Flood Management

A technical solution for the flooding problems encountered in the Lower Moshi area

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Moshi, Tanzania

Delft University of Technology
FT Kilimanjaro

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Preface

This is the final report of a multidisciplinary project for the course CIE4061 "Multidisciplinary Project" as part of the master program Civil Engineering at Delft University of Technology. The purpose of this project is to integrate several specialisations and use the gained knowledge to solve a civil engineering related problem. The work was carried out by five students during nine weeks in Moshi at the NGO FT Kilimanjaro (FTK).

First of all, we would like to thank our hosting organisation FT Kilimanjaro for inviting us to Moshi and supporting the project. Special thanks to Gerbert Rieks for welcoming us and helping us with arranging the accommodation and all the practical issues. In addition, we would like to thank Johnson Dickson for supporting our fieldwork trips and forming the bridge between the project and the local farmers and residents of the Lower Moshi area. Moreover, we would like to thank Joris de Vries for his suggestions regarding the project and introducing us to Keith Ward.

Secondly, we would like to thank our supervisors of the TU Delft. We got a lot of feedback from our main supervisors Ir. H.J. Verhagen, Ir. W.M.J. Luxemburg and Dr.Ir. M.G.C. Bosch-Rekveldt, both during the preparation and the execution of the project. We also would like to thank Dr. Ir. E. Mosselman for helping us with getting licences for computer programs and Ir. A. van der Toorn for answering questions we had during the calculation phase of the project.

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Furthermore, we would like to thank all the local contractors Jan Harm Beukema, Malvinder Bhamra, Dhiru Chauhan and engineers Peter Mapande and Maro Anenmose for advising us about the Tanzanian possibilities. Our meetings were very helpful to ensure that the solution can be implemented with the locally available materials, equipment and knowledge.

Finally, special thanks to Keith Ward who assisted us in the fieldwork and was always available to help with the geo-engineering parts and to give advice as an experienced engineer.

Moshi, 19th of January 2016

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Table of Contents

Summary

This report focuses on the flooding problems in the Lower Moshi area, Tanzania. These floods are the result of the extremely large catchment of the Kilimanjaro region in combination with large peaks in precipitation during the short and the long rainy seasons. The river bordering the area of interest cannot handle these quantities of water which results in the flooding of large plains. The local self-sustaining communities cannot harvest their crops during these seasons complicating their living conditions. Furthermore, essential facilities such as schools and medical care become unreachable during the floods and they lead to diseases being spread and an overall reduction of sanitation. Apart from their negative effects they also have positive outcomes however. The floods decrease the salinity of the soil of the agricultural land by flushing it, thus making it more fertile

Last year a group of students from TU Delft came to the Lower Moshi area to investigate the cause of the flooding and come up with solutions. The report from the previous group is referred to as the prefeasibility study. This year the client, FT Kilimanjaro, made the request to work out their solutions in more detail and come up with a cost estimate.

The goal of this project is to improve the welfare in the Lower Moshi area by developing a technical solution that prevents the short rain flooding and regulates the long rain flooding which is socially acceptable, feasible, and durable.

The prefeasibility study separated their solution into four clusters with corresponding solutions. These solutions included passive structures to control the floods. This was possible because one normative design discharge of 1/5 years was used. This implies that the discharge that the solution was designed for is exceeded once every 5 years.

A few discoveries were made while validating the prefeasibility design based on findings during meetings with informed parties and fieldwork. First, two of the clusters were considered problematic due to their locations. Furthermore, the peaks in discharge of the rainy seasons vary every year. They vary in such an extreme way, that a low peak in a long rain season could be equal to a large peak in a short rain season. The passive structures of the prefeasibility design cannot handle these newly discovered variations. Moreover, more normative discharges should be used so that their costs and benefits can be compared. Additionally, as for the prefeasibility design no soil samples were taken, it was determined though soil testing during the fieldwork that most of the soil in the area is silt.

The validation of the prefeasibility study resulted in the following conclusions. First, only two clusters were deemed necessary. Additionally, passive structures are not capable of handling the variations and needed to be reconsidered. Moreover, instead of using one normative discharge, three return periods for discharges should be considered, namely 1/5, 1/10 and 1/15 years. Lastly, the dimensions of the structures had to be changed because the soil was classified as silt.

In the initial design, three designs were made based on the changes. All designs were capable of preventing the short rain floods while controlling most of the long rains floods, depending on the used return period. The design consisted of the following elements, with varying dimensions for each return period. In cluster 1, a dike along the northern part of the river (Kikuletwa North) including manually operable spillways was designed. In cluster 2, a manually operable control structure at the bifurcation in the river was designed as well as widening the narrow part of the branch flowing south (Kikuletwa South). Various interviews were held with local contractors and engineers to get insight into the ability to build the design and the associated costs.

A cost benefit analysis was conducted, which resulted in the following arguments. The 1/5 year solution had the best benefit cost ratio caused by relatively small differences in benefits between the designs while having lower estimated costs. However, it did not take into account the indirect benefits effects of the additional safety of a longer return period. In addition, it would be easier to find backers for a safer solution and it is likely the farmers are willing to invest more in their lands if they are safer. Furthermore, the costs for the 1/15 year solution were not significantly higher than the 1/5 year and although the cost benefit ratio was the lower for the 1/15 year solution, it did not differ much from the alternatives.

The conclusion was drawn during a progress meeting with the stakeholders to continue with the 1/15 year solution based on the previous arguments. The interviews concluded that the design looked realistic to build with local available materials and equipment.

In the integral design**,** the 1/15 year solution was worked out into more detail; several changes and additions were made. Due to fast flow velocities scour can occur before and after the spillways and the control structure. A combination of soft and hard protection is needed for both. In addition, guiding dikes are necessary along the path of the spillways. Moreover, to decrease to required excavation a dike should be built on the eastern side of the Kikuletwa South. Finally, the control structure needs a crane that can be manually operated. The crane that was chosen to lift the gates of the control structure is a swivel crane with a hand chain hoist.

A morphology study for the prevalent discharge in the dry season was conducted. From the morphology study, it was found that most riverbeds would erode over time. Only the braided part of the Ronga experiences sedimentation. The morphology study concluded that the erosion of most riverbeds would increase the capacity of the rivers which can be seen as an additional benefit. Sedimentation of the Ronga will likely not cause problems, because the floods will flush away accreted sand. No increase in sediment entering the reservoir is expected.

The costs of the design that were determined consisted of the project management, completing the design, construction, realisation, operation and maintenance costs. The benefits of the design consisted of the agricultural and societal benefits expressed in monetary terms. The agricultural benefits are a combination of preventing the current loss of crops because of the flooding and the increase of the utilisation of the farmland. The costs and benefits were determined for the entire lifetime of the project. The costs of this design were determined to be 4.5 million US dollars, with an estimated range between 3.8 and 6.2 million US dollars. The present value of these costs is 3.5 million US dollars. The benefits of the design were estimated at 36 million US dollars with a present value of 11.5 million US dollars. The benefits combined with the costs results in a net present value of 8 million US dollars for the design and the associated cost benefit ratio of 3.23 with an internal rate of return of 22.8%.

Several recommendations were formulated for the next steps to fully realise the project. These recommendations were given for the preparation, construction, operation and maintenance and external factors. Many of these recommendations followed from a risk assessment that was carried out for all the preceding elements. The most crucial recommendations are firstly to survey the area, both horizontally and vertically, and to do more extensive soil testing. These measures are necessary to reduce the uncertainties in the gathered data and assumptions made. Secondly, it is advised to have an engineering company produce a detailed design and a bill of quantities and to take responsibility of the design. Thirdly, for the order of execution it is recommended that either the clusters are constructed simultaneously or to construct cluster 2 prior to cluster 1. Lastly, it is important that the local communities are educated on the solution so that they can optimize their cultivation of crops and assist with the maintenance and operation of the structure.

1 Introduction

This report focuses on the flooding problems encountered by the local community in the Lower Moshi area, Tanzania. These floods are the result of the extremely large catchment of the Kilimanjaro region in combination with large peaks in precipitation during the short and long rain seasons. The river bordering the area of interest cannot handle these quantities of water and therefore large plains flood. The self-sustaining community cannot harvest their crops during these rainy seasons and this complicates the already not ideal living conditions. Additionally, schools and hospitals become inaccessible and the floods cause the spreading of diseases and an overall reduction of sanitation. Although the rains have many negative effects, they also bring some positive effects. The floods flush out the salts of the arable land and thus make it more fertile.

The aim of this report is to improve the welfare in the Lower Moshi area by developing a technical solution that prevents the short rain flooding and regulates the long rain flooding which is socially accepted, feasible, and durable. This has been done by approaching the problem in the form of a feasibility study. The data for the report is collected from a prefeasibility study (Lower Moshi (2015)), an impact assessment (Rieks- van Hal, 2015), fieldwork and meetings with engineers and local contractors.

The report is divided in six parts which each start with an overview and end with a conclusion. The first part is the Basis of Design consisting of chapter 2 and 3. Chapter 2 contains a detailed description of the problem. This includes the project area, an explanation of the seasons, previous research, the scope and the stakeholders. In chapter 3 the requirements and conditions of this project are presented.

Then in the second part, Analysis, chapter 4 up to chapter 9 can be found. In chapter 4, the four clusters used in the prefeasibility study are described. The most important conclusions of the fieldtrips and meetings are found in chapter 5. In chapter 6 the solution proposed in the prefeasibility study is validated. Secondly, an alternative solution is presented with only two clusters and a choice between the two options is made using a multi-criteria analysis. Chapter 7 gives an overview of the bank full capacity of the rivers in the project area. Chapter 8 describes the design discharges of these rivers used for the design and a conclusion is given in chapter 9.

Hereafter the Initial Design is presented in chapters 10 up to 13, which is the third part. In this part the initial designs for three different return periods are given. In chapter 10, the initial design of cluster 1 the new and old dike and the spillways is described. Chapter 11 contains the initial design of cluster 2; the excavation and the control structure. In chapter 12 the cost and benefits for the designs are presented. In chapter 13, the return period that will be used in the next design phase is chosen.

The fourth part contains the Integral Design of the project and this part consists of chapters 14 up to 19. In the Integral Design, the changes made to the initial design are discussed. In chapter 14 the integral design of the dike and spillways is described. Chapter 15 shows the changes made to the excavation and the control structure. In chapter 16, the morphological effect on the rivers after the changes made will be described. The cost and benefits of the integral design are given in chapter 17. The integral design is validated in chapter 18 and a conclusion is given in chapter 19.

Part five, Risks and Implementation, consists of chapters 20 and 21. In chapter 20 the potential risks up to this point in the project are discussed. Chapter 21 describes the recommended implementation plan of the design, including preparation, construction, operation and maintenance plans.

The final part of this report is part six Conclusions and Recommendations. This part consists of chapters 22 to 24. In chapter 22 the final conclusions of this report are given. The limitations of the design are presented in chapter 23 and in chapter 24, the next steps that should be taken are recommended.

Part A - Basis of Design

In the basis of design, the information that is known before starting the project is discussed. First a description of the project is given. The project description consists of an explanation of the project area, the seasons, problem definition and project goal. Thereafter, studies that were done before this project will be introduced and the scope of this project will be formulated. Furthermore, the relevant stakeholders are listed and elaborated. After the project is described, the requirements, preferences and boundary conditions are listed and explained.

Figure 1: Picture taken during fieldwork, bifurcation Kikuletwa North

2 Project Description

The project description is structured as follows; first, the project area is defined, then the problem definition and the project goal are described. This is followed by the introduction of two studies that are related to this project. These two studies are a prefeasibility study, done by another group of students from the TU Delft and an impact assessment done by an independent consultant. This is followed by the formulation of the scope for this project and a list of relevant stakeholders.

2.1 Project Area

The project area is shown in Figure 2. In the north, the area is bound by TPC, a large sugarcane plantation, and in the south by the TANESCO border. Although floods south of the TANESCO border happen as well, this area is not considered part of the project area. This is because the land in this area is inundated from the reservoir Nyumba ya Mungu (Lower Moshi (2015)). The Kikuletwa river enters the project area in the north and also forms the western boundary. The Kikuletwa North splits half way the project area into the Ronga and the Kikuletwa South. The Ronga continues in an eastward direction while the Kikuletwa South continues southward. The TPC channel in the east forms the eastern boundary of the project area.

There are two villages in the project area, Chem Chem and Mikocheni Village. They each have sub-villages. Chem Chem consists of Majengo-Samanga, Miswakini, Chambogo, Forodhani and Kijijini. Chem Chem counts 4460 inhabitants and has a surface area of 50 km^2. Mikocheni consists of Mikocheni Kubwa, Mikocheni Ndogo (not in the figure) and Kirungu, counts 2166 inhabitant and has a surface area of 65 km^2 (government, 2012). The majority of farmers are situated in Majengo-Samanga (65%), Chambogo (17%) and Miswakini (8%), which are all in Chem Chem (Rieks-van Hal, 2015). They use the surrounding land to cultivate their crops.

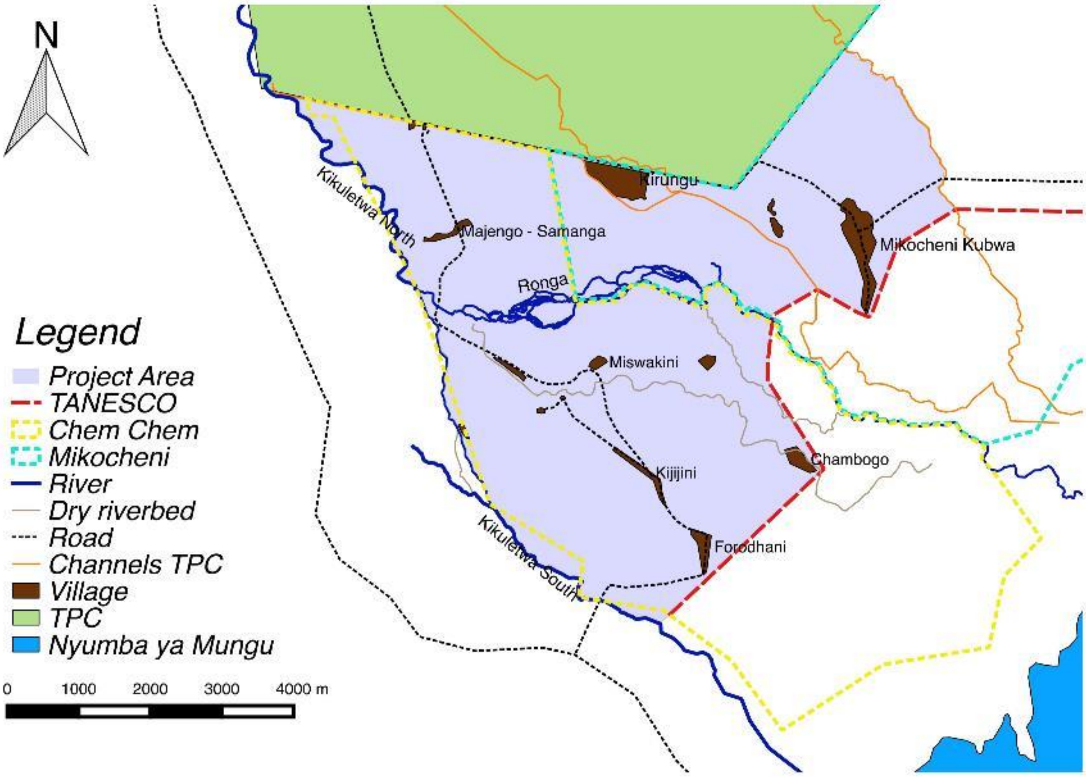

Figure 2: Project area

2.2 Seasons

In the project area, three seasons can be identified; the dry season, the long rain season and the short rain season. Large parts of the project area are subjected to flooding in the two different rainy seasons. A schematization of the seasons is shown in Figure 3. The short rains occur between October and January, the long rains between March and May and the dry season is in February and between July and September. In June, there is a waiting period. This waiting period could either be long rains or dry season, depending on the year.

Figure 3: Schematization seasons

Although the long rains are more severe, the short rains can also have the following consequences. The rains cause inaccessibility of some essential facilities, including schools and hospitals. Clinics in the villages cannot receive medical supplies and referrals to hospitals are not possible. Children cannot go to school and schools are even closed in some cases. The floods cause the spreading of diseases and an overall reduction of sanitation. This is mainly caused by the floods carrying a lot of waste and toilet pits being flooded. In the villages, houses and other buildings are destroyed. Sometimes farm animals even drown. Entrepreneurs such as shopkeepers cannot get supplies due to unavailability of transportation (Rieks- van Hal, 2015).

The short rains are unpredictable and due to this pose the biggest problem to farmers as they destroy crops. The long rains are predictable and most farmers do not harvest during these months. The long rains also have positive sides however, the fertility of the land is increased by the deposition of fertile grounds, the salinity is decreased and the soil is saturated enough to allow plants to grow.

2.3 Problem Definition

Due to the river's capacity not being able to handle the discharge flowing through it in both the long and short rainy seasons, it overflows. This brings both positive and negative effects to the Lower Moshi area. During the short rains, only negative effects can be identified. During the long rains, both negative and positive effects are present.

2.4 Project Goal

The goal of this project is to improve the welfare in the Lower Moshi area by developing a technical solution that prevents the short rain flooding and regulates the long rain flooding which is socially acceptable, feasible, and durable.

The sub-goals of this project are to:

- Identify the costs and benefits of the solution.
- Formulate an advice for the implementation of the solution.

2.5 Previous Research

2.5.1 TU Delft Group

In the period from November 2014 until January 2015, a group of students from Delft University of Technology investigated the flooding problem. They investigated the effects of the floods and provided preliminary solutions to improve the situation. Their solutions included building of dikes and control structures and deepening and widening of parts of the river. The different preliminary solutions that were designed and the research that was done will serve as a starting point for this report. For a more detailed description of the flooding problem, a referral is made to their report (Lower Moshi (2015)). This report will be referred to as the *prefeasibility study* from here on.

2.5.2 Impact Assessment

In the months September and October of 2015, an impact assessment was made to determine the influence of the floods and the potential benefits of reducing the effects of them. This impact assessment came up with monetary values of agricultural and societal losses due to the floods. This report will be referred to as *impact assessment* from here on (Rieks- van Hal, 2015).

Both of these documents will be referred to throughout this report as many assumptions follow from their findings.

2.6 Scope of the Project

The scope of this project includes the prevention of short rain floods and the management of the long rain floods in the Lower Moshi area. The design considerations include only the immediate area of the river's reach. The design will include locations of the constructions, quantities and materials types, dimensions, cost and benefit estimates and recommendations for the construction process and maintenance and operation plans. These elements will have the accuracy of a feasibility study.

The design of irrigation and drainage channels are not in the scope of the project. However, the solution should not discard that these are necessary. A detailed design will not be produced, as this lies out of the expertise of the current study.

2.7 Relevant Stakeholders

The prefeasibility study conducted a stakeholder analysis. In Figure 4 the most important stakeholders are listed. The most important points from the analysis conducted in the prefeasibility study will be presented in this section.

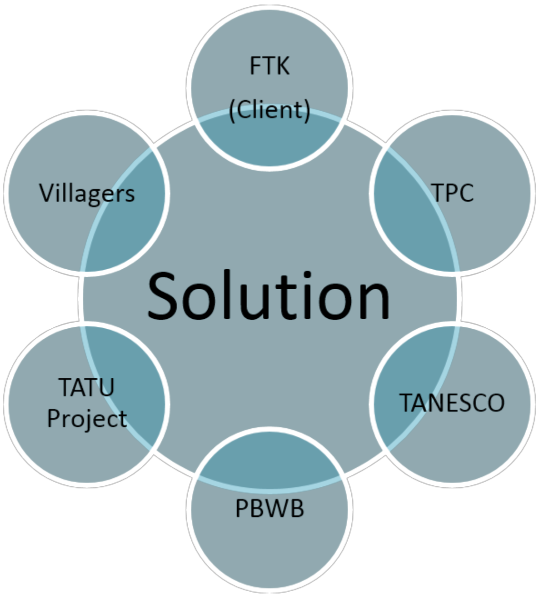

Figure 4: Stakeholders

2.7.1 FT Kilimanjaro (FTK)

FTK is a registered NGO in Tanzania. It is a joint initiative of the Dutch company FEMI Foundation and the sugarcane plantation TPC Company ltd. FTK is committed to improving the education, infrastructure, livelihoods and health in the Lower Moshi area in a self-sufficient and a sustainable way. FTK is the client of this project and therefore has a high interest and high power and should therefore be managed closely.

2.7.2 TPC

TPC Company ltd. is a large shareholder of FTK as part of their corporate social responsibility. It is a large sugar cane estate of 16,000 hectares located north of the project area. The company uses a large amount of water to flush its lands, mostly from the Weru-Weru (a river which joins the Kikuletwa upstream of the project area) and from boreholes. Their drainage channels go through the project area but do not affect the flooding. They have a high interest and a high power and should be managed closely.

2.7.3 TANESCO

TANESCO is responsible for the operation and maintenance of the Nyumba ya Mungu hydroelectric dam located south of the project area. The water needed for generating electricity comes from a reservoir located before the dam. The dam is a large source of energy for Tanzania. The responsible party for the management and availability of water to the reservoir is the Pangani Basin Water Board. The government regulates the amount of energy that the plant should generate. The water levels in the reservoir do not affect the water levels upstream of the rivers. The interest of TANESCO is low and their power high. They should be kept satisfied.

2.7.4 Pangani Basin Water Board (PBWB)

PBWB co-ordinates water resources management and water pollution efforts in the Pangani river basin area: '*Our Mission is to ensure that water resources are managed sustainably, through water governance and integrated water resources management principles*' (Pangani Basin Water Board, 2015).

The rivers flowing in the project area are under their jurisdiction. Their main objective is that the water levels in the Nyumba ya Mungu reservoir stay sufficiently high. The sediment deposit in the reservoir should not increase and the flow of water into the reservoir should not decrease. They have jurisdiction of the river and must approve of any implementation of a solution within its area. Their interest and power are high and should be managed closely.

2.7.5 Villagers

In the project area, there are two villages with each a number of sub-villages. In total, there are nine settlements, five belonging to Chem Chem and four to Mikocheni. Essential facilities such as schools and medical care become unreachable during the floods and they lead to diseases being spread and an overall reduction of sanitation. A survey conducted by FTK states: '*floods turned large parts of the village into an unsightly pool of water with floating trash and debris*' (Kilimanjaro, 2013). The main occupations of these villagers include fishing, farming, trading and cattle breeding. Their interest is high but their power low. They should be kept informed.

Farmers

Most of villagers rely of cultivation of farmland as their livelihood. The farmers mainly produce maize, beans, watermelons and tomatoes. A large part of this is for personal consumption the remainder is for selling. Almost half of the farmers gave the absence of accessible land as the main reason for not cultivating land (Kilimanjaro, 2013). Some farmers illegally use TANESCO property to cultivate crops because of the inaccessibility of their own land. Farmers require water for irrigation during the dry season and flooding of land during the wet season in order to fertilize the land with nutrient rich sediment. The farmers have high interest but low power. They should be kept informed.

