scenarios (1,2,3,5,6 and 8)	Joost Remmers
	Example scenario 4	Siebe Dorrepaal
	Example scenario 7	Joost Remmers
	Remarks	Siebe Dorrepaal, Martijn van Wijngaarden
	Final report (and check)	Joost Remmers (Thijmen Jaspers Focks and Siebe Dorrepaal)
Outfall structures	Maintenance	Peter Vijn
	Scope	Peter Vijn
	Methodology	Peter Vijn
	Example koker	Thijmen Jaspers Focks

	Remarks	Thijmen Jaspers Focks, Peter Vijn
	Final report (and check)	Thijmen Jaspers Focks, Peter Vijn (Joost Remmers)
Additional observations	Policy and Governance (and check)	Joost Remmers (Martijn van Wijngaarden)
	Technical Remarks	Ruben van Montfort and Joost Remmers
	Data management (and check)	Joost Remmers (Siebe Dorrepaal)
	Long term strategy	Siebe Dorrepaal
Results	Local drainage system (and check)	Jos Muller, Ruben van Montfort (Joost Remmers)
	Primary drainage channels (and check)	Joost Remmers (Martijn van Wijngaarden)
	Outfall structures (and check)	Thijmen Jaspers Focks, Peter Vijn (Joost Remmers)
Recommendations	General recommendations (and check)	Entire team (Joost Remmers, Martijn van Wijngaarden)
	Local drainage system (and check)	Ruben van Montfort, Jos Muller (Joost Remmers)
	Primary drainage channels (and check)	Joost Remmers (Martijn van Wijngaarden, Siebe Dorrepaal)
	Outfall Structures (and check)	Peter Vijn, Thijmen Jaspers Focks (Joost Remmers)

Table 21 Task division of project chapters/components appendices

Appendix	Subcomponents	Responsible team member(s)
Administrative notes	Full appendix	Joost Remmers
Additional material	Full appendix	Joost Remmers
Content DRR Report	Full appendix	Siebe Dorrepaal
Topographical data	Full appendix	Joost Remmers
Design/risk approach	Full appendix (and check)	Ruben van Montfort, Jos Muller (Joost Remmers)
Hydrological Analysis	Rainfall Analysis	Jos Muller
	Frequency analysis daily rainfall	Jos Muller
	Hourly rainfall intensity	Jos Muller
	Runoff analysis	Joost Remmers
	Remarks	Joost Remmers
Local drainage system	Area description addition	Ruben van Montfort
	Theoretical background model (and check)	Jos Muller (Joost Remmers)
Model clarification	Full appendix (and check)	Siebe Dorrepaal, Martijn van Wijngaarden (Joost Remmers)
Outfall structures	Full appendix (and check)	Thijmen Jaspers Focks, Peter Vijn (Joost Remmers)