Fishermen

After the construction of the dam, the reservoir contained many fish and served as a main source of food. However, the quality of water is decreasing and so is the fish population. Most of the fishermen are already farming (PBWB/IUCN, 2008). According to the impact assessment, fishermen benefit from the floods because breeding places are created in between grasses (Rieks- van Hal, 2015). The fishery has low interest and low power and should therefore only be monitored.

Livestock Keepers (Maasai)

The livestock includes chickens, cattle, goats and sheep. The livestock keepers migrate through the area to areas of potential food. Having access to these areas is thus of importance to them. Furthermore, their livestock can drown during the floods. The interest of these people is high but their power low. They should be kept informed.

Entrepreneurs

Entrepreneurs in the villagers include shopkeepers and transportation units. They experience a lack of customers during the floods. Their interest is high but their power low. They should be kept informed.

2.7.6 TATU Project

TATU project is an NGO working in Msitu wa Tembo, which is an area on the western side of the Lower Moshi area. The prefeasibility study concluded they are not of importance to this project, as they do not want a solution to the flooding problem. They have low interest and low power and need to be monitored.

3 Requirements, Preferences and Boundary Conditions

The prefeasibility study has, in combination with FTK, defined boundary conditions and requirements for this project. The same boundary conditions and requirements will be used as a starting point for this project.

3.1 Requirements

- The solution has to reduce the negative consequences of the yearly floods on farming activities.
- The solution has to reduce the negative consequences of the yearly floods on community infrastructure and livelihoods.
- Farming land should be protected from flooding during the short rain season.
- Aspects of the solution must be tangible or visible on short-term to gain trust of the villagers.
- The solution may not enhance the problem of saline soil in the problem area.
- The solution should be executable with local equipment and labour. This includes the equipment and experience of TPC.

3.2 Preferences

Some stakeholders have expressed their preferences, which they wish will be included in the solution. FTK also has some wishes they would like to include. Those desired conditions are:

- The flooding depth should be lowered during the long rain season.
- Some of the flood volume should be captured and stored for irrigation purposes during the dry season.
- Farmers would like to be able to open the gate near the dike breach to provide their farming land with water.
- The solution should not negatively influence the current infrastructure and other social facilities.
- The surrounding area (Msitu wa Tembo, TPC and Mikocheni Kubwa) should preferably not be influenced negatively.

3.3 Boundary Conditions

These conditions follow from the surroundings and involved parties:

3.3.1 Environmental/Natural Conditions

- When the solution entails changing riverbeds' elevations and profiles, the consequences of these changes on sedimentation should be evaluated.
- The amount of sediments that enter the reservoir should be kept to a minimum.

3.3.2 Legal Conditions

- When (a part of) the solution lies in the TANESCO area, permission has to be asked and received from the responsible organisation.
- Each measure on the river system must be announced to, and approved by the Pangani Basin Water Board.
- The solution has to comply with Tanzanian law.

3.3.3 Societal Conditions

- Local villagers must support the solution.
- Current farmers' irrigation access levels must be maintained.
- The district should be in favour of the solution.

3.3.4 Financial / Economic Conditions

- The solution should be economical feasible and realistic in relation to the FTK budget.
- The costs of the solution should be proportional to the benefits.

Part B – Analysis

The prefeasibility study is analysed in this chapter. This is done to ensure that the final solution meets all the requirements. Furthermore, the design discharges that are necessary for the design stages are elaborated.

Firstly, the clusters that were used in the prefeasibility study are explained. This is followed by practical research that was conducted. This research includes fieldwork that was done and meeting that were held with local contractors and engineers.

Hereafter, the solution proposed in the prefeasibility study is validated. This is done using an extensive validation system with eight criteria. During the validation, each part of the integral solution is validated individually. From the validation, the conclusion was drawn that the design in the prefeasibility study was in a very early stadium. More flaws were found than initially expected. Therefore, an alternative design is presented, staying as close as possible to the concept of the first design. The alternative design is compared to the old design using a multi-criteria analysis.

The rivers in the project area are then elaborated. The dimensions of the rivers are described and the maximum possible discharge in each river is presented. This is done based on the research from the prefeasibility report with only very small adjustments. Finally, the design discharges that will be used for this project are explained. This starts with the discharge entering the project area and followed by the discharge throughout the river system.

Figure 5: Picture taken during fieldwork, Samanga area flooded during the short rains

4 Cluster Explanation

In the prefeasibility study, the project area was divided into four different clusters. Figure 66 of the prefeasibility study is copied here, see Figure 6. The same idea of cluster formation is used in this project. However, the division of the area has been adjusted slightly. This is because the new formation made more sense taking the location of the different solutions (presented in the prefeasibility study) into account, see Figure 7. The biggest change noticeable is the smaller cluster 4. This is done because the lower part of the Ronga will not be used in any of the presented solutions.

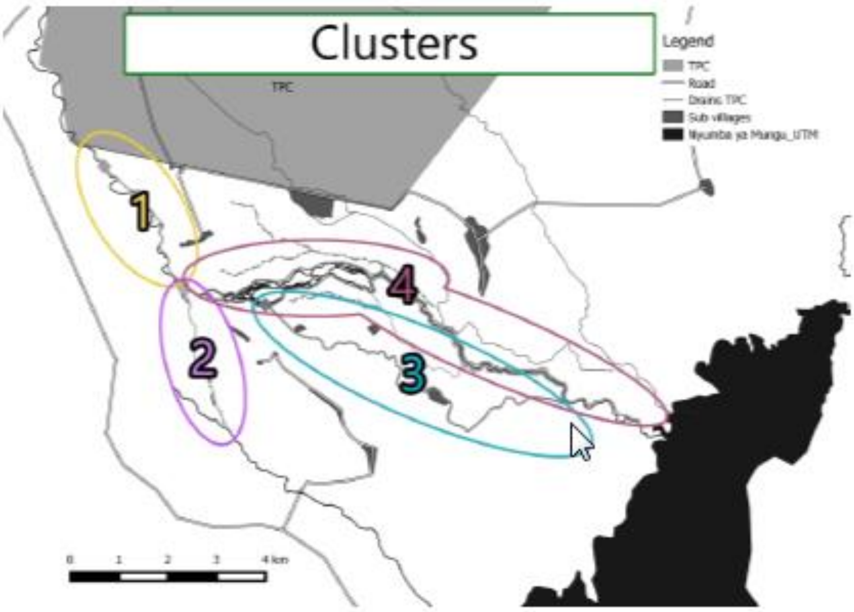

Figure 6: Clusters as determined in the prefeasibility study (Lower Moshi (2015)).

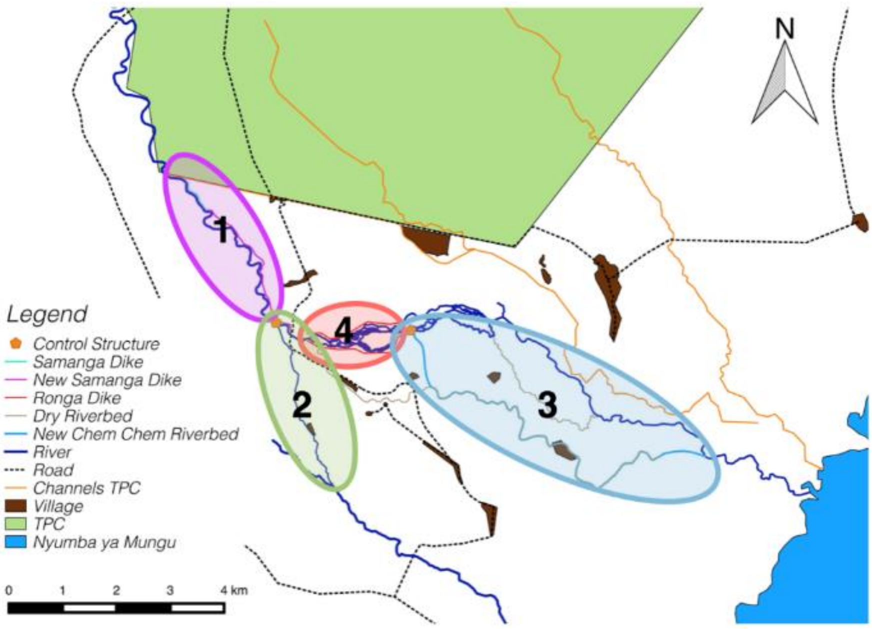

Figure 7: Cluster classification for this project

The new cluster division is indicated in Figure 7. Below the four clusters and the different components within the clusters are elaborated:

- Cluster 1: Kikuletwa North
    - Dike along Kikuletwa North;
    - passive spillways in the dike.
- Cluster 2: Kikuletwa South
    - Enlarging the Kikuletwa South;
    - passive control structure at the bifurcation.
- Cluster 3: Chem Chem
    - New channel from Ronga to Chem Chem riverbed;
    - passive control structure at the start of the Chem Chem channel.
- Cluster 4: Ronga
    - Short rain dikes along both sides of the river.

For a more detailed description of the solution that was proposed in the prefeasibility study the reader is referred to chapter 11 of the prefeasibility report.

5 Practical Research

In this chapter, the most important findings and conclusions of the fieldwork that was done and the meetings with local contractors and other experts that were held are presented.

5.1 Fieldwork

During the first four weeks of the project six field trips had been done. From those trips to different locations of the project area some important conclusions can be drawn. The most important ones are listed per cluster below.

- Cluster 1: Kikuletwa North
  - A lot of vegetation is present at the top and sides of the old dike. This has to be removed.
  - After a rain event a logjam can occur, which can lead to erosion of the riverbank. This has to be prevented.
  - At the moment only two spillways and a lot of irrigation canals along the Kikuletwa North are present. After building the new dike more spillways will be necessary.
  - A lot of vegetation is present along the Kikuletwa North. This has to be removed.

- Cluster 2: Kikuletwa South
  - More space is available on the west side of the Kikuletwa South Small than on the east side. Therefore, it should be best to build the control structure on the west side.
  - At some point, the river diverges and later converges again. At the confluence, the soil is hard and no deeper than 0.5 m can be excavated.
  - Just before the river diverges, an old riverbed of the Kikuletwa South can be found. This can be used instead of the part from where the river diverges.

- Cluster 3: Chem Chem
  - The planned location of the Chem Chem channel is going through a village called Miswakini. The planned location should be changed.
  - A lot of vegetation is present in the old riverbed. This has to be removed.
  - At the planned connection between the Ronga and Chem Chem channel the Ronga is braided. It will be difficult to find a location where a stream of the Ronga will be large enough to discharge enough to the Chem Chem channel.

There was no field trip planned to Cluster 4 because it was inaccessible.

During the fieldtrips, soil samples were taken at various locations and it was determined on the spot that most of the soil was silt. This was determined through a series of tests, with the supervision of a geotechnical engineer.

5.2 Meetings

During the time of the project, several meetings with local contractors, engineers and Pangani Basin Water Board have taken place. The most important findings and conclusions from these meetings are stated below.

- Local contractors
    - These meetings provided us with unit prices and cost estimations for:
        - Quantities
        - Personnel
        - Design
        - Equipment
        - Clearing of vegetation
    - Additionally, they could provide an estimate for the time span for building.
- Engineers
    - There is no pure clay available in the region.
    - Cost estimations for
        - Quantities
        - Clearing of vegetation
- Pangani Basin Water Board:
    - PBWB owns 60 meters on both sides of the river according to the law.
    - Data from IDD1.

6 Validation Prefeasibility Design

Several criteria will be used to validate the design proposed in the prefeasibility study. The criteria are based on the prefeasibility study, the impact assessment, meetings with stakeholders and informed parties and fieldwork. With the use of these criteria the weak and strong points of the design can be identified, which will allow improvements to be made.

6.1 Validation System

In this paragraph the different criteria will be discussed which will be used to validate the designs.

6.1.1 Risks

The risks associated to the design should be kept to a minimum. The uncertainties in the data and assumptions should be taken into account to reduce the risks.

6.1.2 Cost Effectiveness

In the meeting with J. Gadek, a retired engineer from the World Bank, it was proposed that several designs should be made for different safety levels. The return periods will be based on the number of unwanted floods in the area, as some floods are good for the land. The cost and benefits associated with those safety levels should provide a positive result for the analysis.

6.1.3 Water Management of the Agricultural Land

The irrigation and drainage of the land is not something that will be specifically designed. However, the design should not hinder the irrigation or the drainage of the lands. Furthermore, it should allow for flooding of the land when it is required. In other words, the design should only be a barrier against the water when it is required. The solution should distribute the discharges that they are designed for, accounting for uncertainties that may occur.

6.1.4 Locations for the Designs

The planned locations for the designs should be available for construction. This means that the different stakeholders agree with the construction locations and that the area is reasonably accessible. Additionally, there should be no obstacles in the way, which are too expensive to remove, in monetary or social terms. Furthermore, the impact of the location of the design should be as small as possible on the surrounding area.

6.1.5 Resources and Construction Methods

The design should be constructible with local resources (equipment/materials/knowledge). This also includes the availability and properties of the soil in the area. The properties of the soil determine if the design will function as intended and if the foundation of the structures will be sufficient. Additionally, the design should have a low complexity of execution. It should be possible to build the structures in the dry season, as floods and rain will only complicate the project. As well as that, the construction method should be taken into account in the design. It should be possible to build what is designed.

6.1.6 Morphological Effects

The sediment transport in the river and the potential changes of the sediment transport resulting from the solution should be taken into account. Changes to the river system can have long-term morphological effects on the river and the maintainability of the design. The design should not make matters worse and consider these effects.

6.1.7 Operation

The operation of the design should be consistent; the same types of structures should be used as much as possible. I.e. only controllable spillways or only non-controllable spillways, not a combination as this may be favourable to one farmer but not to another. Furthermore, the design should be easily operable; it should require as little effort as possible.

6.1.8 Maintenance

The design should be easily maintainable, if possible not requiring specialist knowledge. Furthermore, the effort required for maintenance should be as low as possible.

6.1.9 Longevity

During the first meeting on the 10th of November 2015, a time horizon for the project of 20-25 years was proposed. For that time span, the design should function for the set criteria. The solution should be able to adapt to climate change and other changing circumstances.

6.2 Validation of the Prefeasibility Study

The prefeasibility design will be validated using the different criteria. The validation will be done for the separate clusters, which can be seen in Figure 8.

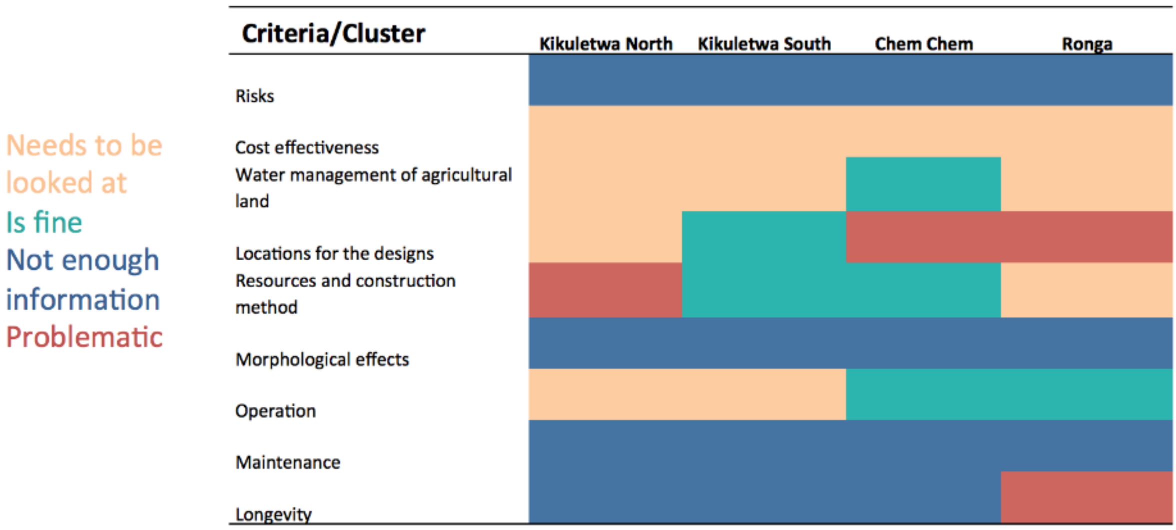

Figure 8: Validation of the prefeasibility design

In Figure 8, the results of the validation of the design can be seen. All of the criteria that are green meet the requirements. For those that are orange further investigation is necessary. Red criteria indicate the most problematic situations. These situations are discussed in detail later. For all other criteria more research will be required to determine if changes are required, it is not known yet if these criteria will pose a problem. This is because either in the design, no attention has been paid to these criteria, or insufficient information was available at that time. In Appendix A.1 - Validation of the Prefeasibility Design the entire validation can be seen.

6.2.1 Problems Found During the Validation

In this part, the very problematic situations that were found during the validation are elaborated. These are the resources and construction of the Samanga dike, the location of the Chem Chem river, the location of the Ronga connection and the longevity of the Ronga.

Resources and Construction Method Samanga Dike

There are two problems regarding the construction materials of the dike. In the prefeasibility study, the dike is designed with a sand core and a clay cover. The first problem is that it is difficult and expensive to get the large amount of sand needed for the dike at the construction location. The second problem is that the clay cover is likely not available at all.

Another problem relates to the construction method. During a meeting with a local contractor it was discovered that the dike crest should be 2.5 meters at least so that a compactor can go over it. The designed dike crest width is not wide enough for a compactor. This makes it impossible to compact the soil sufficiently. While improving the design this should be taken into account.

Locations for the Chem Chem Design

During the field visits, a number of problems were identified regarding the location of the Chem Chem solution. Both the location of the Chem Chem channel and the old riverbed are problematic, see Figure 9. The planned Chem Chem channel goes through a village, called Miswakini and relocation of the channel results in a longer channel. The old riverbed also goes through a village, called Chambogo. This is very problematic because many houses should be removed or relocated. Even if this would be possible, the river would form a huge barrier for the local population. Furthermore, the riverbed is wider than expected, resulting in the risk that the flow speed in the river will not be high enough to maintain a significant flow. These large problems are hard to overcome which makes the added value of the Chem Chem solution doubtful.

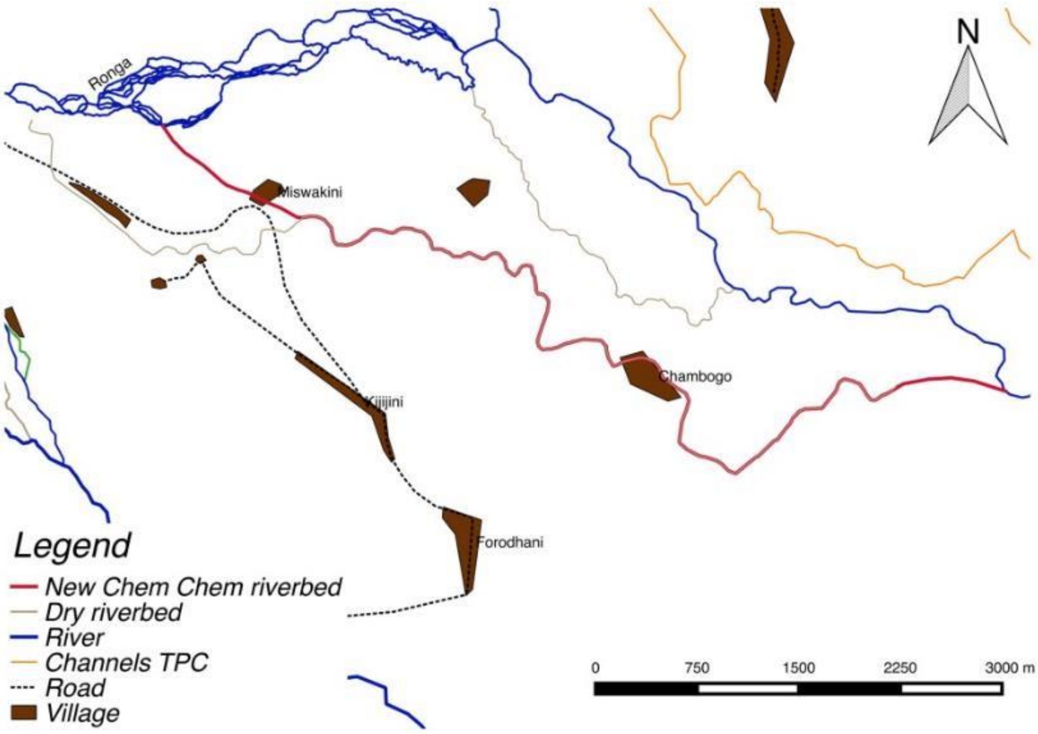

Figure 9: Villages blocking the planned course of the Chem Chem

Locations for the Ronga Design

The planned connection between the Ronga and the Chem Chem channel must be adjusted, when the Chem Chem channel's location is changed, for reasons explained above.

The location of the planned control structure at the bifurcation point between the Ronga and the Chem Chem channel is not ideal. The Ronga is very braided at this point, therefore it is hard to determine the exact discharge that will flow to the Chem Chem. This is mainly a concern when there is a low water level, because there might not be enough discharge available for the Chem Chem area.

Longevity Ronga

The dikes that are planned along the Ronga have a very low height. Therefore, the dikes might not be recognised or respected by the local population. This could possibly result in people walking over or on the dike. It is very likely that this will damage the small dikes after a longer period.

6.3 Proposed Solution

The validation of the design that is proposed by the prefeasibility study indicates some problems. Therefore, an alternative is proposed in this chapter, which will be compared with the original design. A decision between the two options will be made based on a multi criteria analysis.

6.3.1 The Two Options

Option 1 is the design that is presented in the prefeasibility study (Lower Moshi (2015)). Option 2 is an alternative that has been conceived based on the foregoing validation. This option is based on the design of the prefeasibility study and is intended to stay as close as possible to the original design, which was supported by the stakeholders. Both options are presented briefly below.

Option 1

The original design consists of four parts:

1) Construction of a dike along the Kikuletwa North including spillways.
2) Widening of the Kikuletwa South Small and building a control structure at the bifurcation point.
3) Construction of a new riverbed that connects the old Chem Chem river to the Ronga and building a control structure at the resulting bifurcation point.
4) Construction of small dikes along the first part of the Ronga.

Option 2

The alternative consists of two parts:

1) Construction of a dike along the Kikuletwa North including spillways.
2) Widening the Kikuletwa South Small and building a control structure at the bifurcation point.

The widening of the Kikuletwa South Small will be larger in option 2 than in option 1. As a result, the Kikuletwa South is able to discharge more water, which will lead to a decrease of discharge in the Ronga. For that reason, the new Chem Chem riverbed and the dikes along the Ronga will no longer be needed.

6.3.2 Choice between the Options

In order to make a choice between the two options, a multi- criteria analysis was performed. The criteria used are the same as those that were used for the validation of the prefeasibility study. Each team member gave a score out of five to every criterion, with five being the best score. The scores were averaged for each criterion and were then multiplied by their respective weights. The weights, criteria and the respective scores are described in Appendix A.2 - Weight Criteria and MCA Scores.

Table 1: Scores of the MCA

Design option	MCA Score
Option 1: Prefeasibility study	2.6
Option 2: Alternative design	3.3

In Table 1 the scores of the respective options for the MCA can be seen. It is concluded that Option 2: the alternative design scores better. As such, this design option shall be used for this project, as demonstrated in Figure 10.

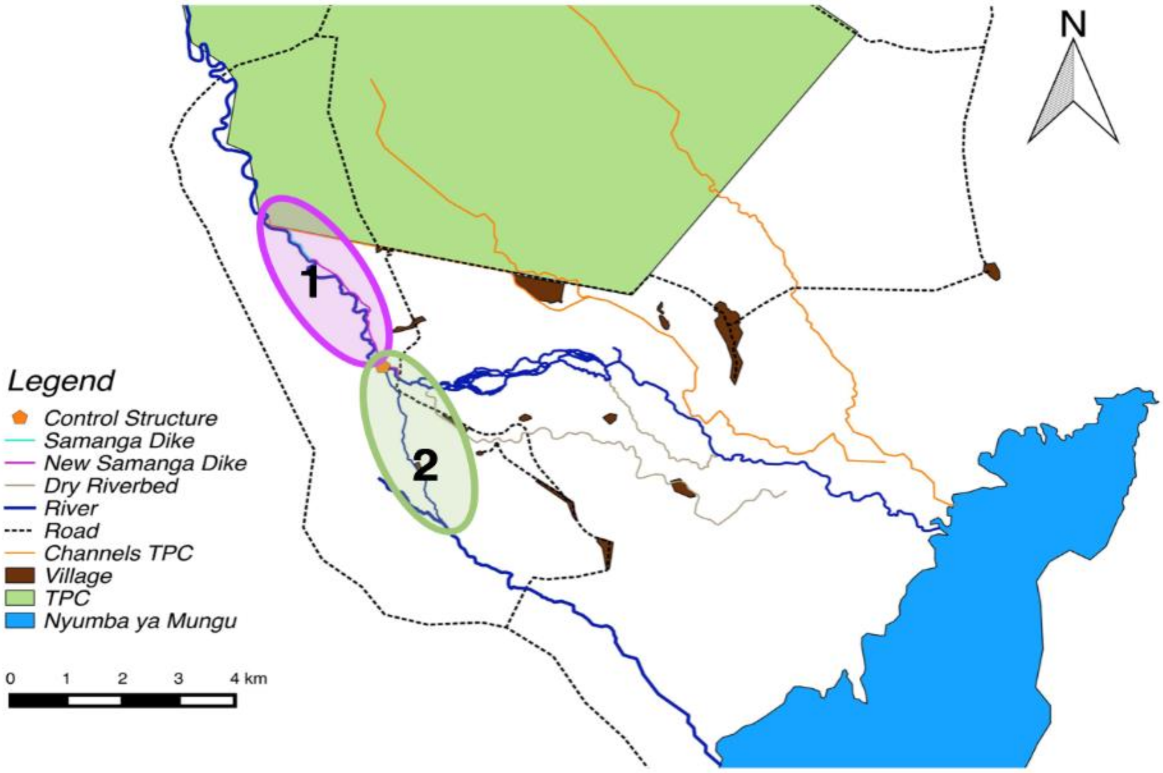

Figure 10: Chosen alternative

7 The Rivers

In this chapter, the rivers that are important in the project area will be elaborated. Most of the data that is presented will be a repetition from the prefeasibility study. It is however included in this report to raise the readability and clearness of the report as a whole. First, the dimensions of the rivers will be determined, followed by the maximum discharge possible through the rivers.

7.1 Dimensions of the Rivers

In the prefeasibility study, some measurements were done to determine the dimensions of the rivers. The river system is divided into six river stretches, see Figure 11. For most calculations, it is assumed that the dimensions within each stretch do not vary. There is only one cross-section measurement for each stretch, therefore it is recommended to do more measurements before making the detailed design. This can be done using a survey team that can measure the width and depth of the rivers at multiple locations. In Appendix A.3 - Current River Dimensions a summary of the most important dimensions is given. These dimensions are assumed to be correct and will therefore be used in this project.

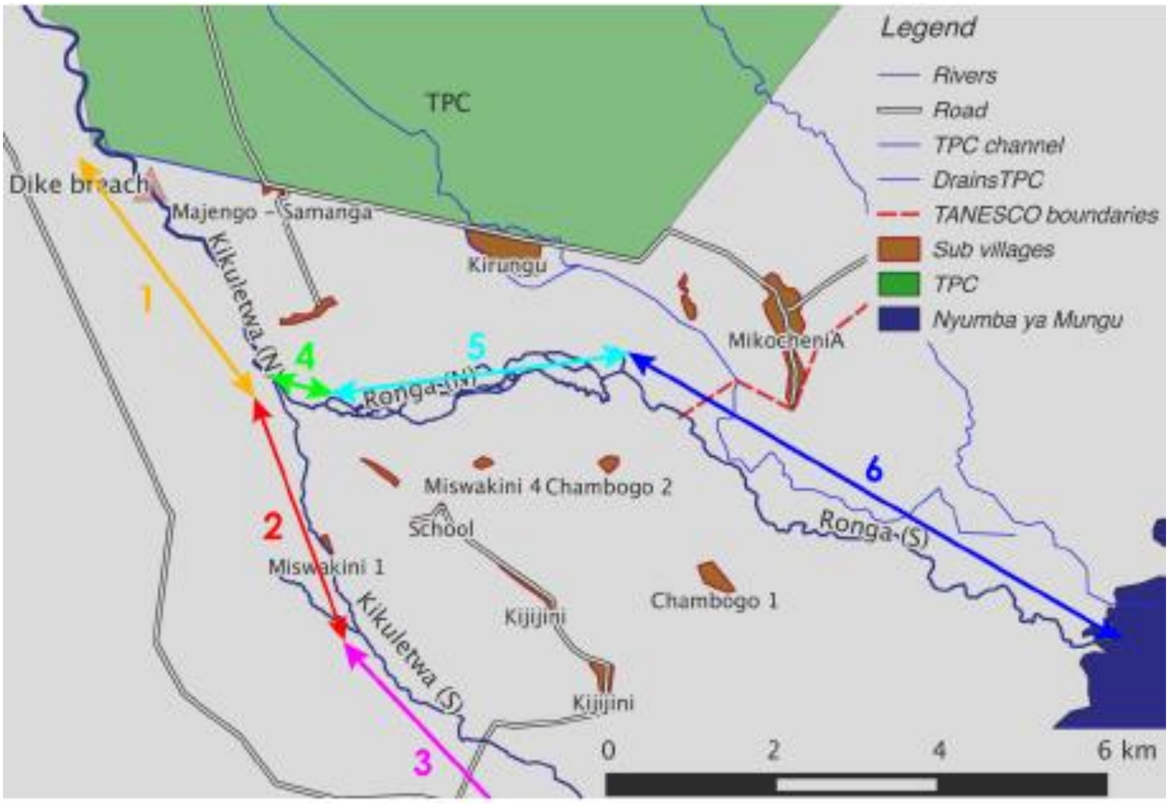

Figure 11: Definition of the river stretches (Lower Moshi (2015)).

In this report, one stretch is added to the existing six. This stretch is part of the Kikuletwa North, from the IDD1 measurement station to the start of stretch 1. The newly defined stretch will be called Stretch 0. The IDD1 measurement station is located approximately 11 kilometres upstream of where the Kikuletwa enters the project area. IDD1 is a measuring station owned by Pangani Basin Water Board which measures the water level of the Kikuletwa North twice a day upstream of the project area.

7.2 Maximum Discharge per Stretch

The discharges have been recalculated using the same method as in the prefeasibility study. This was done to verify the results and to change them in some cases. The resulting discharges are listed in Table 2. Note that the discharge in stretch 5 is different than the discharge that was calculated before. The discharges here are the maximum discharges possible for the listed stretches, referred to as the bank full discharge. If there is a higher discharge in reality, the river will overflow.

Table 2: Maximum possible discharge per section

Stretch	Discharge Q [m^3/s]
0 IDD1	159
1 Kikuletwa (N)	48
2 Kikuletwa (S) small	2
3 Kikuletwa (S) large	230
4 Ronga (N) before braiding	34
5.1 Ronga (N) braiding 1	10
5.2 Ronga (N) braiding 2	15
5 Ronga (N) braiding total	25
6 Ronga (S)	27

8 Design Discharges

For the determination of dimensions of the works in this project, design discharges are used. The discharges are formulated using data collected in the past years. With the data, a chance of exceeding can be linked to a certain discharge, called a design discharge. First, the discharges into the project area will be determined. Then the flow through the rivers is followed in order to determine the discharge in different parts of the project area. This is done using the cluster approach that was explained before.

8.1 Discharges into the Project Area

The design discharges entering the project area have been formulated in a deterministic way. This entails that normative discharges have been taken for different return periods based on measurements from IDD1. As proposed by J. Gadek different return periods will be used for this study so that their costs and benefits can be compared. For this project the return periods are 1/5, 1/10 and 1/15 years, read as 'once in the 5, 10 or 15 years'. This implies that the discharge is exceeded once every 5, 10 or 15 years respectively. So the higher the denominator of the return period the safer the solution will be. A safer solution also implies a higher design discharge however and thus higher costs.

The discharge that is measured at the IDD1 station is assumed to be equal to the discharge that comes into the project area through the Kikuletwa North. For this project, there will be two different design discharges, the design discharge for the long rains and the design discharge for the short rains. This is because there are different criteria for the discharge during the long rains and discharge during the short rains. During the long rains, flooding is a requirement while during the short rains, flooding should be prevented.

One other discharge is important for this project. This is the lowest possible peak discharge during the long rains. This discharge is important because much of the land in the project area should be flooded during the long rains. This is in order to ensure that the soil remains fertile.

8.1.1 Long Rain Design Discharges

The long rain design discharges are the peak discharges that can occur linked to a certain probability, indicated as return period. The long rain design discharges are determined in Appendix A.4 - Design Discharges. The discharges are determined using water level data from the IDD1 measurement station. In Table 3, the results are displayed.

Table 3: Design discharges long rains, determined from the annual maxima graph

Return period [years]	Design discharge [m^3/s]
1/5	190
1/10	220
1/15	230

During the long rains, it is very important that a large part of the project area is flooded. Therefore, flooding should be possible even for the low long rain discharges that can occur. The lowest possible peak discharge during the long rains is also determined in Appendix A.4 - Design Discharges. The discharge that will be used is 45 m^3/s.

8.1.2 Short Rain Design Discharges

The short rain design discharges will be determined in a slightly different way because less data is available. In Appendix A.4 - Design Discharges is described how the short rain design discharges are determined. In Table 4, the results are displayed. Notice that the design discharges of the short rains are higher than the lowest possible peak discharge during the long rains.

Table 4: Design discharges short rains

Return period [years]	Design discharge [m^3/s]
1/5	52
1/10	68
1/15	75

8.1.3 Variations of Discharges

In the previous sections, the maximum discharges that can occur during both the long rain and short rain season have been described. What has not been taken into account is that there are large variations in the discharges per year. These vary in such an extreme way, that a low peak in a long rain season could be equal to a large peak in a short rain season, as can be seen in Figure 12. In this figure the long rain discharges have been plotted from low to high, while the short rain discharges have been plotted from high to low. These variations need to be considered because it could lead to land not being flooded, when this is desired. Passive structures cannot handle these variations. Therefore, the passive structures that were recommended in the prefeasibility study should be reconsidered in the next design phase.

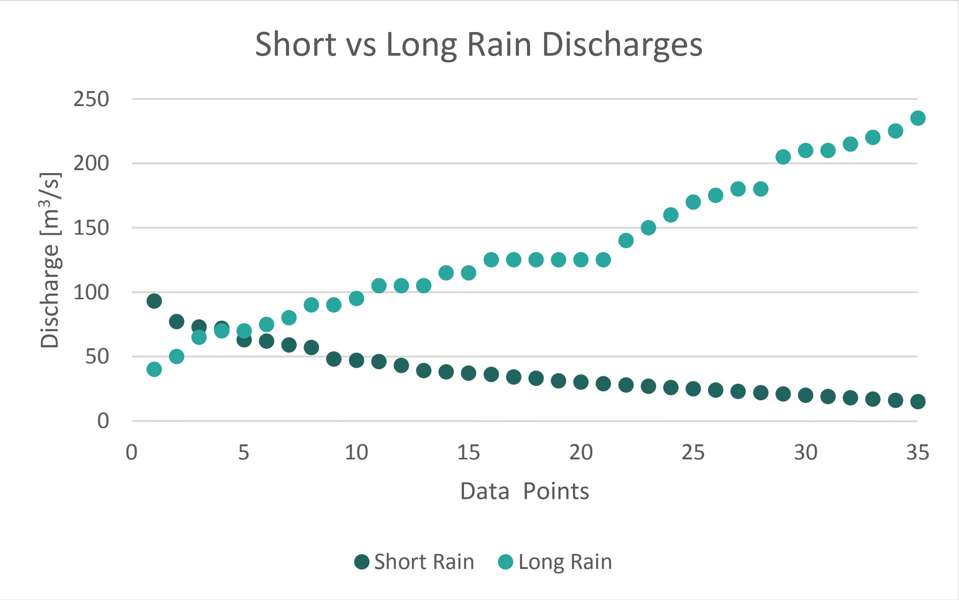

Figure 12: Variations discharges

8.2 Discharges through the Clusters

The design discharges that enter the project area have been determined. The next step is to analyse the flow through the river system. First, the flow through cluster 1 will be looked at in order to determine the discharge that reaches the bifurcation after of the Kikuletwa North. Then calculations for both the Samanga floodplain and the Ronga floodplain are elaborated. When the discharge arriving at the bifurcation and the floodplains are determined, the distribution of water between the Ronga and the Kikuletwa South will be looked at. This is done in order to determine what discharge is needed in the Kikuletwa South. Following this approach, all of the discharges in the project area are determined.

8.2.1 Cluster 1

In cluster 1, the design discharge (originating from IDD1) enters the project area and is then divided between the Kikuletwa North and the floodplains on the Msitu wa Tembo side, assuming that a dike will stop the floods to the Samanga floodplain. The bank full discharge of the Kikuletwa North is 48 m^3/s and therefore insufficient for the peak design discharges (both for short and long rains).

Approach

The discharge for the river and the floodplain are formulated using Manning's equation. For both the river and the floodplain, the flooding height is unknown and equal. Furthermore, the design discharge is known for 1/5, 1/10 and 1/15. Solving these equations thus results in a design flooding height and the distribution of discharge between the river and the floodplain, see equation 1. For the assumptions, calculations and uncertainties of the resulting discharges Appendix A.5 - Design Discharge Cluster 1 should be consulted upon. A schematization of the river and the floodplain can be seen in Figure 77.

$$Q_{river} + Q_{floodplain} = Q_{design} \tag{1}$$

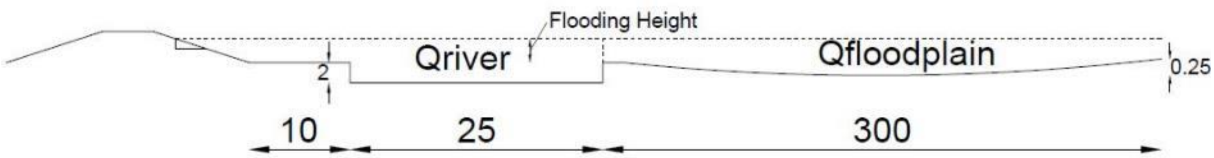

Figure 13: Schematization flooding

Resulting Discharges

Long Rains

The resulting flooding heights for cluster 1 are listed in Table 5.

Table 5: Design flooding heights cluster 1 long rains

	Q (1/5)	Q (1/10)	Q (1/15)
Flooding height [m]	0.65	0.76	0.79

This design height of flooding for the long rains for each design discharge will be the starting point for the design height of the dike. The resulting discharge distributions resulting from these design heights are listed in Table 6.

Table 6: Design discharge distribution long rains

	Q (1/5)		Q (1/10)		Q (1/15)	
	River	Floodplain	River	Floodplain	River	Floodplain
Discharge [m^3/s]	71	119	77	143	79	151
Distribution [%]	37	63	35	65	34	66

Short Rains

The resulting flooding heights for the short rains are listed in Table 7.

Table 7: Design flooding height short rains

	Q (1/5)	Q (1/10)	Q (1/15)
Flooding height [m]	-0.0085	0.0988	0.1407

The design flooding height for the discharge of 1/5 years is negative because the floodplain has enough capacity to absorb the flood. The resulting discharge distributions are listed in Table 8.

Table 8: Design discharge distribution short rains

	Q (1/5)		Q (1/10)		Q (1/15)	
	River	Floodplain	River	Floodplain	River	Floodplain
Discharge [m^3/s]	39	13	43	25	45	30
Distribution [%]	74	26	64	36	60	40

8.2.2 Floodplains

Along the Kikuletwa North and the Ronga the land is used to cultivate crops. To ensure that the soil remains fertile the area should be flooded during the long rains. The floodplain of both the areas and how much discharge is needed to flood those areas will be discussed. The floodplains were indicated by the client as areas that are flooded for a longer time.

Samanga Floodplain

The new dike along the Kikuletwa North will prevent the floods of the Samanga area, see Figure 14. In order to flood the Samanga area during the long rains spillways have to be built in the dike. To design these spillways, the discharge that is needed to flood the area has to be known. The discharge through the spillways will not be subtracted from the incoming discharge flowing through the Kikuletwa North and the Msitu wa Tembo floodplain. This is because the spillways can also be closed resulting in the discharge without subtraction. The maximum discharge for the Samanga area is shown in Table 9. For the calculations of the total discharge see chapter 7 The Rivers.

Table 9: Maximum discharge Samanga area

	Samanga area
Total discharge [m^3/s]	10.6

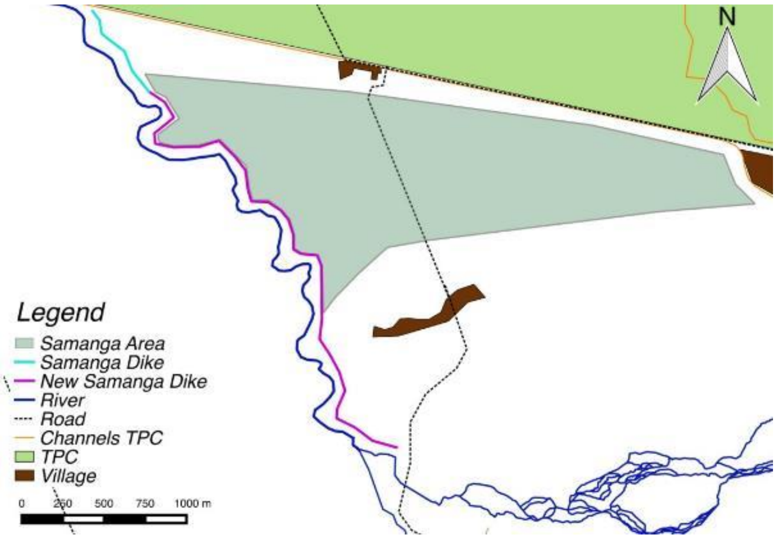

Figure 14: Samanga Area

Ronga Floodplain

The area along the Ronga that needs to flood during the long rains is indicated in Figure 15. To make sure that either enough water or not too much water is discharged through the Ronga to flood the area, a control structure is placed in the Kikuletwa South. The discharge that is needed to flood the Ronga area is given in Table 10. For the calculations of the discharge, see chapter 7 The Rivers.

Table 10: Maximum discharge Ronga area

	Ronga area
Maximum discharge [m³/s]	43

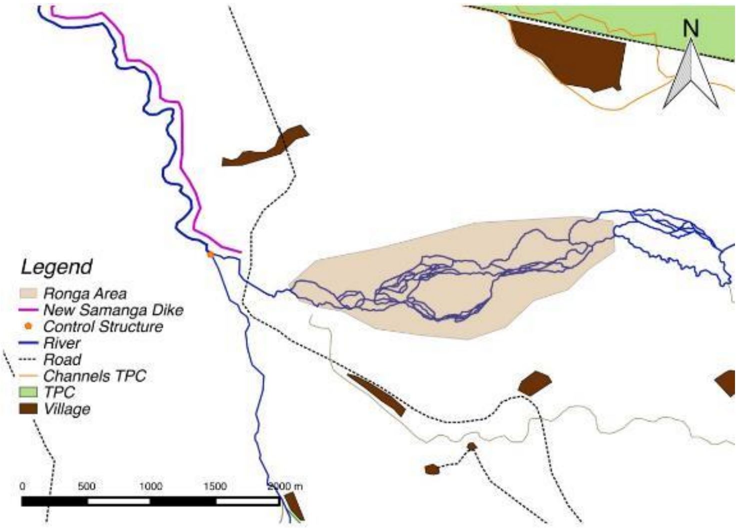

Figure 15: Ronga Area

8.2.3 Cluster 2

The design discharge that enters at the bifurcation point is divided between the Kikuletwa South and the Ronga. During the long rains, flooding of the Ronga is a requirement and therefore the design discharge should be higher than the capacity. During the short rains, flooding of the Ronga should be prevented and therefore the design discharge of the Ronga should be equal to the minimum capacity of the Ronga, see chapter 7 The Rivers.

Approach
The design discharge during the long rains is determined by the fixed discharge through the Ronga. The design discharge during the short rains is determined by the minimum discharge capacity of the Ronga. In other words, the discharge of the Ronga during the long and short rains is fixed and the discharge through the Kikuletwa South is variable for the different return periods.

Assumptions
For the discharge of the Ronga for the long rains, the following assumption is made; see the section about the Ronga Floodplain: the design discharge of the Ronga is 43 m^3/s.
For the fixed discharge of the Ronga for the short rains, the following assumptions are made, see chapter 7 The Rivers the minimum capacity of the Ronga is 25 m^3/s.

Resulting Discharges
Long Rains
The resulting discharge distributions for the long rains are listed in Table 11.

Table 11: Design discharges long rains

	Q (1/5)		**Q (1/10)**		**Q (1/15)**	
	Kikuletwa S	Ronga	Kikuletwa S	Ronga	Kikuletwa S	Ronga
Discharge [m^3/s]	28	43	34	43	36	43
Distribution [%]	39	61	44	56	46	54

Short Rains
The resulting discharge distributions for the short rains are listed in Table 12.

Table 12: Design discharges short rains

	Q (1/5)		**Q (1/10)**		**Q (1/15)**	
	Kikuletwa S	Ronga	Kikuletwa S	Ronga	Kikuletwa S	Ronga
Discharge [m^3/s]	14	25	18	25	20	25
Distribution [%]	36	64	42	58	44	56

Conclusion
From the resulting discharges can be concluded that the design discharge of the New Kikuletwa South is larger than the current capacity of the Kikuletwa South Small, see chapter 7 The Rivers. This means that the Kikuletwa South Small has to be widened and deepened. The discharge of the long rains is normative for the design of the New Kikuletwa South.

8.3 Discharge Flowcharts

The following discharge flowcharts have been made to make the discharges that flow through the project area clear. They schematize the discharges that have been described earlier on in the chapter. A flowchart has been made for both the short rains and long rains.

8.3.1 Short Rain Season

Figure 16 shows a discharge flowchart for the short rains.

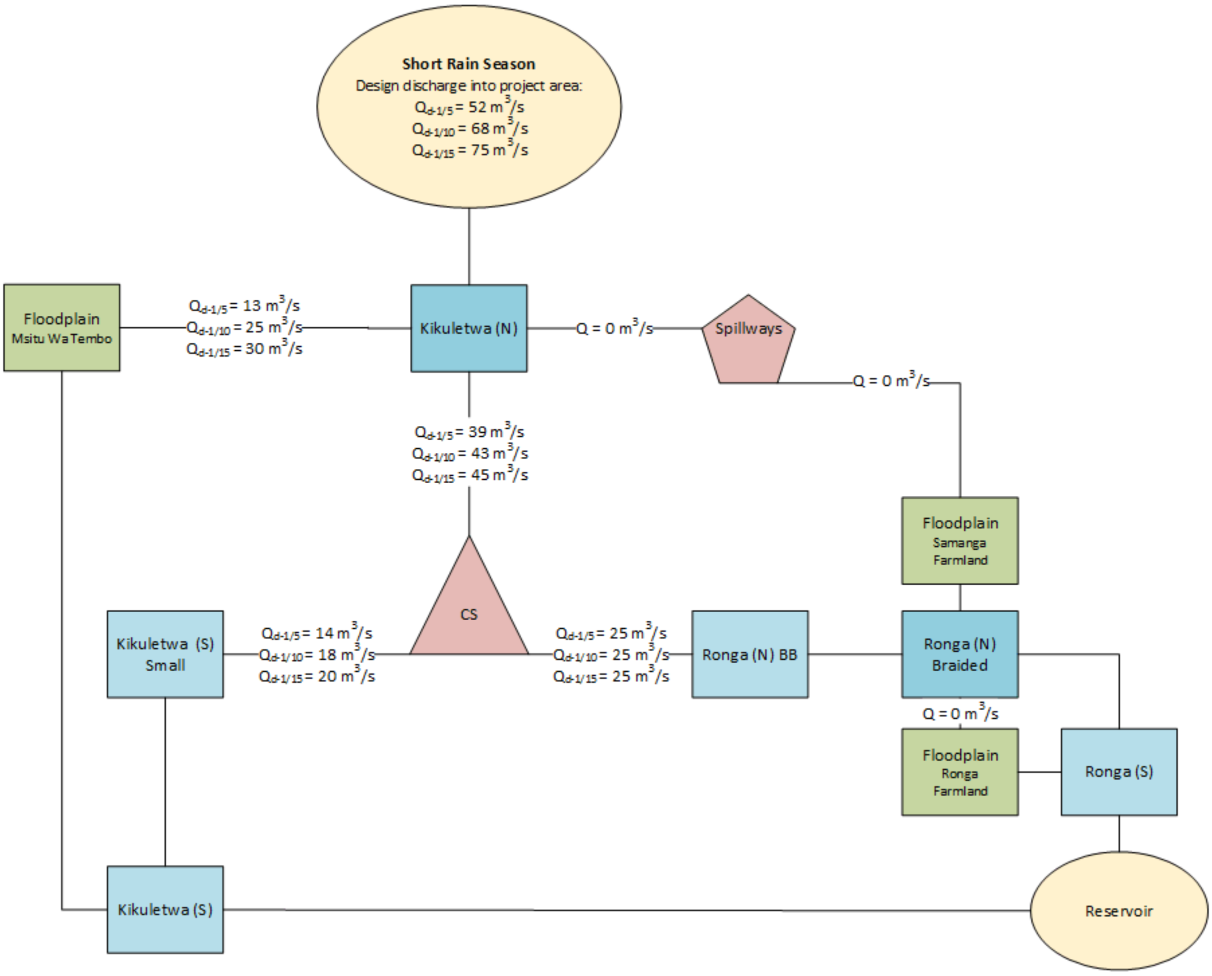

Figure 16: Discharge flowchart short rain

8.3.2 Long Rain Season

Figure 17 below shows the discharge flowchart for the long rains.

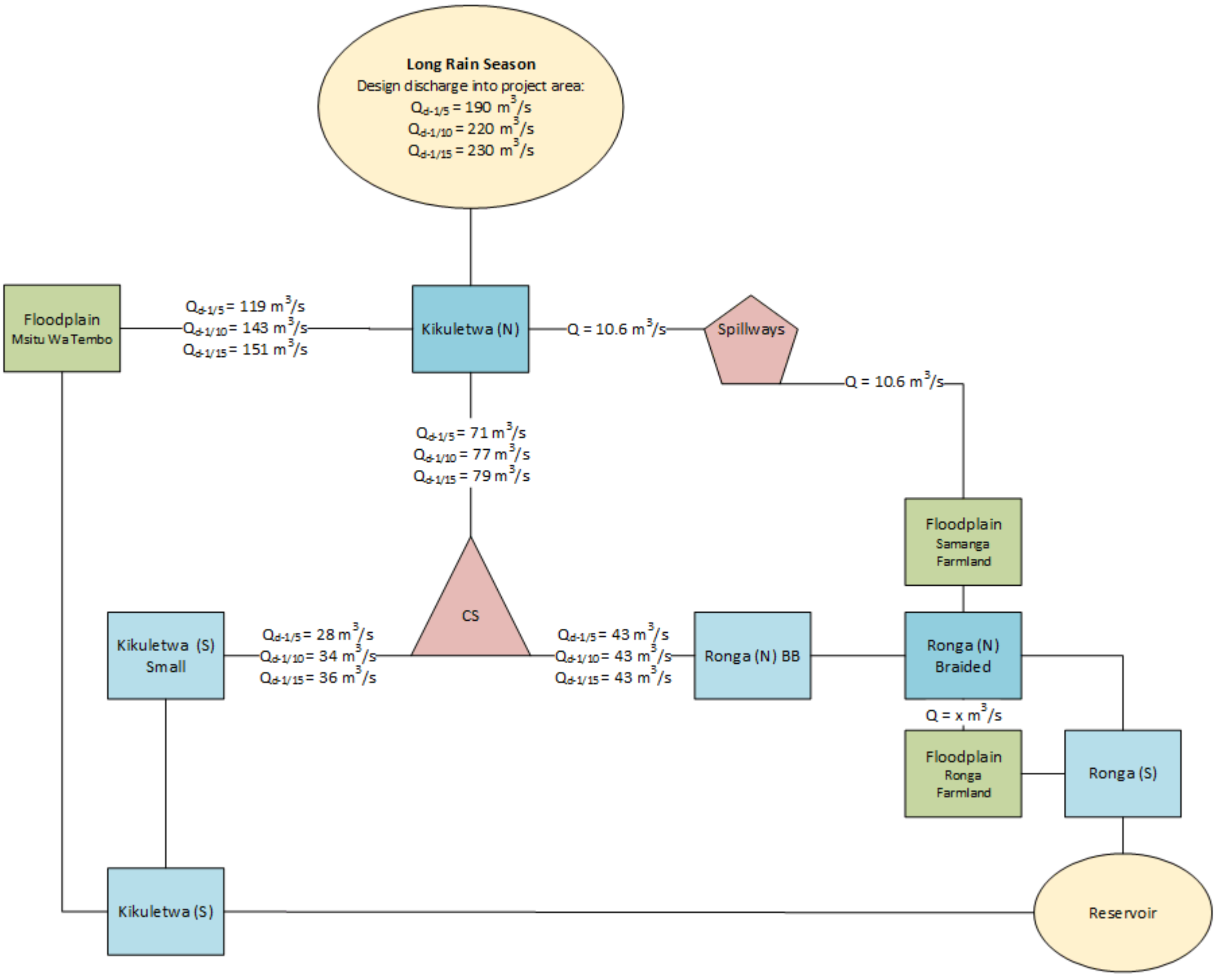

Figure 17: Discharge flowchart long rain

9 Conclusions Analysis

During field visits most of the soil in the area was determined to be silt through observations. The dimensions of the structures have to be changed because the soil is classified as silt.

While validating the prefeasibility design based on the findings during meetings with informed parties and fieldwork a few discoveries were made. First, two of the clusters were considered problematic due to their locations; the Chem Chem and Ronga clusters (cluster 3 and 4). Only clusters 1 and 2 are deemed necessary after conducting a multi criteria analysis and consulting with the client. The validation was done at a very early stage of the project and is mainly based on field observations. This implies that the changes made during the validation will not be the only changes to the prefeasibility design. The solution that will be used in the next phase consists of:

Cluster 1:

- A dike along the Kikuletwa North including passive spillways.

Cluster 2:

- Widening the narrow part of the Kikuletwa South.
- A passive control structure in the Kikuletwa South, just after the bifurcation.

Secondly, the resources and construction method of the Samanga dike needs to be altered, as it is not feasible.

For the design discharge, three return periods for discharges should be considered, namely 1/5, 1/10 and 1/15 years. Furthermore, the peaks in discharge of the rainy seasons vary every year. They vary in such an extreme way, that a low peak in a long rain season could be equal to a large peak in a short rain season. Passive structures cannot handle these variations and therefore need to be reconsidered.

By studying the rivers and discharges the conclusion is drawn that the capacity of the rivers cannot handle the maximum discharges that were determined, which is why the flooding occurs.

Part C - Initial Design

This part includes an explanation of the initial designs within cluster 1 and 2 and the associated costs and benefits. The main goal is to select one of the three return periods. The selected return period will be used in the integral design stage of this project. Thus, the quantities mentioned in the initial design are not the final quantities. The final locations, dimensions and costs will be treated in part D – Integral Design.

In cluster 1; the initial design of the new dike is described. The location, dimensions and construction method of the dike are stated. Following the dike design, the initial design of the spillways is shown. This initial spillway design includes the requirements for the discharge that needs to flow through them, the locations of the different spillways, the dimensions and a brief explanation of the construction and operation.

In cluster 2, the initial designs of the excavation of the New Kikuletwa South and the control structure at the bifurcation are presented. The current situation is defined firstly. Following this, the initial design of the New Kikuletwa South is described. This includes the location and dimensions of the excavation. Hereafter the initial design of the control structure is described. The discharge that needs to flow through the structure is defined. Then the type of structure, the dimensions of the structure, the construction and operation are stated.

Finally, in the cost benefit analysis, the costs resulting from the dimensions of the initial designs are calculated. The potential benefits associated to the designs are compared with these costs, to reach a conclusion on the return period selection.

Figure 18: Picture taken during fieldwork, local ferry for crossing the Ronga

10 Cluster 1 Initial Design

Cluster 1 is the northern part of the project area. This cluster consists of the Kikuletwa North, which enters from the border of TPC and stretches until the bifurcation point, see Figure 19. Along the first part of the river, a dike is present as well as two spillways. The plan is to continue this dike until the bifurcation point and to include more spillways. Firstly, the design of the new dike is described, followed by changes to the old dike and then a description on the spillways is given.

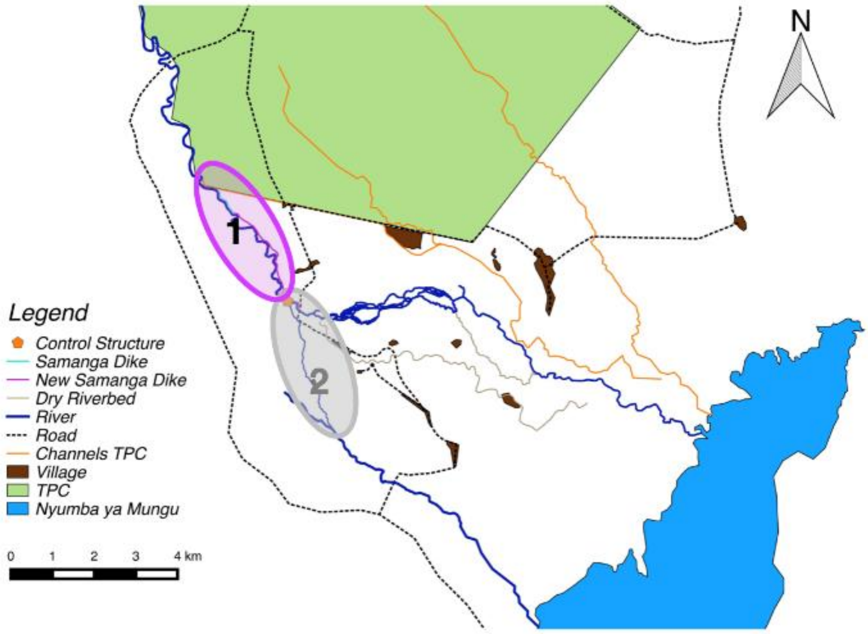

Figure 19: Location cluster 1

10.1 New Samanga Dike

Here the location, dimensions, failure mechanisms and construction of the new Samanga dike are elaborated.

10.1.1 Location

The dike should be situated 10 meters from the riverbanks at least. This buffer is taken due to possible erosion at the outer bends of the river as well as possible observed logjams causing erosion of around seven meters. Additionally, the alignment of the river continually changes, and this buffer can take this into account. Making the dike as straight as possible is advisable for simplicity and reduction of materials needed.

10.1.2 Dimensions

The following discharges and flooding heights were determined from the discharge analysis in chapter 8 Design Discharges and are listed in Table 13.

Table 13: Discharge and flooding heights Kikuletwa North

	Q (1/5)	Q (1/10)	Q (1/15)
Discharge [m^3/s]	71	77	79
Flood height [m]	0.65	0.76	0.79

By performing checks on the possible failure mechanisms, it is possible to determine whether the assumed dimensions are good. This iterative process is repeated until the dimensions are such that the structure is safe for the considered failure mechanisms. Presented below are the final dimensions for which all the checks of the failure mechanisms are sufficient. For the calculations of the failure mechanisms and the associated checks see Appendix B.1 - Design Calculations Dike.

Crest Height

The crest height has been determined by using flooding height resulting from the design discharge and adding a freeboard of 0.65 meter. This freeboard is sufficient for almost all the deviations identified in Appendix A.5 - Design Discharge Cluster 1. One deviation has been identified for which the flooding height could be even higher than this freeboard. This would require the bed slope to be very small. Looking at the elevation map this seems highly unlikely.

Width

The width of the crest has been determined by using the average dimensions of the existing dike, which is 4 meters, as can be seen in the next section. During field visits, it became apparent that the farmers use the dike as a pathway too. The crest should be at least 2.5 meters so a compactor can go over it, according to a local contractor. 3.5 meters is taken as the normative width for 1/5 years increasing with 0.5 meters for increasing design discharge. A slope of 3.5:1 (horizontal to vertical) is used as the normative slope. This is less steep than the slopes of the current dike (which have an average slope of 2:1). This slope is taken at both the river and the landside for simplicity.

The resulting dimensions of the initial design can be found in Table 14 and Figure 20. The values in the figure have been rounded up.

Table 14: Dimensions initial design new Samanga dike

	Q (1/5)	Q (1/10)	Q (1/15)
Design water level [m]	0.65	0.76	0.79
Freeboard [m]	0.65	0.65	0.65
Design height [m]	1.31	1.41	1.45
Slope [-]	1V3.5H	1V3.5H	1V3.5H
Horizontal length slope [m]	4.59	4.94	5.08
Dike crest [m]	3.5	4	4.5
Total width dike [m]	12.67	13.87	14.65

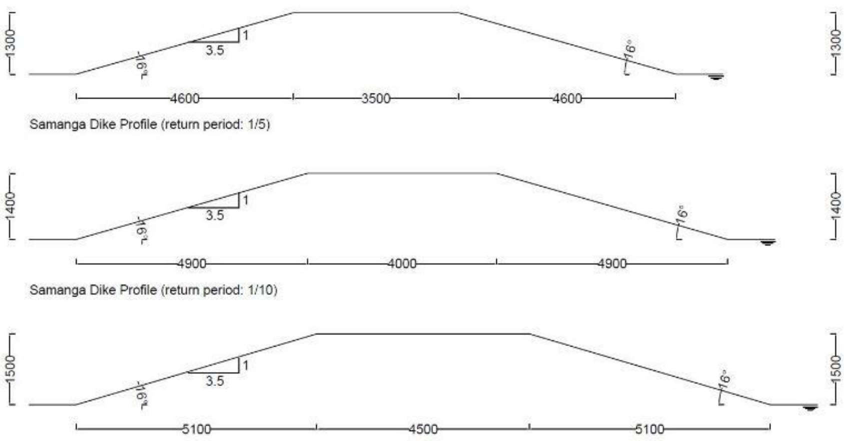

Figure 20: Dimensions initial design new Samanga dike [mm]

10.1.3 Failure Mechanisms

A number of failure mechanisms were tested for the given dimensions of the dike. A detailed explanation of these can be found in Appendix B.1 - Design Calculations Dike. The failure mechanisms looked at were mass instability, seepage, external erosion, internal erosion and settlement.

10.1.4 Construction

The volume of soil that is needed for the dikes will be provided by the land on the other side. The volume of soil needed for each option is known per meter. However, the dike will not follow the river exactly so probably less material is needed. This number also excludes a possible decrease in volume due to compaction. Assuming 75 cm is dug out the length of digging can be determined, as can be seen in Table 15.

Table 15: Soil for dike

	Q (1/5)	Q (1/10)	Q (1/15)
Area soil [m^2]	10.6	12.6	13.9
Digging length [m]	14.1	16.8	18.5

The construction will have the following steps (CIRIA, 2013); these will be explained further in chapter 21 Implementation:

- Site clearing
- Topsoil removal
- Construction of embankment
- Top soiling and vegetation

10.2 Existing Samanga Dike

The existing dike and the new Samanga dike can be seen in Figure 21. A dike is present along the first 0.6 kilometres of the Kikuletwa North entering the project area. The dike was built two years prior by the government. They used the land in front of the dike to construct it.

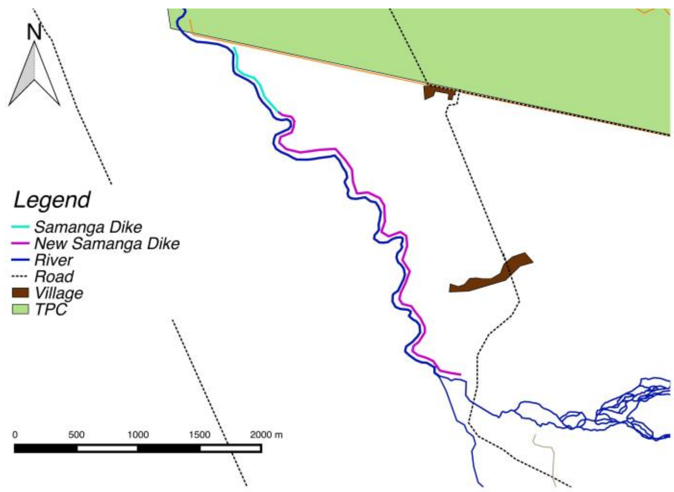

Figure 21: Existing and new Samanga dike

10.2.1 Dimensions

Several profiles of this existing dike were measured. The dimensions of the profiles are given in Table 16 and an example of one of the profiles is shown in Figure 22.

Table 16: Dimension existing dike

Profile	**Height river side [m]**	**Height land side [m]**	**Width crest [m]**	**Total width [m]**	**Slope river side [-]**	**Slope land side [-]**
B	1.1	1.2	5.3	10.3	1:1.8	1:2.5
D	1	1.1	3.4	7.2	1:1.4	1:2.2
E	1	1	3.3	7.3	1:2.3	1:1.7
Average	**1.0**	**1.1**	**4.0**	**8.3**	**1:1.8**	**1:2.1**

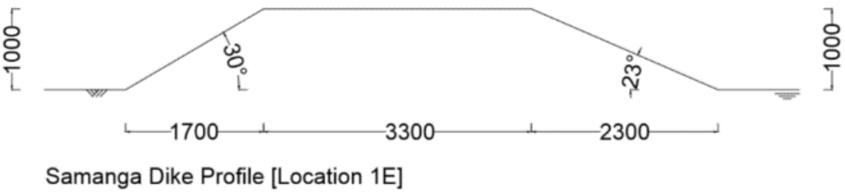

Figure 22: Example profile existing dike

During field visits it was observed that there was a lot of vegetation growing on the dike. It was also noted that some parts of the dike were too close to the river. These parts may have to be moved or strengthened.

10.2.2 Failure Mechanisms

The failure mechanisms were not tested for the current dike. The current dike should be adapted to the chosen design of the new dike. This means that it will have to be heightened and widened.

10.2.3 Construction

The current dike needs to be adapted to the new chosen profile. This means that the vegetation currently growing on it will need to be removed before it is heightened and widened.

10.3 Spillways Samanga Dike

This part will treat the initial design of the new spillways that are needed to ensure flooding of the Samanga area. Firstly, the requirements that are used are presented. Secondly, the locations of the spillways in the Samanga dike are determined and lastly the design is described and dimensions are provided.

10.3.1 Requirements

A number of requirements need to be met in order to have a successful spillway design:

- Irrigation should still be possible, through the spillways.
- The Samanga area should always be able to flood, even when low water levels during the long rains occur.
- The discharge through the spillways should not be too high, because this can cause erosion in the Samanga area.
- The construction must have successful tests on its design failure mechanisms.

10.3.2 Irrigation of the Samanga Area

One of the requirements is that, the structures that are built do not negatively influence the current situation with regard to irrigation. At this moment there are several irrigation channels located where the Samanga dike is planned and building the dike would severely hinder the irrigation. In order to meet the requirement, the spillways will need to be designed to allow for irrigation. For the design of irrigation possibilities, two variants were considered:

1. Improving the current situation by improving the access to water.
2. Preventing the situation from worsening by not diminishing the access to water.

Improving the Current Situation

In order to improve the current situation, the availability of water for farming purposes during the dry seasons can be increased. The lowest water levels were looked at in the Kikuletwa North in the past four years and this data was used to ensure that 97.7% of the time water would be available, using irrigation channels through the spillways, see Appendix B.2 - Design Calculations Spillways for the details.

The irrigation channels would be 1.3 meters deep and the spillways would be 1.3 meter below ground level. Getting the water past the dike is not the problem in this variant, the real difficulty is that after the water is past the dike it is still 1.3 meter below ground level. A deep and complex irrigation network can distribute the water across the Samanga area. This is however a very expensive and unrealistic solution.

Preventing the Situation from Worsening

In order to prevent the situation from getting worse, the spillways can be designed in such a way that irrigation is possible through the spillways in the same way as irrigation is possible in the current situation. This is done by placing the spillways on locations where a lot of irrigation channels are present currently. This is the variant that will be used in the spillway design.

10.3.3 Discharge Requirements Spillways

In the requirements set for spillways with regard to the discharge, two extremes can be distinguished; maximum water level and minimum water level, as explained in chapter 8 Design Discharges.

During the maximum water level, it is important that the discharge through the spillways is not too high, causing erosion in the Samanga area. The resulting maximum discharge through the spillways is 10 m^3/s.

During minimum water level it is important that there is a high enough discharge through the spillways, so that the Samanga area can flood sufficiently. In Appendix B.2 - Design Calculations Spillways is determined that the minimum water level was exceeded for approximately 14 days during the long rains. Two days are necessary to flood the entire area

Taking these requirements into account, five spillways are needed to flood the area sufficiently. The calculations to determine this can be found in Appendix B.2 - Design Calculations Spillways.

10.3.4 Type of structure

As previous explained the passive structures in the prefeasibility design had to be reconsidered, see Part B- Analysis. It is not possible to design a passive control structure that fulfils all the new requirements Consequently, a choice has been made to make a new design for the spillways. The new spillways have a design that is similar to the existing spillways. This means that they are designed as manually operated vertical gates under submerged underflow. This type of structure is also preferred by the local farmers.

10.3.5 Locations

Two important factors should be taken into account while determining the initial locations of the spillways. The first factor is that the irrigation of the land behind the dike must not deteriorate with respect to the current situation. During field visits, many irrigation channels along the Samanga area were mapped. They can be seen as the blue points in Figure 23. In order to ensure enough irrigation possibilities, it is favourable to place the spillways close to the areas where many irrigation channels are present currently.

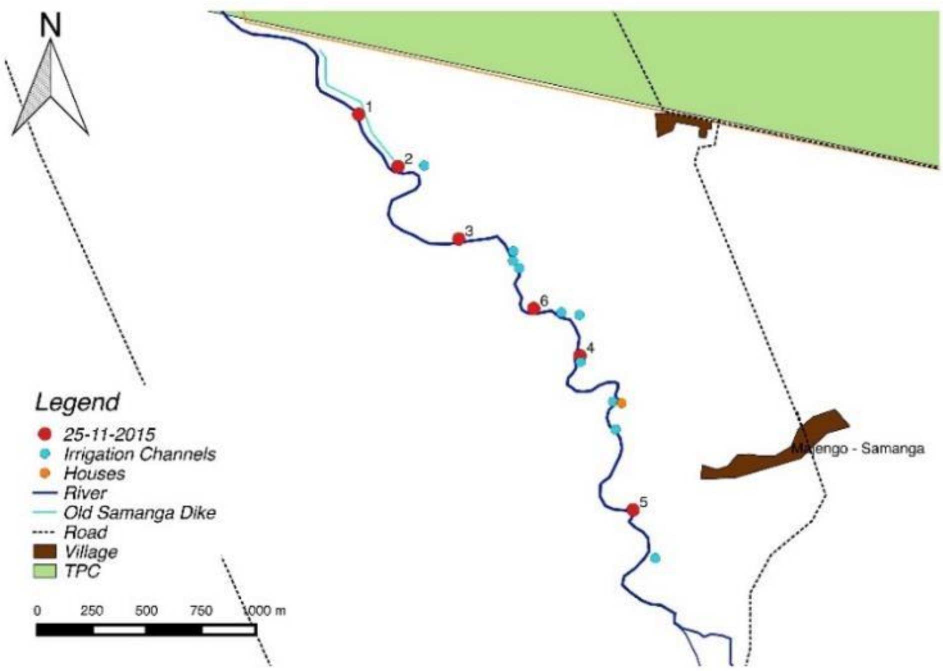

Figure 23: Locations irrigation channels

The second factor has to do with the possibility of erosion at the outer bends of the river. Erosion in the vicinity of a spillway can cause unwanted failure. Therefore, it is not desirable to place a spillway at an outer bend. The preferred location would be at a straight part of the river. Considering these two factors, the locations in Figure 24 have been identified as suitable locations for spillways.

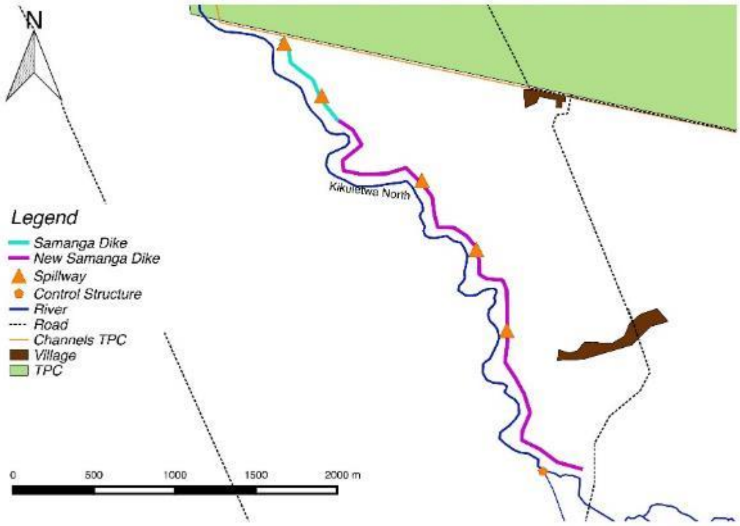

Figure 24: Locations initial design spillways

10.3.6 Design

The new spillways have a design that is similar to the existing spillways. This means that they are designed as manually operated vertical gates under submerged underflow; see Figure 25 for an impression of the design. From section 10.3.3 Discharge Requirements Spillways, it is concluded that five new spillways need to be built.

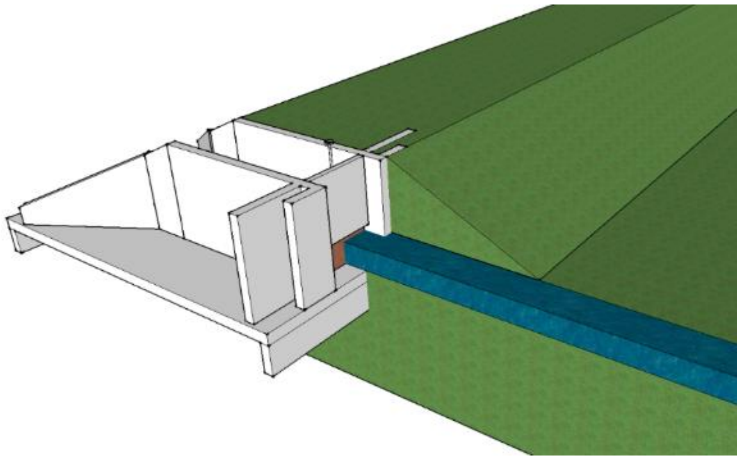

Figure 25: Impression initial spillway design

The dimensions of the spillway are determined by looking at the dimensions of reference projects. By performing unity checks on the possible failure mechanisms later on, it is possible to determine whether the assumed dimensions are good. This iterative process is repeated until the dimensions are such that the structure is safe for the considered failure mechanisms. Presented below are the final dimensions of the initial design for which all the unity checks of the failure mechanisms are sufficient. For the calculations of the failure mechanisms and the associated unity checks see Appendix B.2 - Design Calculations Spillways.

Dimensions

Foundation

The foundation of the spillway will be a slab. This slab foundation is situated 0.6 meters below the dike, to allow for irrigation when the water level is low and flooding during long rains with low water levels. It starts under the crest of the dike and continues until past the slope of the dike at the landside. At each side of the slab, there will be a coffer for seepage and scour purposes. The dimensions of the foundation and coffers can be found in Table 17. This foundation is assumed to be identical for all design discharges.

Table 17: Dimensions initial design foundation and coffers spillway

	Length [m]	Width [m]	Height [m]
Foundation	10	6	0.3
Coffers	0.25	6	0.8

Steel Door

The required opening width of the door is determined previously. The thickness has been determined by using the maximum allowable moment; this calculation is shown in Appendix B.2 - Design Calculations Spillways. The resulting dimensions are found in Table 18.

Table 18: Dimensions initial design steel door spillway

	Length/thickness [m]	Width [m]	Height [m]
Steel door	0.01	1.8	0.80

Spillway

The concrete walls of the spillway will have a thickness of 0.25 meters, except the part above the steel door, which will be 0.15 meters. There will be walls starting from the beginning of the crest until the end of the dike slope. The dimensions vary per discharge capacity. The spillways have been divided into six parts, see Figure 26. The assumptions for each part are stated in Table 19.

Table 19: Dimensions initial design concrete parts spillway

Part	Length [m]	Width [m]	Height [m]
1	Crest of dike	0.25	Crest top to foundation
2	0.25	1	Crest top to foundation
3	0.25	2	Crest top to foundation
4	1	0.25	Following dike contour
5	Length left until end of dike	0.25	Linear slope until 0.5 m
6	0.15	Width opening	From opening until top of dike

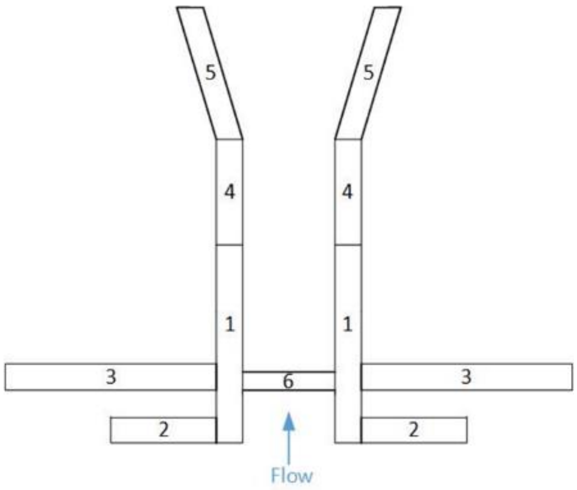

Figure 26: Numbering concrete parts spillway

10.3.7 Construction

The spillways should be constructed before the dike is placed. The construction will be elaborated in chapter 21 Implementation.

10.3.8 Operation

The spillways will be operated by the farmers in the area. They can determine how much they want their land to flood. The operation of the design will be elaborated in chapter 21 Implementation.

11 Cluster 2 Initial Design

Cluster 2 is the part south of the bifurcation point, see Figure 27. This bifurcation point splits into the Ronga and the Kikuletwa South Small. To control the floods of the Ronga the Kikuletwa South Small will be widened and deepened and a control structure will be placed in the New Kikuletwa South. First a description of the current situation of the Kikuletwa South area is given. Then the initial design of the New Kikuletwa South is presented, followed by the excavation of the New Kikuletwa South and at last, the initial design of the control structure is given.

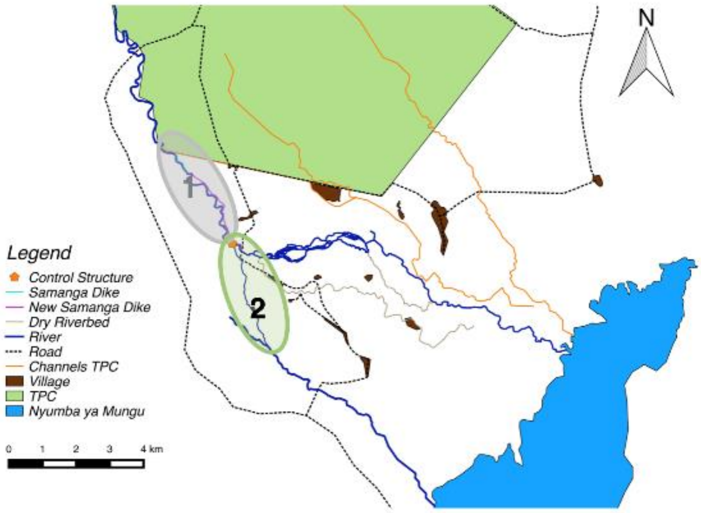

Figure 27: Location cluster 2

11.1 Current Situation

From the bifurcation, the Kikuletwa South Small flows to the south where it eventually enters the Kikuletwa South. During fieldwork, it became clear that at some point an old riverbed existed of the Kikuletwa South. Other points of interest were the connection of the irrigation canal and the connection of the Kikuletwa South Small to Kikuletwa South. This can all be seen in Figure 28.

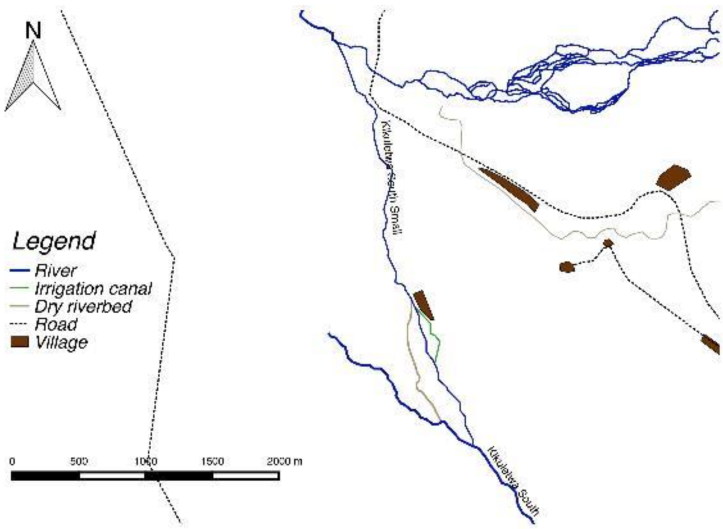

Figure 28: Current situation Kikuletwa South area

11.2 New Kikuletwa South

11.2.1 Location

The New Kikuletwa South will follow the first part of the Kikuletwa South Small until it reaches the dry riverbed. From here on the old riverbed will be used to discharge the water to the Kikuletwa South, see Figure 29. The dry riverbed will be used for three reasons. The first one is that the soil, where the Kikuletwa South Small and the irrigation canal meet again, is very hard. According to the local villagers, some kind of rock is present in the soil and therefore no deeper than half a meter can be excavated. The second reason is that along the second part of the Kikuletwa South Small some small villages are built close to the river. By using the old riverbed, those houses do not have to be moved. The third reason is that it is shorter than the current river.

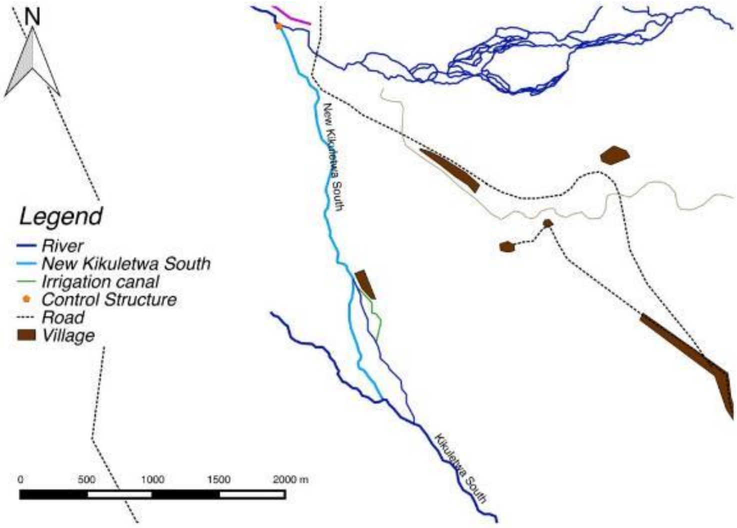

Figure 29: Initial design New Kikuletwa South

11.2.2 Design

The discharges for the New Kikuletwa South from chapter 8 Design Discharges are used to determine the dimensions of the New Kikuletwa South, see Table 20.

Table 20: Discharge New Kikuletwa

	Q (1/5)	Q (1/10)	Q (1/15)
Discharge [m^3/s]	28	34	36

Cross-section

It is assumed that a minimum discharge of 1 m^3/s is needed in the New Kikuletwa North to prevent sedimentation. In order to make sure that this discharge is not spread over the total width of the river, a deepened section in the middle of the river should be made. An example of the cross-section is shown in Figure 30.

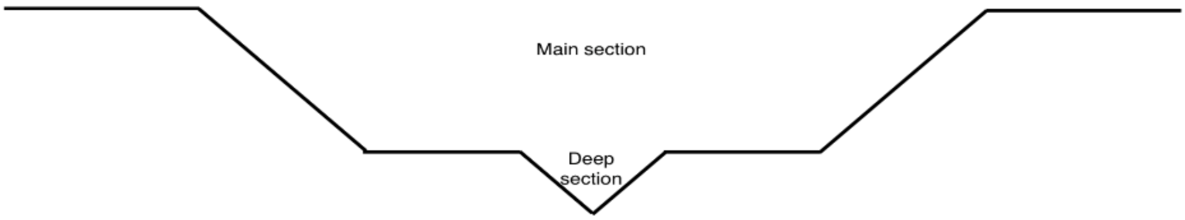

Figure 30: Cross-section New Kikuletwa South

Dimensions

The required height and width of the river are determined with Manning's equation. The calculations and the determination of the bed slope and roughness coefficient, n, can be found in Appendix B.3 - Design Calculations Kikuletwa South. The assumed values for the bed slope and roughness coefficient are given in Table 21.

Table 21: Bed slope and roughness coefficient New Kikuletwa South

	Q (1/5)	Q (1/10)	Q (1/15)
Bed slope [-]	0.0022	0.0021	0.0021
n deep section [$s/m^{1/3}$]	0.04	0.04	0.04
n deep + main section [$s/m^{1/3}$]	0.05	0.05	0.05

The dimensions for the deep section are the same for all the different design discharges. The maximum water level for each return period is 0.2 m below surface level and is given in Table 22. The resulting dimensions of the initial design of the New Kikuletwa South are presented in Figure 31.

Table 22: Maximum water level New Kikuletwa South

	Q (1/5)	Q (1/10)	Q (1/15)
Maximum water level [m]	3.1	3.5	3.6

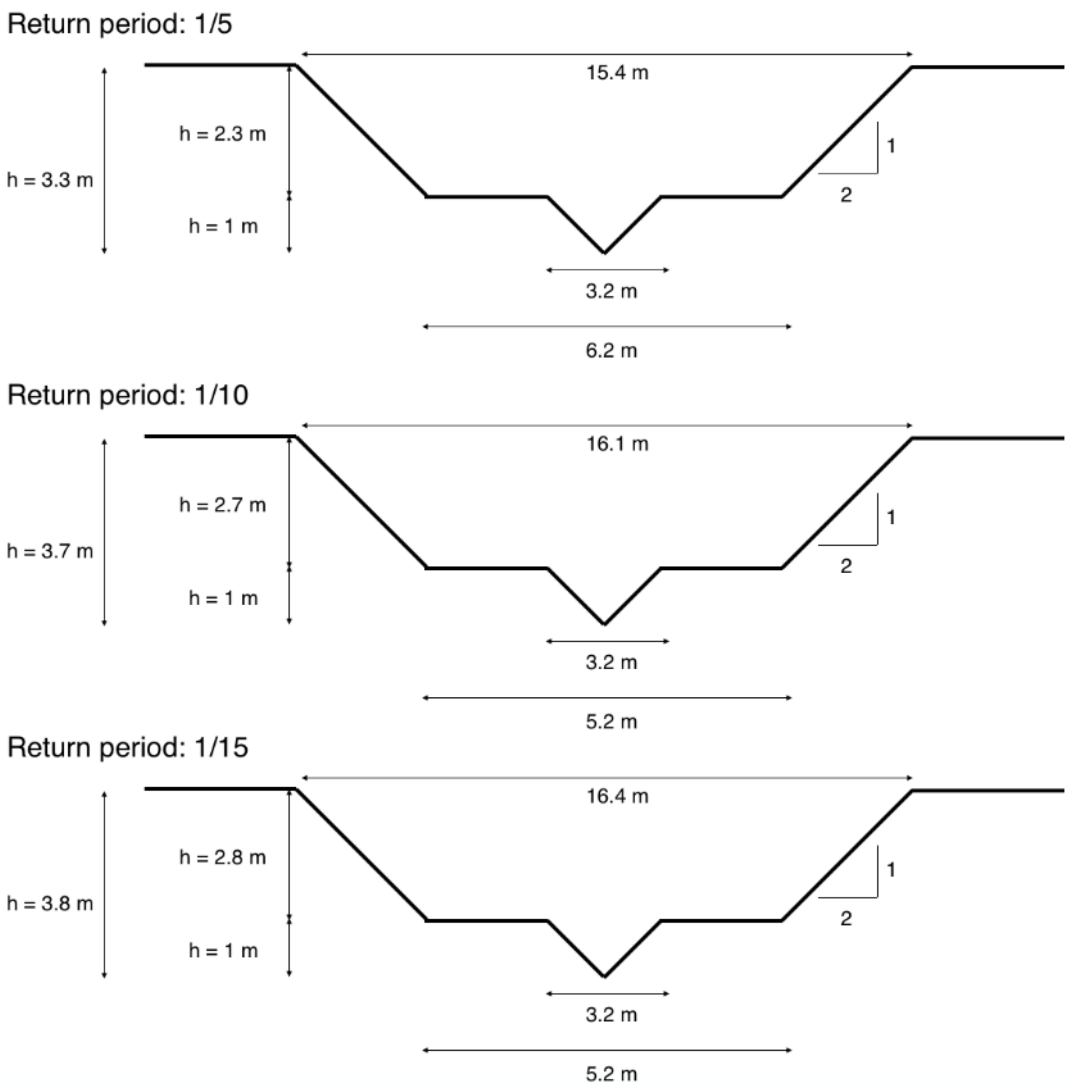

Figure 31: Dimensions initial design New Kikuletwa South

11.2.3 Excavation

The excavation of the New Kikuletwa South consists of two parts. The first part is where the Kikuletwa South Small is used. Along this part of the Kikuletwa South Small dikes are present on both sides of the river. The cross-sectional area and the length of the river and dikes are given in Table 23. The second part is the excavation of the old riverbed.

Table 23: Length and cross-sectional area of Kikuletwa South Small, dikes and old riverbed

	Kikuletwa South Small	Dikes	Old riverbed
Length [m]	2,200	2,200	1,000
Cross-sectional area [m^2]	6,537	6,600	-

The difference of the cross-sectional area of the Kikuletwa South Small and the dikes is negligible. Therefore, they are not taken into account for calculating the total excavation of the New Kikuletwa South.

The cross-sectional areas of the New Kikuletwa South are determined with the dimensions. These are multiplied with the total length of the new river, which results in the volume that has to be excavated. The excavation for each return period is presented in Table 24.

Table 24: Excavation of the New Kikuletwa South

	Q (1/5)	Q (1/10)	Q (1/15)
Length [m]	3,200	3,200	3,200
Cross-sectional area [m^2]	26	31	32
Volume [m^3]	84,671	98,051	101,951

11.2.4 Construction

Construction, or excavation, should start downstream at the connection with the Kikuletwa South. This will be explained further in chapter 21 Implementation.

11.3 Control Structure

In this section, the control structure at the bifurcation of the Kikuletwa and the Ronga is explained. Firstly, the discharge that will go through each branch after the bifurcation is explained. Hereafter a choice is made for the type of control structure. Then the initial design of the control structure will be explained and the dimensions of the structure will be presented. Lastly, the operation procedure of the control structure will be explained.

11.3.1 Discharge

An indication of the maximum design discharges at the bifurcation is given in Table 25. These are the discharges that should flow through the Kikuletwa South and the Ronga for the short and long rain periods after the implementation of the solution. The indication is based on the design discharges calculated in chapter 8 Design Discharges.

Table 25 Design Discharges Ronga and Kikuletwa South

	Return Period	Kikuletwa South	Ronga	Arriving at the Bifurcation
Discharge Long Rains [m^3/s]	1/5	28	43	71
	1/10	34	43	77
	1/15	36	43	79
Discharge Short Rains [m^3/s]	1/5	14	25	39
	1/10	18	25	43
	1/15	20	25	45

Long Rain Season

The discharge in the long rain season is highly variable, as explained in section 8.1.3 Variations of Discharges. The years with low peak discharges should be taken into account when designing, to ensure that the Ronga floods during these years. To determine the discharge that is normative in the described situation the long rain return period graph is used, see Appendix A.4 - Design Discharges. It is essential that the Ronga floods every year to ensure fertile lands. Therefore, a minimum peak in discharge during the long rain season of 45 m^3/s is used, this discharge occurs at least once every year. Most of this discharge is needed to flood the Ronga area. This is the discharge measured at the IDD1 station, but with the discharge the Kikuletwa North does not flood and therefore the same discharge arrives at the bifurcation.

Short Rain Season

During the short rains the Ronga should not flood. During peak discharges of short rains, the Ronga cannot handle the discharge arriving at the bifurcation. This excess discharge should go to the Kikuletwa South. However, when there is a low peak during the short rains there should be sufficient water flowing through the Ronga for irrigation.

Dry Season

In the dry season, there should be enough discharge to prevent that one of the riverbeds runs dry. This is the case assuming that there is a possibility for the water to flow into the river branches. A low water level in the Kikuletwa North should be taken into account.

11.3.2 Type of Structure

As previous explained the passive structures in the prefeasibility study had to be reconsidered, see Part B-Analysis. Consequently, a choice has been made to make a new design for the control structure at the bifurcation of the Kikuletwa South and the Ronga that meets the new requirements. Appendix B.4 – Design Drafts Control Structure gives a description of the possible design drafts for the control structure.

From the different possibilities in design drafts, it was concluded that it is not possible to design a passive control structure that fulfils all the requirements. Therefore, the choice has been made to make a manual controllable structure in the Kikuletwa South.

Location

The control structure is placed in the Kikuletwa South, see Figure 32. This follows from the design considerations in Appendix B.4 – Design Drafts Control Structure.

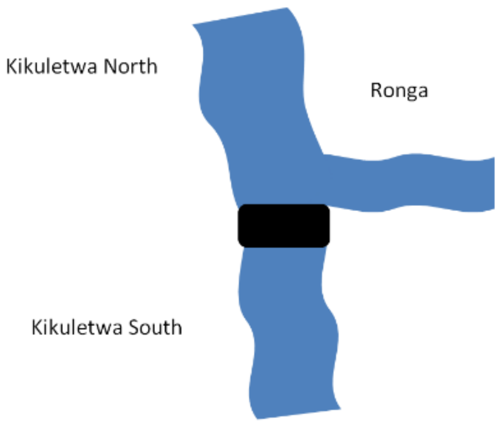

Figure 32: Location initial design control structure

Operation

The choice was made for a control structure with four openings with slots in which concrete beams can be placed. An impression of the structure is given in Figure 33. This figure shows the reference project “Regelwerk Hondsbroekse Pleij”. By removing or adding beams, the height of the structure can be adapted. The height of the structure determines the amount of water that can be discharged by the structure. Thus by adapting the amount of beams the discharge capacity of the control structure can be regulated. The choice for this kind of manually controllable structure means that the design of the structure should take the weight of the movable parts into account so they will not be too heavy to handle manually.

Figure 33 Impression reference project "Regelwerk Hondsbroekse Pleij" (Lokven, 2016)

Long Rain Season

The control structure should be opened during the long rain season to ensure sufficient, but not catastrophic, flooding of the Ronga area, while the excess water flows into the New Kikuletwa South.

Short Rain Season

During the short rain season, the structure can be opened when the water level in the Ronga is too high. After opening, the water will be discharged by the New Kikuletwa South so that the Ronga will not flood during the short rain season. An advantage of a manual controllable structure is that the control structure will not only function during the high design discharges from Table 25, but it can also be adapted in such a way that enough water is present in the Ronga during lower discharges.

Dry Season

During the dry season a certain minimum discharge of 1 m^3/s should go through the openings of the weir so that the Kikuletwa South will not dry up. An impression of the control structure and the way it is operated during different water levels can be found in Appendix E.4 - Visualisation Operation Phase.

11.3.3 Design

The initial design of the control structure is shown in Figure 34. There are four openings, in each opening three gates can be placed. The flow of water through the structure is indicated with the blue arrows. There will be two platforms in the middle of these openings, which can support cranes. These cranes should be manually operable. Since there will be excavation of the Kikuletwa South Small, there will be a bed level difference as the water flows through the bifurcation.

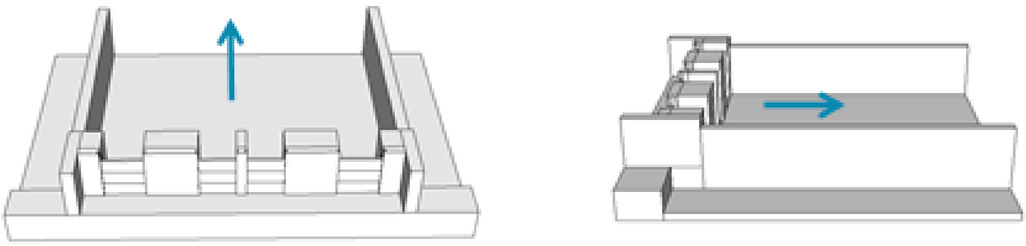

Figure 34: Initial design control structure

By performing unity checks on the possible failure mechanisms, it is possible to determine whether the assumed dimensions are feasible. This iterative process is repeated until the dimensions are such that the structure is safe for the considered failure mechanisms. Presented below are the dimensions for which all the unity checks of the failure mechanisms are sufficient. For the calculations of the failure mechanisms and the associated unity checks see Appendix B.6 - Failure Mechanisms Control Structure.

Dimensions

In Figure 35, a plan view is shown of the control structure. The most important elements are the foundation, the supporting structure and the gates.

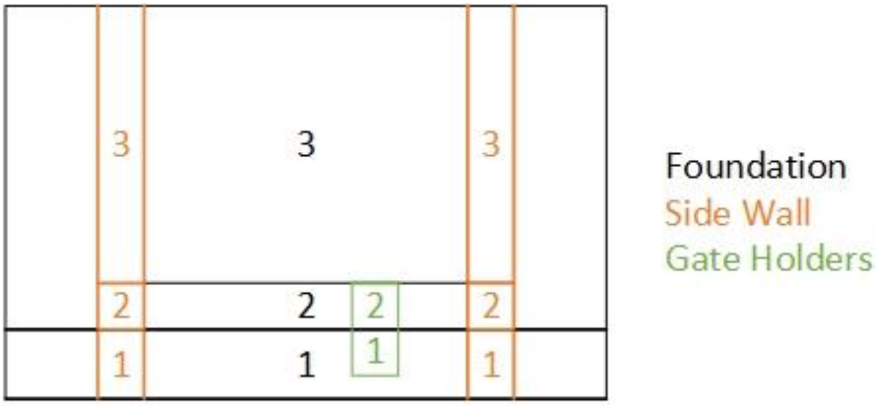

Figure 35: Top view control structure

Foundation

The foundation will be a slab with coffers on each side. A gravel layer should be present below the foundation. The dimensions can be found in Table 26. The foundation has the same length, outer width and inner channel width for each return period. The thickness differs per return period. For 1/10 and 1/15 years they are the same. The foundation on the Kikuletwa North side will be thicker than the foundation on the Kikuletwa South side because the foundation will be placed on one level and the bottom level of the river in the New Kikuletwa South is lower.

Table 26: Dimensions initial design foundation

	Length [m]	Width [m]	Height [m]
Foundation	18	22	Varies
Coffers	0.25	22	1.5

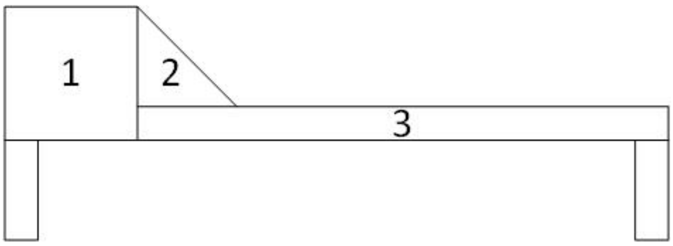

Figure 36: Dimensions foundation parts, side view

The foundation has been separated into three parts, as can be seen in Figure 36. The foundation of the first part is thicker. The second part only stretches along the inner channel (it is placed on top of the third part). The third part is the largest part, located on the Kikuletwa South side. The dimensions of these parts are stated in the Table 27. For some variables, two values are presented. This first value is for a return period of 1/5 years and the second for both 1/10 and 1/15 years.

Table 27: Dimensions initial design foundation parts

	Length [m]	Width [m]	Height [m]
Part 1	2	22	1.5/1.9
Part 2	1/1.4	15	1/1.4
Part 3	16	22	0.5

Supporting Structure

The supporting structure consists of the gate holders and the sidewalls.

Side Walls

The width of the inner channel is 15 meters for all discharges. Along this inner channel, two walls are present. The width of these walls is 0.6 meters. The sidewalls have also been separated into three parts; see Figure 37 and Table 28.

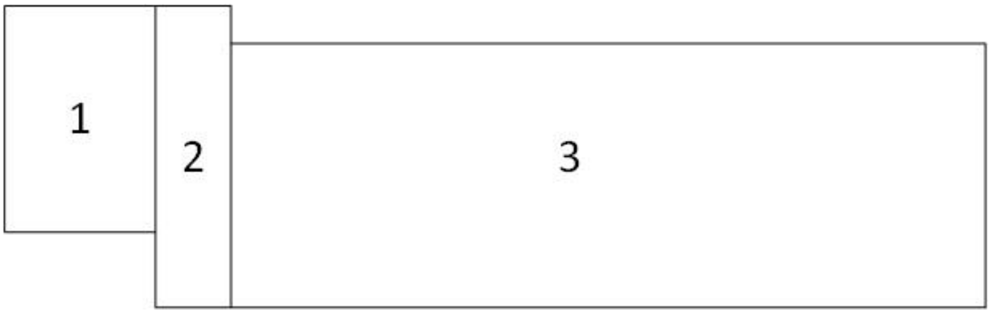

Figure 37: Dimensions sidewalls, side view

Table 28: Dimensions initial design sidewall

	Length [m]	Width [m]	Height [m]
Part 1	2	0.6	2.8/3
Part 2	1/1.4	0.6	3.8/4.4
Part 3	15/14.6	0.6	3.4/3.8

Gate Holders

The gate holders support the gates. The gates should be able to slide into the holders. This means the width of the openings is smaller than the actual width of the gates. The gate holders stretch over the sill in the foundation and for 0.5 meters on the Kikuletwa North side. The total length of the openings and the holders is 15 meters (the inner channel). The total width of the gate openings should amount 7.5 meters, see

Appendix B.5 - Total Gate Width Control Structure.

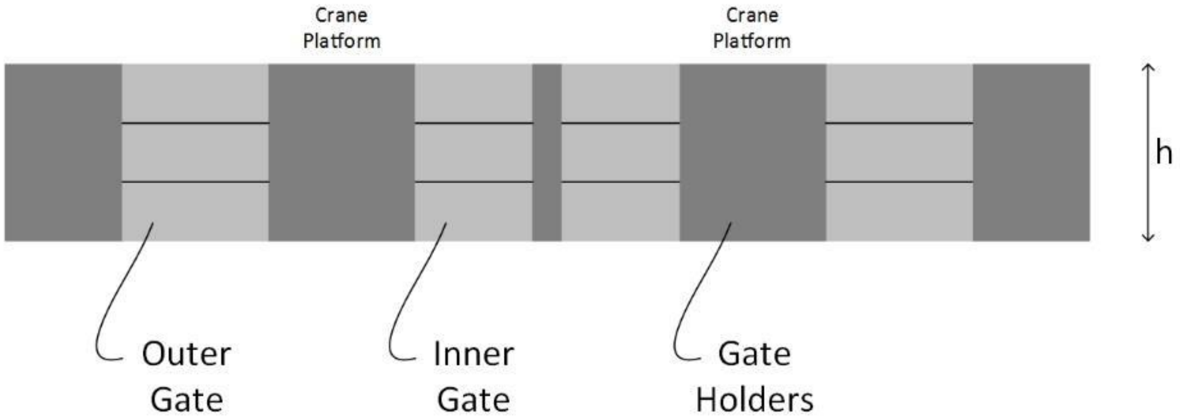

Figure 38: Gates and gate holders' front view

In Figure 38, the gates in the closed situation are visualised. A cross-section of a gate holder can be seen in Figure 39. There are five gate holders in total, only one is visualised in Figure 35. The gate holder is divided in two parts. It has been designed to fit exactly over the sill. In Table 29 the dimensions of part one are given and in Table 30 the dimensions of part two.

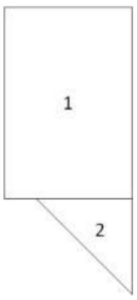

Figure 39: Gate holder side view

Table 29: Dimensions initial design gate holders part 1

	Amount [#]	Length [m]	Width [m]	Height [m]
Outer holders	2	1.5/1.9	1	2.8/3
Crane holders	2	1.5/1.9	2.5	2.8/3
Middle holder	1	1.5/1.9	0.5	2.8/3

Table 30: Dimensions initial design gate holders part 2

	Amount [#]	Length [m]	Width [m]	Height [m]
Outer holders	2	1/1.4	1	1/1.4
Crane holders	2	1/1.4	2.5	1/1.4
Middle holder	1	1/1.4	0.5	1/1.4

Gates

The gates will be made from concrete. This material has been chosen, as it is more resistant against erosion from water than steel. The chosen dimensions of the gates are stated below; the total width of the gates should amount to 7.5 meters. These dimensions have been chosen so that a crane can lift them. The limit that the crane can handle is 1000 kilograms (Workstation Lifting Products: Manual Products, 2016). The outer gates are wider than the inner gates so that the total needed width is reached as described in

Appendix B.5 - Total Gate Width Control Structure. In each slot, three gates can be placed to reach a total height of 2.4 meters. The total amount of gates is therefore twelve. There will be six inner gates and six outer gates. These dimensions are equal for all return periods.

Table 31: Dimensions initial design gates

	Amount [#]	Length [m]	Width [m]	Height [m]
Outer Gates	6	0.15	2.2	0.8
Inner gates	6	0.15	1.95	0.8

Scour Protection
Scour protection has not been included at this stage.

11.3.4 Construction
During construction, a building pit needs to be made. This should be a certain distance from the river so that no water enters. If it does, it should be pumped out. The river will be diverted to the control structure after it has been built. This requires the New Kikuletwa South to be excavated already. The construction method will be explained further in chapter 21 Implementation.

11.3.5 Operation
The gates will need to be altered depending on the season and the peak during that season. A warning system, from for instance, IDD1 would be a good solution. The gates should be operated manually because no power is available. The cranes to lift the gates should be stationary. This is because during severe weather conditions cranes cannot be transported to the location. The operation of the structure will be explained further in chapter 21 Implementation.

12 Cost-Benefit Initial Design

In this chapter, the monetary benefits and cost associated with the initial designs for the different return periods will be presented. The values given are the present values for the time horizon of 25 years and presented in millions of US dollars. In Appendix C.1 - Determining the Present Value it is explained what discount rate and inflation were used for the calculations. The benefits will be discussed first, followed by the different costs and lastly both will be compared for the different return periods.

12.1 Monetary Benefits

For the different return periods, the associated benefits will be presented in this section. In Appendix C.2 - Monetary Benefits of the Designs it is explained how the different benefits were determined. There are two main categories of the benefits, the agricultural and the social.

12.1.1 Agricultural Benefits

The agricultural benefits are preventing the current loss of crops because of the flooding and increasing the utilisation of the farmland. The loss of crop occurs at this moment when the farmers take the risk to work their fields even though the rain season approaches. The majority of the crop loss happens during the short rain season.

The increase of utilisation of the farmland entails farmers starting to use their lands when normally they would not due to the rain seasons. When their farmlands are protected against sudden floods, they will gain the option to use their land with little risk.

12.1.2 Social Benefits

The social benefits are preventing damage to assets, for example goods of entrepreneurs, schools and clinics. Furthermore, due to the floods large areas become inaccessible, preventing people from going to their work and entrepreneurs missing out on income.

12.1.3 Total Benefits

In Table 32, the results of the calculation of the present value benefits of the different return periods can be seen. For the short rain season, the benefits are the same for all the solutions, as they are designed to be able to handle all short rain discharges. The only difference is for the long rain season; this is caused by the different return periods of the floods.

Table 32: Present value benefits for different return periods

Present Value Benefits	**Short rain season**	**Long rain season**		
	1/5 - 1/10 - 1/15	1/5	1/10	1/15
Agriculture				
Prevention loss of crops [mln. $]	5.2	1.5	1.7	1.8
Increased harvest [mln. $]	1.3	2.0	2.3	2.3
Social				
Damage of assets [mln. $]	0	0.1	0.1	0.1
Salary/income [mln. $]	0	0.2	0.2	0.2
Total	6.5	3.8	4.3	4.4

What has not been taken into account for this calculation is the sense of safety/confidence the local farmers could get because of the designs. This means that the safer they feel, the more likely they are to invest in their farmland and harvest more. It can be expected that there are differences between the designs. However, in the calculations the same assumptions are used for the growth of the farmland for all return periods. With other words, it is assumed that for all return periods, the growth of the harvests will be the same. The reason for this is that it is hard to predict the future, taking this into account would only increase the number of assumptions, without knowing if it will come close to the reality.

12.2 Costs

There are different kinds of costs associated with the realization of the designs and the lifetime expenses like maintenance and operation. These costs will be presented in this section and a comparison will be made between them. First, the expected cost for the construction work will be presented, after which the total realization cost will be discussed. Lastly, the total estimated cost will be presented.

12.2.1 Construction Costs

The construction costs are the fees that have to be paid to the contractor to do the work. These are based on the tender prices received during meetings with contractors, the quantities determined for the designs and assumptions made for the building method. In Appendix C.5 - Cost Determination Initial Design Phase the quantities and prices used can be found.

Table 33: Division of cost between major works

Construction cost	1/5	1/10	1/15
Cluster 1			
Preparing area [%]	47.8	49.2	50.4
Samanga Dike [%]	11.0	11.6	12.0
Spillways [%]	2.8	2.5	2.4
Cluster 2			
Preparing area [%]	19.9	18.3	17.4
Control structure [%]	7.1	6.7	6.3
Kikuletwa South [%]	11.4	11.7	11.5

In Table 33, the division of costs between the major works is presented. What can be seen is that the preparation of the area (clearing of vegetation) forms roughly 65-70% of the total cost for all three return periods. This is deemed unrealistic and will be re-evaluated for the final cost estimation.

12.2.2 Realisation Costs

There are several costs associated with the process of bringing the designs presented in this report to full realization. This will be explained briefly, in Appendix C.5 - Cost Determination Initial Design Phase a more extensive explanation can be found.

In Table 34 the cost for the realization of the designs can be seen. These prices are the current prices, assuming construction will start within a year from now.

Table 34: Realization cost for the different return periods

Cost	Return periods		
	1/5	1/10	1/15
Design [mln. $]	0.2	0.2	0.2
Exp. Construction [mln. $]	3.1	3.5	3.7
Project Management [mln. $]	0.4	0.4	0.4
Contingency [mln. $]	0.9	1	1.1

The different types of realization costs are:

- Design, this is for making the detailed design and the bill of quantities, it is assumed to be 6% of the estimated construction cost.
- The estimated construction cost, this is the price based on the tender prices received and the quantities determined for the designs. This is the calculated construction price.
- Project management, this is for the management of the project for the client. This includes the salary for a project leader, assistants, surveyors and 10% of estimated construction cost for overhead expenses.
- Contingency, this is for taking into account the risks during the construction phase. It has been set at 30% of the estimated construction costs.

All the amounts are determined on basis of limited data. Furthermore, there are still many uncertainties with relation to the design. Of most river parts only one profile is known, also there is no detailed height map available. As such, the realization cost cannot be presented as a single, certain value, but more as a range between which the costs likely are.

The range has been determined based on the AACE International class system for projects. For the level of detail and the end usage (feasibility study), the estimated class is 4. This correlates with a lower bound of -15% to -30% and an upper bound of +20% to + 50% (AACE International, 2015). The calculations for the structures exceed the feasibility level, however taking into consideration that there are still many unknowns, a range of -15% to +40% of the calculated cost is chosen. In Table 35, the range for the realization cost for the different return periods is presented.

Table 35: Total realization cost initial design

Cost	Return periods		
	1/5	1/10	1/15
Calculated [mln. $]	4.6	5.2	5.5
Lower bound 85% [mln. $]	3.9	4.4	4.7
Upper bound 140% [mln. $]	6.4	7.2	7.7

12.2.3 Total Lifetime Costs

Besides the realization cost (initial cost), there are also expenses during the lifetime of the structures. These relate to maintenance, operation and repairs due to exceeding. In the initial design, the operation costs have not been taken into account because they are the same for the different return periods.

Table 36: Total Cost for Lifetime of the structures

Present Value Cost	1/5	1/10	1/15
Realization [mln. $]	4.6	5.2	5.5
Maintenance [mln. $]	0.3	0.3	0.3
Repairs due exceeding [mln. $]	0.2	0.1	0.1
Total [mln. $]	5.0	5.6	5.8

In Table 36, the present value of the cost for the lifetime of the structures can be seen. There are three cost categories:

- Realization costs; these have been discussed in the previous paragraph, which relate to the process of building the structures.
- Maintenance costs; this is the cost associated with maintaining the different structures. This has been estimated at 0.5% of the construction cost.
- Repairs due exceeding; this are the cost for the required repairs when the design discharge is exceeded.

12.3 BCR, NPV, Payback Period

The present values of the calculated benefits and cost determined in the previous sections will be compared to get an estimation of profitability of the project. Based on the made assumptions it can be seen in Table 37 that the return period of 1/5 years has a better ratio between the benefits and the cost. For the benefit cost ratio, all scores above 1 indicating that the benefits are more than the cost.

Table 37: Benefit cost comparison

Results	1/5	1/10	1/15
Benefit cost ratio	2.03	1.92	1.85
NPV [mln. $]	5.3	5.2	5.1
Payback Period [years]	9	9	9

The net present value (benefits – cost) for all three return periods are positive with only minor differences. The payback period in years is the same. However, 1/5 year solution is paid back earlier in that year than the 1/10 and 1/15 solutions.

Overall, there are no significant differences between the different return periods. However, as explained before the sense of security/confidence based on the chance of flooding has not been taking into account. This means that it is possible that in reality the benefits for the higher return periods increase faster over the time than the lower return period.

13 Conclusions Initial Design

In the analysis, the conclusion was drawn that the passive structures had to be reconsidered. Different alternatives were studied in the initial design; manually controllable and passive structures and combinations of the two. It was concluded that the only option was to design manually controllable structures for both the spillways and the control structure. The design therefore consists of:

Cluster 1:

- A dike along the Kikuletwa North including manually controllable spillways.

Cluster 2:

- Widening the narrow part of the Kikuletwa South.
- A manually controllable control structure in the Kikuletwa South, just after the bifurcation.

The design was worked out for the three different return periods; 1/5, 1/10 and 1/15 years. During a progress meeting, on the 15^{th} of December 2015, with the stakeholders of this project, the three different solutions from this chapter were presented. The aim of the meeting was to choose one design that would be worked out further. The positive and negative aspects of each solution were weighed against each other.

All of the proposed solutions would protect against the highest peaks of the short rain floods. Regarding the safety level, they are all equal for these floods. The designs are based on the return periods of the long rains discharges. Consequently, the 1/15 year solution is considered the safest option in terms of long rain floods. The 1/5 year solution is the least safe option. The costs associated to the designs are higher for the 1/15 year solution than for the 1/5 year solution. This follows from the dimensions of the 1/15 year solution, which are larger. The cost benefit ratio is highest however for the 1/5 year solution, and lowest for the 1/15 year solution.

Considering all these things, the decision was made to continue designing for a return period of **1/15 years**. The arguments to support this decision were that the costs were not significantly higher and it would be easier to find donors for a safer solution. A second argument was that, although the benefit cost ratio was lowest for the 1/15 year solution, it did not differ much from the other solutions.

Furthermore, the clearing costs are very high. The stakeholders expect these costs to decrease in the next design stage and therefore are confident that the costs of the 1/15 solution will decrease as well. However, for the next design stage it is necessary to carefully look at these costs in order to try to reduce them.

Part D - Integral Design

In this chapter, the integral design is presented for 1/15 solution as has been decided in the preceding design stage. Essential information that is necessary for a clear picture is repeated from the initial design. Other information that is not found to be crucial is not included or only described briefly. For detailed descriptions on the reasoning behind the designs decisions, a referral is made to the initial design.

In cluster 1, the integral design of the Samanga dike and the spillways is described. The location, dimensions and changes made to the failure mechanisms of the dike are stated. Following this, the integral design of the spillways is presented. This includes the location and the dimensions of the spillways.

In cluster 2, the changes made compared to the initial design of the New Kikuletwa South and the control structure at the bifurcation are presented. Firstly, the integral design of the New Kikuletwa South is described. This includes the location, the new cross-section and its dimensions, a stability check and the total excavation. Hereafter the integral design of the control structure is described. The location of the control structure with respect to the reference level is given, scour protection is included and the workings of the crane construction are explained. Hereafter, the morphological effects of the total solution are investigated.

In the cost benefit analysis, the costs resulting from the dimensions of the integral designs are calculated. In order to improve the cost estimation, the different parameters have been revaluated and the clearing cost for the vegetation has been revised. A sensitivity analysis of the costs is conducted hereafter. Finally, the integral design is validated in the same manner as the prefeasibility study.

Figure 40: Picture taken during fieldwork, short rain flood at Samanga dike

14 Cluster 1 Integral Design

Cluster 1 consists of the Samanga area and its corresponding dike, spillways and floodplains as shown in Figure 41. In this section, firstly, changes and additions to the initial design of the Samanga dike are described and the integral dike design is shown. Secondly, elaborated drawings of the integral design of the spillways are presented as well as scour protection.

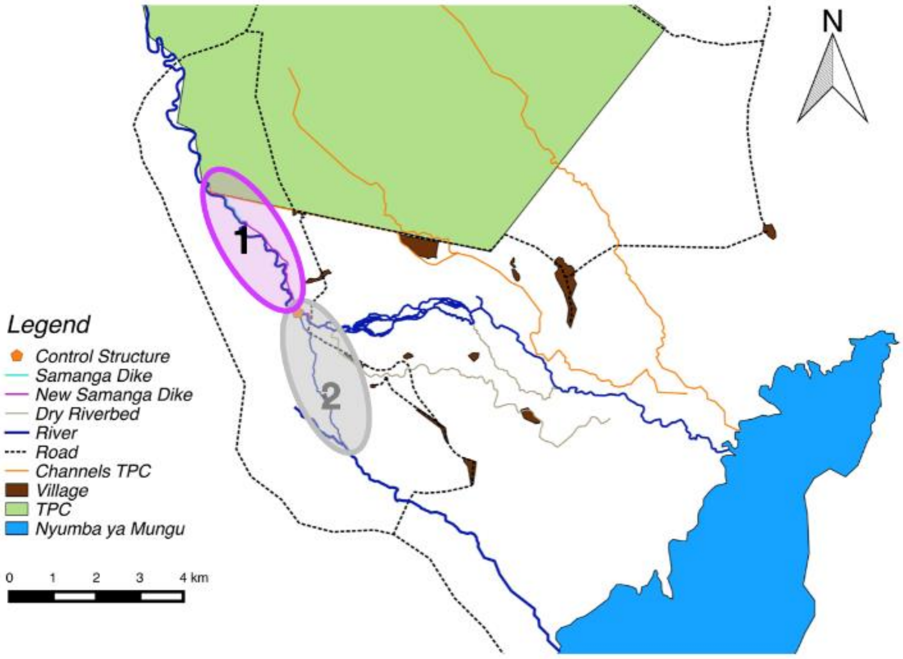

Figure 41: Location cluster 1

14.1 Samanga Dike

In this design stage, more attention is paid to the details regarding the location and the manner in which it will be constructed.

14.1.1 Location

The exact location of the dike is shown in Figure 42. The dike is situated a certain distance from the riverbanks, the biggest reason for this is because this area is heavily vegetated and it would be very expensive to clear. The distance between the dike and the river should be at least 10 meters. This buffer is taken due to erosion at the bends of the river as well as possible logjams causing erosion. The total length of the dike is 4450 meter. It should be connected directly to the TPC dike.

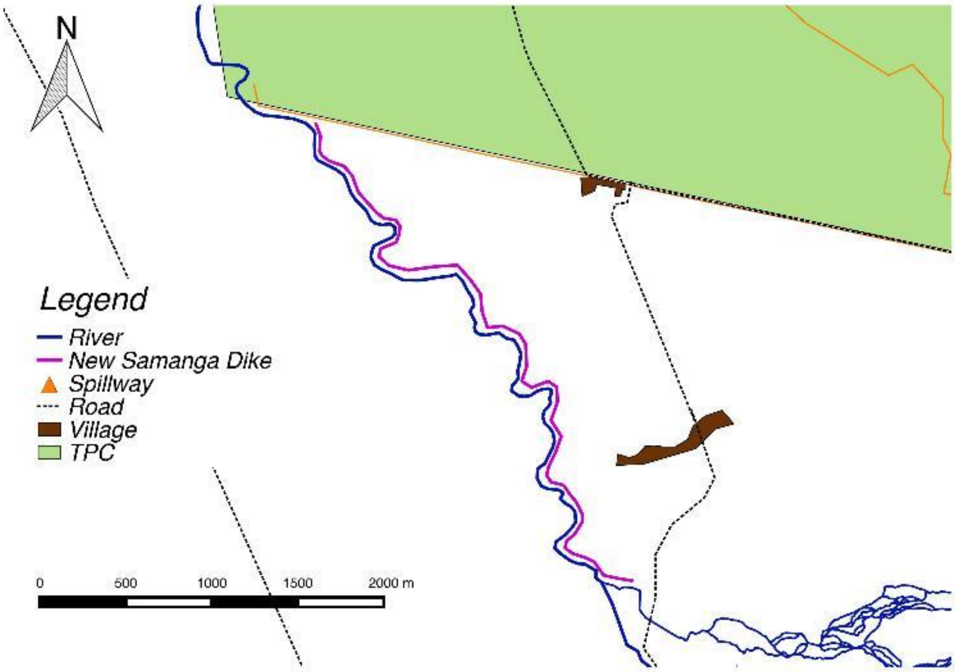

Figure 42: Location Samanga dike

14.1.2 Dimensions

The dimensions of the dike have not changed. The values can be read in Table 38. The dimensions in Figure 43 have been rounded up.

Table 38: Dimensions Samanga dike

	Q (1/15)
Design water level [m]	0.79
Freeboard [m]	0.65
Design height [m]	1.45
Slope [-]	1V3.5H
Horizontal length slope [m]	5.08
Dike crest [m]	4.5
Total width dike [m]	14.65

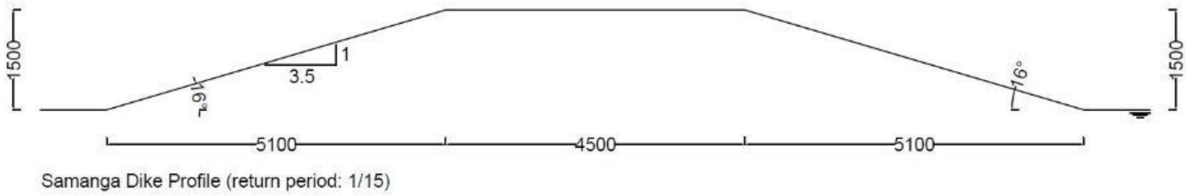

Figure 43: Integral design dike

14.1.3 Failure Mechanisms

Assumptions

The following assumptions have changed or been added with regard to the initial design. These changes in assumptions affect the failure mechanisms.

Soil Properties

The strength parameters of the soil have slightly changed after receiving the soil results from TanRoads. An overview of the soil results can be found in Appendix D.1 - Soils. The weight of soil above phreatic level (wet) is 17 kN/m^3. The weight of soil below phreatic level (fully saturated) is 20 kN/m^3. The cohesion and friction angle remain 15 kPa and 33° respectively.

Digging Depth

The area on the east side of the dike needs to be dug out 0.75 meters for a length of 20 meters. This is including the compaction factor determined in Appendix D.1 - Soils. The soil that is dug out will be used to construct the dike, in a similar manner as which the current dike has been constructed. In the initial design, this area was not included.

The dimensions of the dike have not been changed with regard to the initial design. The soil properties and the digging depth have been altered in this design stage and therefore the mass instability of the dike needs to be revaluated. The other failure mechanisms are not affected by the changes in assumptions.

Dike Instability

The different scenarios that are explained in Appendix D.1 - Soils were run through the D-Geo Stability program for the integral dike design. Apart from the unit weight and the change in digging depth, the same assumptions for each scenario are taken as for the initial design. An impression of the long rain flood scenario is found in Figure 44. The resulting safety factors for each scenario are found in Table 39.

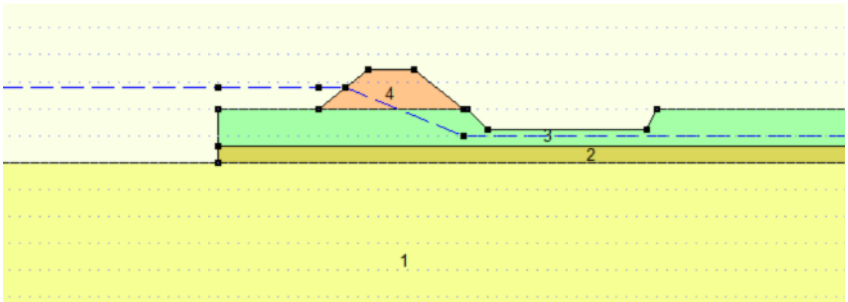

Figure 44: Dike profile design during long rain flood

Table 39: Factors of Safety D-Geo

Loading scenario	Factor of Safety
Short rain	2.2
Long rain	2.5
Post flood	1.9
Dry season	2.5

The dike remains safe against mass instability. Therefore, using this approach the construction of the dike is feasible.

14.1.4 Additional Consequences

The dike will be constructed with the soil on the east side of the dike. As can be seen in Figure 44, this creates quite a large gap. During the long rain season the spillways will be opened to flood the Samanga area, however this water will be trapped by this gap and it will act as a river of its own. Therefore, the following measure has been thought of to prevent this from happening. After the outlet of each spillway, small dikes will be constructed in the created gap. The guiding dikes will be constructed on both sides of the spillway, directing the flow into the Samanga area. A 3D impression of this solution can be seen in Figure 45. The dikes will prevent flow in the created river and direct the water into the Samanga area. An elaboration of these guiding dikes can be found in Appendix D.2 - Elaboration on the Guiding Dikes.

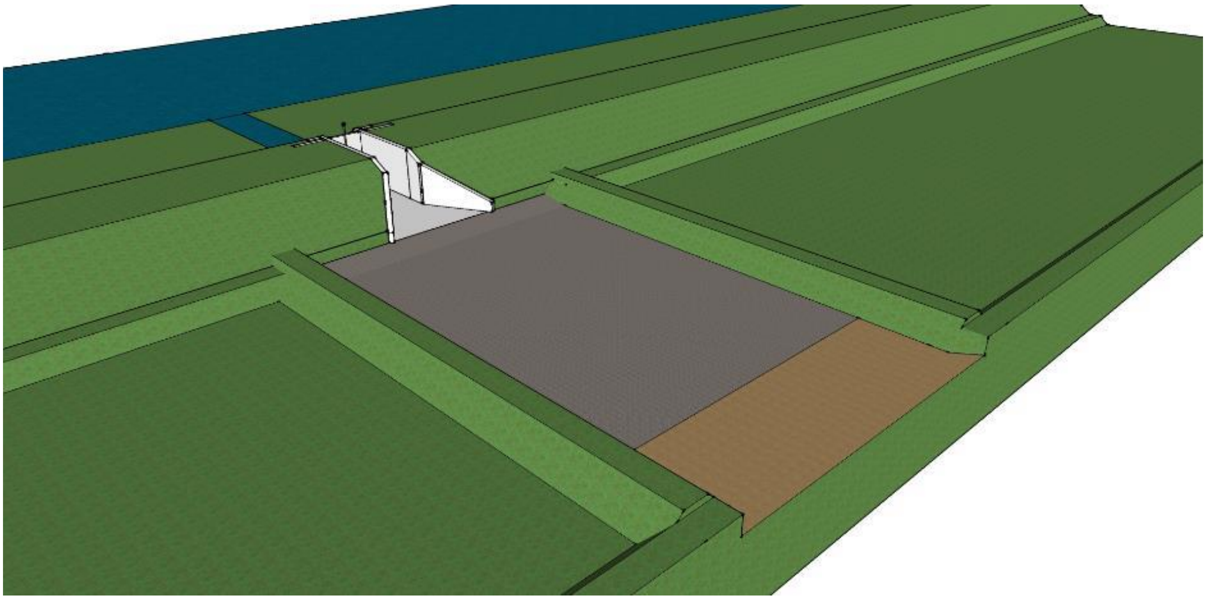

Figure 45: Plan view guiding dikes

14.2 Spillways Samanga Dike

The integral design of the spillways is the same as in the initial design. In this section, the most important aspects of the design are repeated. The failure mechanisms are not repeated, as they remain the same. A more elaborate design is presented including front, side and top views and their corresponding dimensions.

14.2.1 Location

The locations (orange triangles) in Figure 46 have been identified as suitable locations for spillways, see chapter 10 Cluster 1 for the elaboration of the choice for these locations. Five new spillways are necessary to flood the Samanga land during long rain floods.

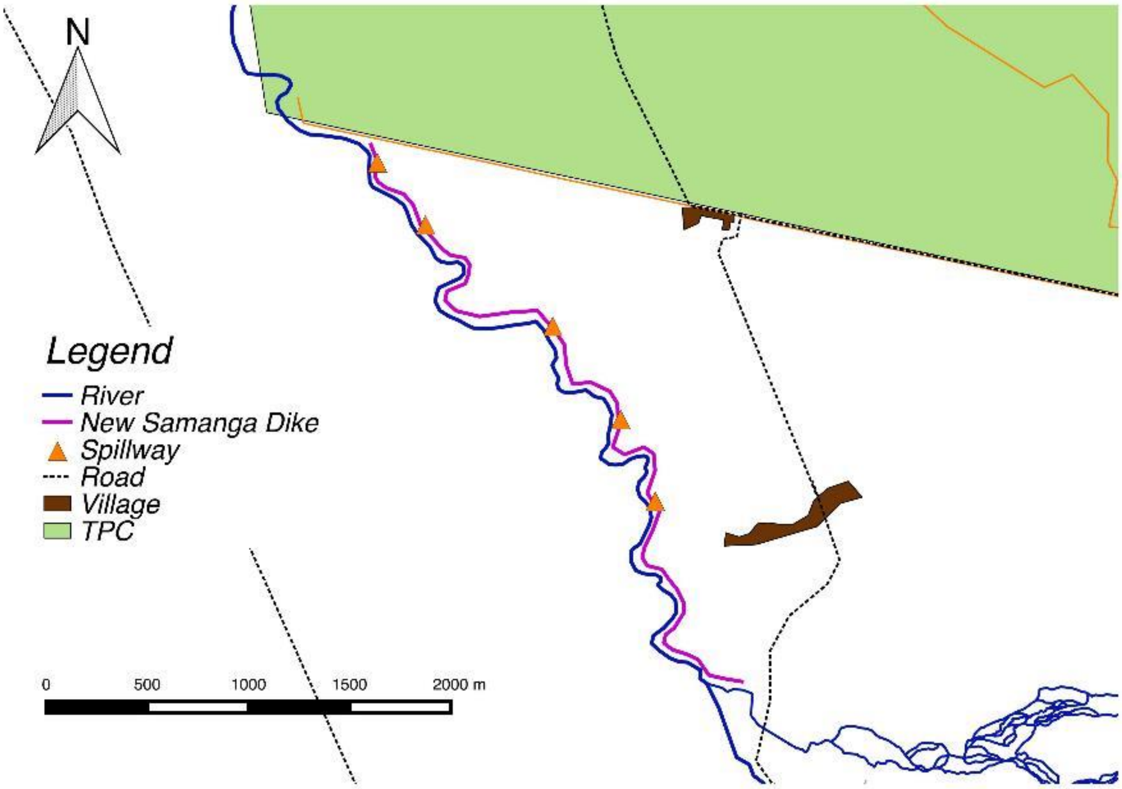

Figure 46: Location spillways

14.2.2 Design

The spillways are designed as manually operated vertical gates under submerged underflow; see Figure 47 for a 3D impression.

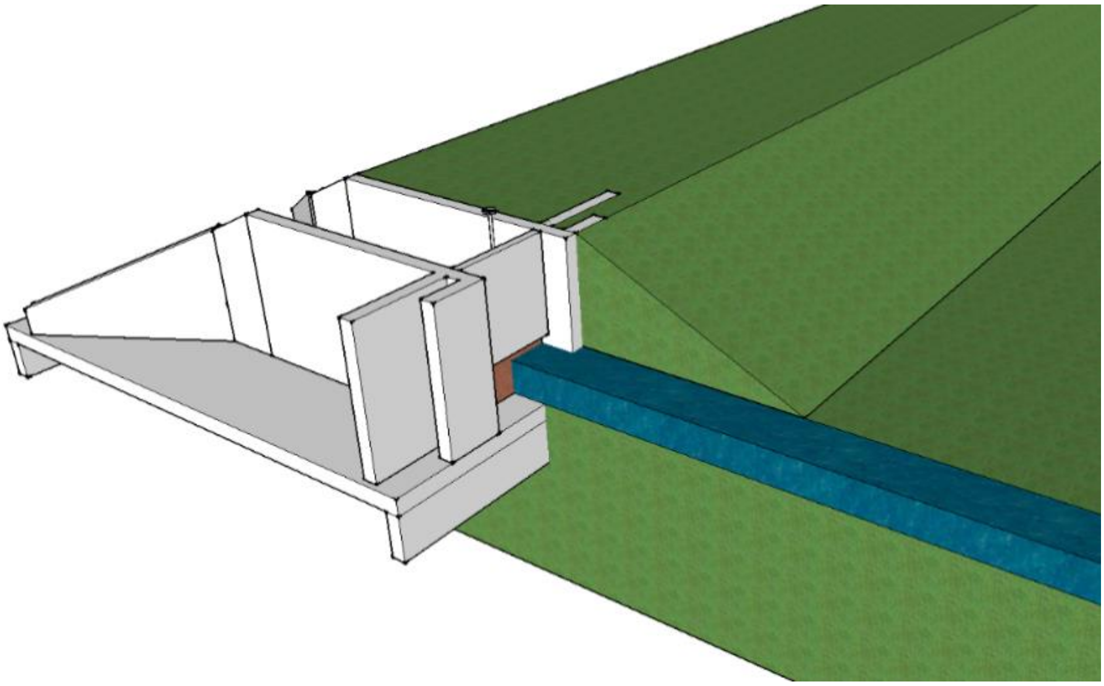

Figure 47: Impression spillway design

Dimensions

The final dimensions of the spillways are shown in the figures below. They are the front, side and top views. The reasoning behind these dimensions can be seen in the initial design. These dimensions should not be considered as detailed construction drawings.

Front View

In Figure 48, the front view of the spillway is shown.

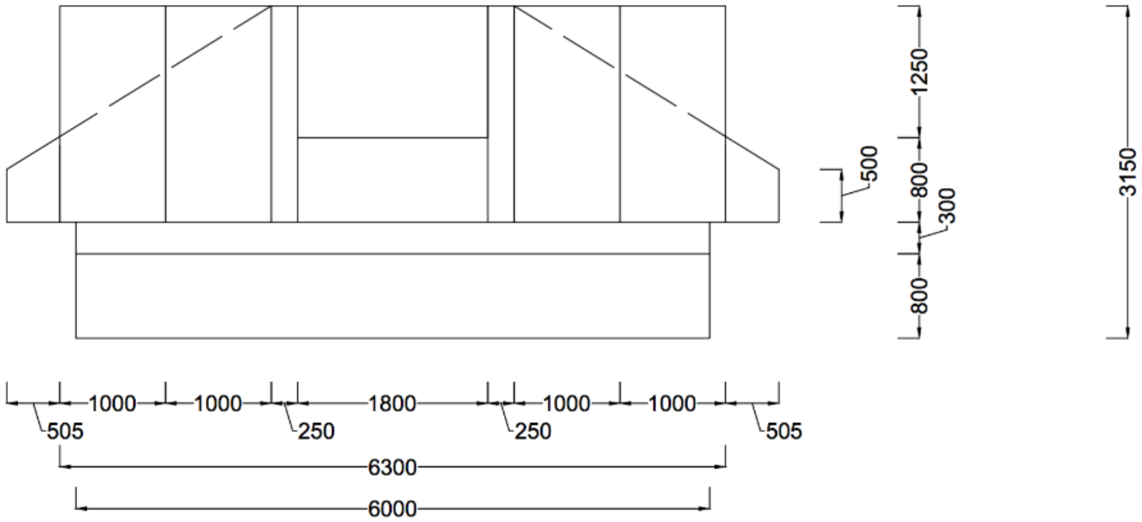

Figure 48: Front view spillway

Side View

In Figure 49, the side view of the spillway is shown

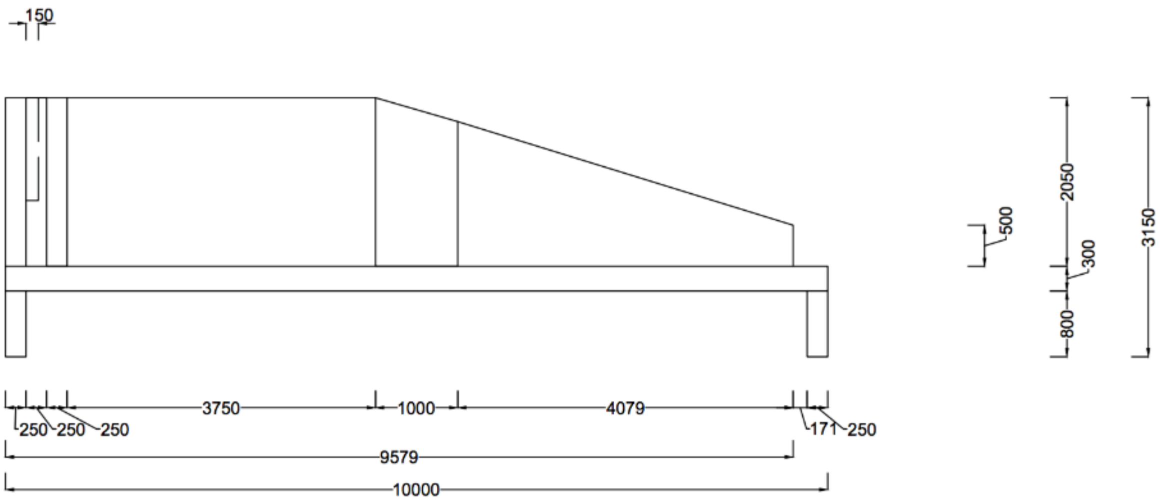

Figure 49: Side view spillway

Top View

In Figure 50, the top view of the spillway is shown.

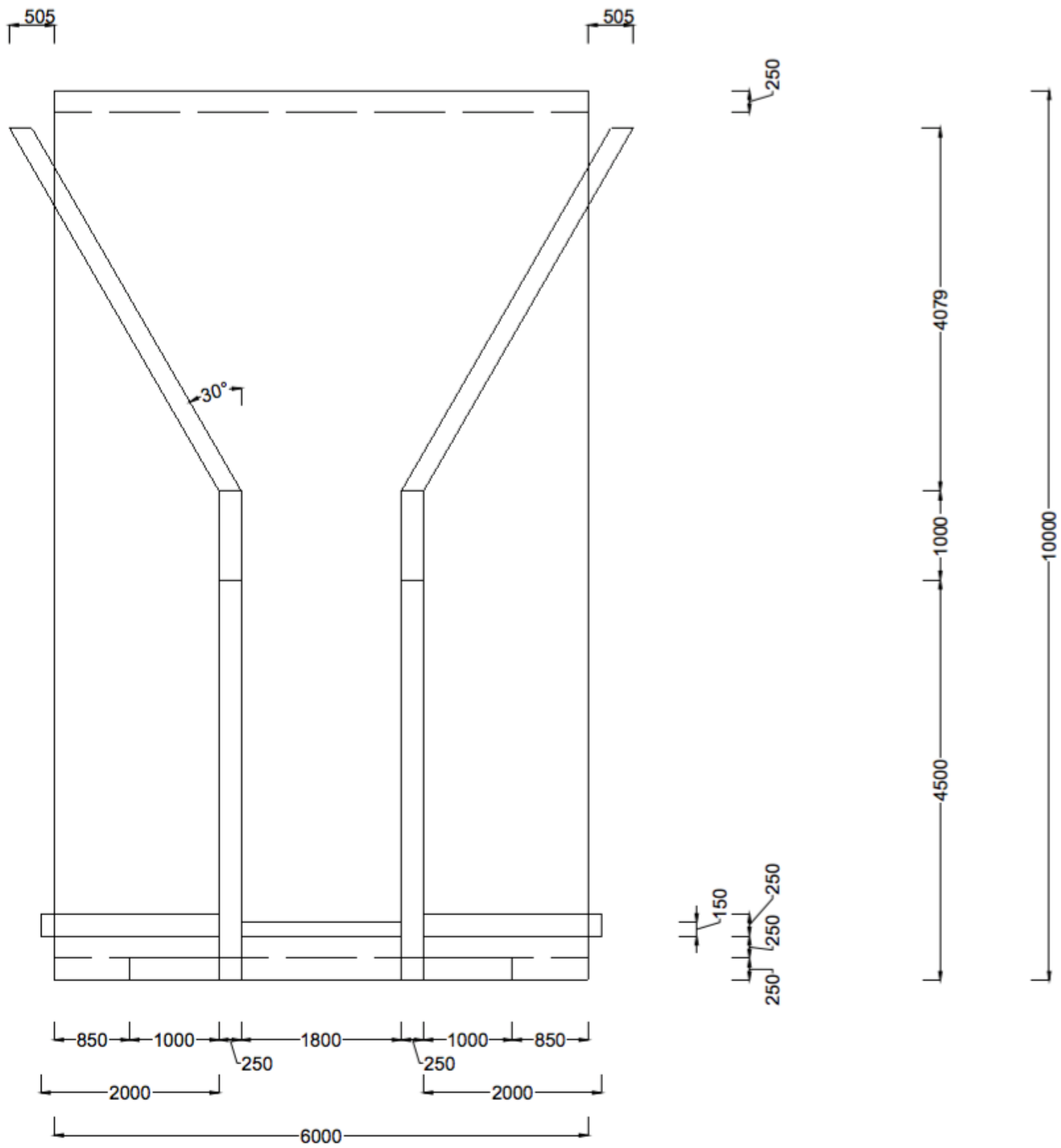

Figure 50: Top view spillway

14.2.3 Scour Protection

A summary of the scour protection at the spillway is provided here, for a more detailed description the reader is referred to Appendix D.3 - Scour Protection. The appendix mainly focuses on bed protection against bed erosion. Other types of scour are not worked out in detail. Bed protection is needed to prevent scour holes close to the spillways. Two different approaches can be distinguished: soft protection and hard protection. With soft protection, eroded soil will be replenished after a flood, treating the consequences of the scour. Hard protection moves the problem of scour to a location where it no longer relevant.

At the front of a spillway soft protection will be used, restoring any erosion after a flood will prevent damage to the structure in the long-term. After an average flood, the soil that needs to be replenished is estimated at 2 m^3 per spillway. Hard protection for the front was considered, but the risk that the river washes the protection material away during a flood is too high.

At the back of a spillway hard protection is necessary, because soft protection will not solve all the scour problems that occur during a flood. The first layer of the hard protection will consist of 15 cm of sand, which is placed directly on the current soil. This functions as a filter layer that ensures that the native soil, consisting mainly of silt will not erode through the coarse top layer. The top layer will consist of a 10 cm thick gravel layer, which is designed not to erode during the flood discharge. The exact grading of the layers and the minimal median nominal diameter can be found in the appendix. The length of the protection was determined using the maximum scour depth. This resulted in a minimum length of 17 m. For the width, the total distance between the guiding dikes is taken, which is 12.5 m. A plan view of the bed protection can be seen in Figure 51.

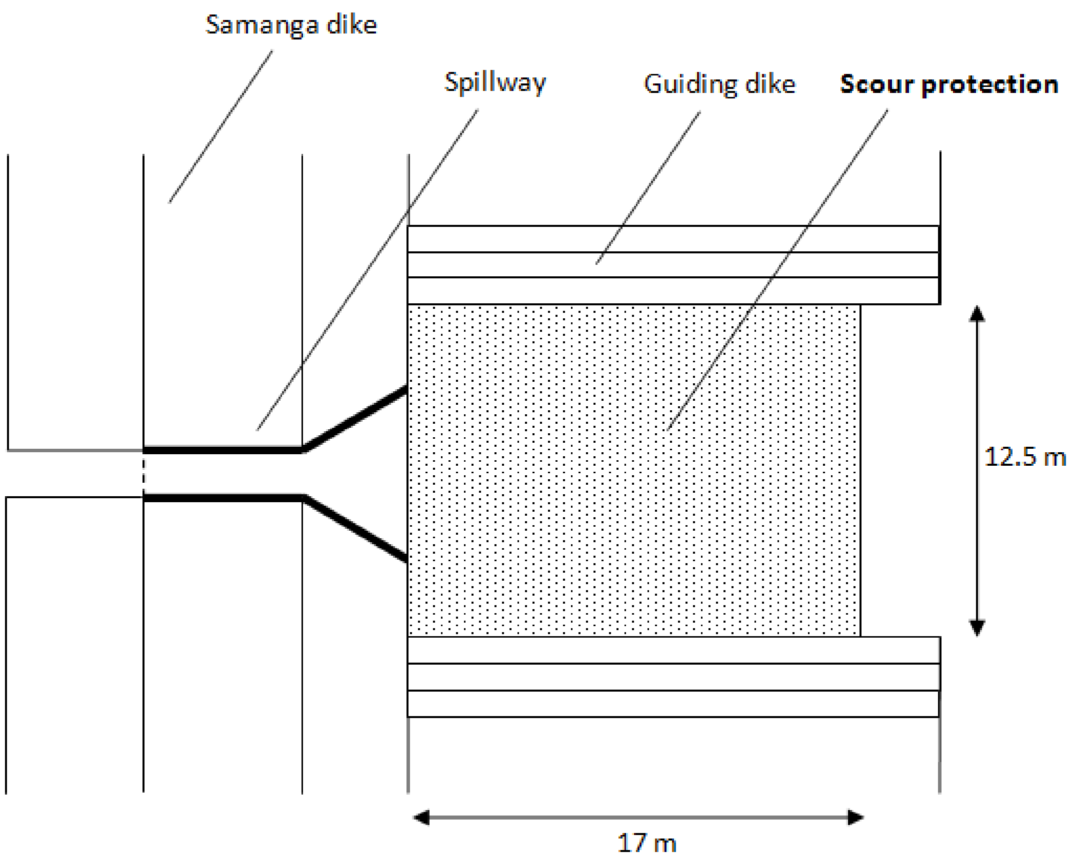

Figure 51: Plan view of the bed protection at the back of the spillway

15 Cluster 2 Integral Design

Cluster 2 compromises of the bifurcation point where the Kikuletwa North splits into the Kikuletwa South and the Ronga. In this cluster, a control structure intends to be placed near to the bifurcation point and excavation of part of the Kikuletwa South should take place, see Figure 52. The design of cluster 2 has changed significantly with respect to the initial design. Firstly, the changes made to the New Kikuletwa South will be explained. The location of the New Kikuletwa South is refined, the new dimensions are given, the stability of the new design is tested and the excavation is determined. Secondly, the integral design of the control structure is explained. The changes in reference level are described and the following final dimensions are shown including front, side and top views. Additionally, measures against scour are presented.

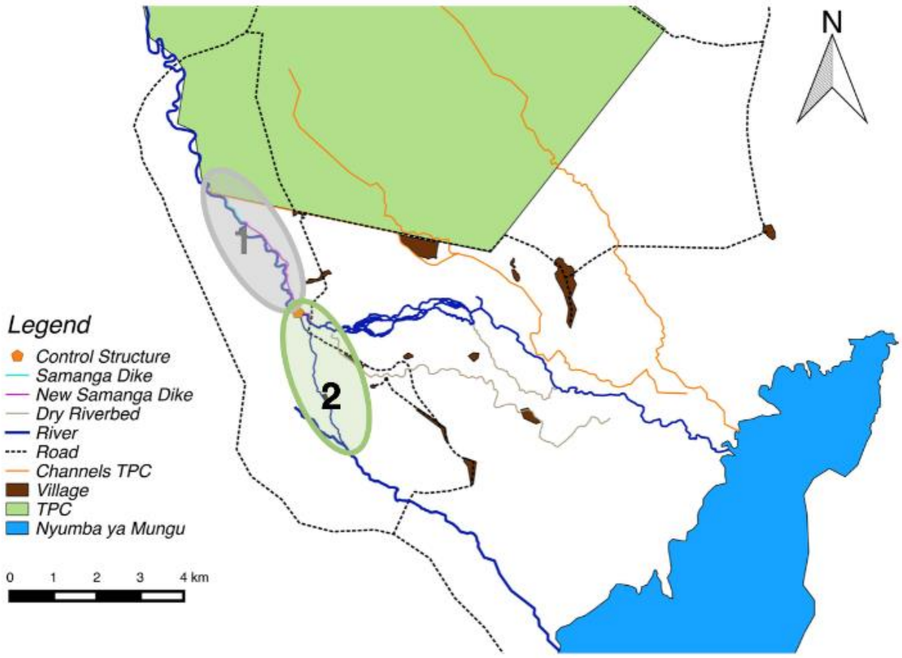

Figure 52: Location cluster 2

15.1 New Kikuletwa South

Two changes have been made to the New Kikuletwa South. The first one is that the location of the New Kikuletwa South has changed. The connection with the bifurcation has moved to the east side of the current Kikuletwa South Small after which it connects with the Kikuletwa South Small and follows it until the start of the old riverbed. From here on the New Kikuletwa South will be excavated to the connection with the Kikuletwa South in the shortest possible way where only little vegetation is present.

The second change is that a dike will be built to the east of the New Kikuletwa South. There are several reasons for this change. The first reason is that less soil needs to be excavated, which results in lower costs. The second reason is that some of the soil can be used to build the dike. The last reason is that the farmers along the river will have better access to the water in the river. This is because the river will be less deep compared to the initial design; the depth is measured with respect to the surface level.

15.1.1 Location

The new location of the New Kikuletwa South can be seen in Figure 53. The first change compared to the initial design is the connection with the bifurcation. This is now to the east of the Kikuletwa South Small. The second change is that the new river does not follow the old riverbed completely. The planned location for the new river will be just east of the old riverbed. This change has been made because the old riverbed consists of much vegetation, which is expensive to remove. Another reason is that the new location is shorter.

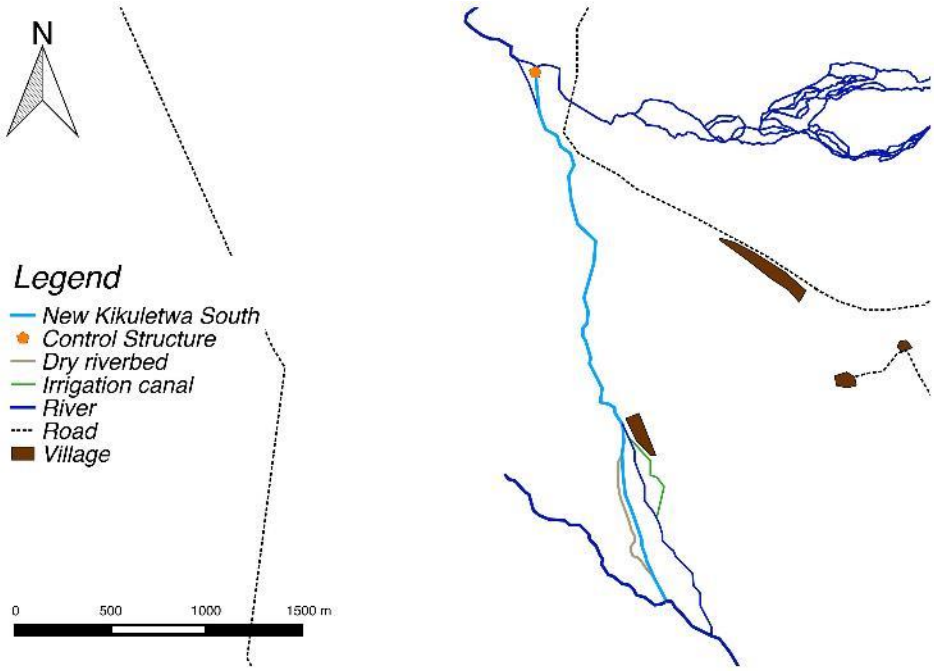

Figure 53: Location New Kikuletwa South

In Figure 54, an impression of the cross-section of the planned location of the New Kikuletwa South compared to the Kikuletwa South Small is shown. This location for the New Kikuletwa South has been chosen to lose as little as possible land on the east side of the river.

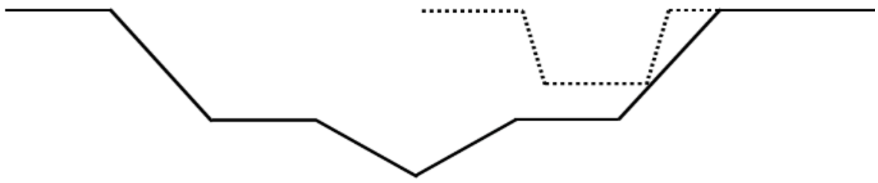

Figure 54: Cross-section New Kikuletwa South indicating Kikuletwa South Small (dotted line)

15.1.2 Design

The design discharge for both the short and the long rains for 1/15 year solution is used for the integral design of the New Kikuletwa South. These are given in Table 40.

Table 40: Design discharge New Kikuletwa South

	Short rains	Long rains
Discharge [m^3/s]	20	36

Cross-section

The basis of the integral design of the river is the same as the initial design. The river has a deep section for the dry season such that 1 m^3/s can be discharged. The main section of the river is, however, designed for the short rains instead of the long rains. Due to this change, the dimensions of the river are smaller. To make sure that the Kikuletwa South area does not flood during the long rains a dike is built. The dike is only built on the east side of the river because the district on the west side of the river wants that area to flood during the long rains. The cross-section of this new design is shown in Figure 55.

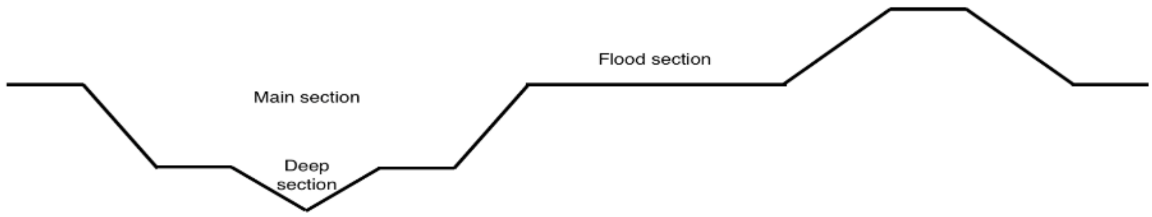

Figure 55: Cross-section New Kikuletwa South

Dimensions

The dimensions of the new river are calculated using Manning's equation. The assumed values for the bed slope and the roughness coefficient used for the integral design are given in Table 41. The explanation of the roughness coefficient for the flood section is given in Appendix D.4 - Design Calculations Kikuletwa South. The values of the roughness coefficient for the deep and main section are the same as for the initial design. The explanation for these and for the determination of the bed slope can be found in Appendix B.3 - Design Calculations Kikuletwa South.

Table 41: Bed slope and roughness coefficient New Kikuletwa South

	Q (1/15)
Bed slope [-]	0.0026
n deep section [$s/m^{1/3}$]	0.04
n deep + main section [$s/m^{1/3}$]	0.05
n deep + main + flood section [$s/m^{1/3}$]	0.05

The dimensions of the New Kikuletwa South and its accompanying dike are shown in Figure 56. The calculations are explained in Appendix D.4 - Design Calculations Kikuletwa South. The maximum water levels during the different seasons are given in Table 42.

Table 42: Maximum water level New Kikuletwa South

	Dry season	Short rain season	Long rain season
Maximum water level [m]	0.8	2.3	2.9

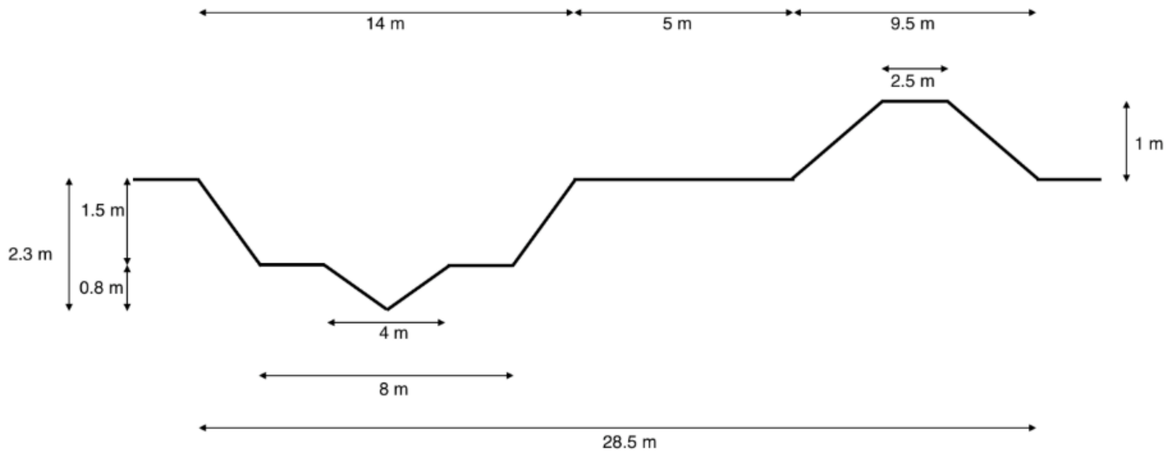

Figure 56: Dimensions New Kikuletwa South and dike

15.1.3 Stability

The different scenarios that are explained in Appendix D.1 - Soils were run through the D-Geo Stability program for both the excavation of the New Kikuletwa South and its accompanying dike. Impressions of the flooding scenarios are found in Appendix D.4 - Design Calculations Kikuletwa South. The resulting safety values for each scenario are found in Table 43 for both the dike and the excavation slope.

Table 43: Factors of Safety D-Geo

Loading scenario	River slope	Dike
Short rain	2.9	2.8
Long rain	1.4	2.3
Post flood	2.8	2.8
Dry season	1.3	2.3

The river slope has relatively low factors of safety for both the long rain flood and dry season. However, both of these situations disregard cohesion. As there is probably cohesion this factor of safety is accepted.

15.1.4 Excavation

The changes made to the location in the integral design resulted in a shorter distance that needs to be excavated. The cross-section of the new river is much smaller than in the initial design. Therefore, the total excavation has decreased compared to the initial design.

The volume that needs to be excavated is shown in Table 44.

Table 44: Excavation of the New Kikuletwa South

	Q (1/15)
Length [m]	3100
Cross-sectional area [m^2]	18
Volume [m^3]	56110

The volume of soil that is excavated should be used for the construction of the dike. The remainder can be used to make the dike even larger or to build a dike on the west side of the river if Msitu wa Tembo wishes.

15.1.5 Dike

The total volume of soil that is needed to build the dike is shown in Table 45. The volume needed for the dike is multiplied with 1.2 for compaction purposes, as explained in Appendix D.1 - Soils. The soil from the excavation of the river can be used to build the dike. Vetiver grass should also grow on this dike.

Table 45: Soil volume dike

	Q (1/15)
Length [m]	3,100
Cross-sectional area [m^2]	6
Compaction factor [-]	1.2
Volume [m^3]	22,320

15.2 Control Structure

The integral design of the control structure is presented here. Special attention is paid to the reference level, the workings of the crane constructions and the connection to the New Kikuletwa South. Additionally, scour protection is included in this stage.

15.2.1 Location

Reference level

In this stage, the reference level has been taken into account, which changes the dimensions of the control structure. A more detailed explanation is given in Appendix D.5 - Reference Level. The head level difference has been altered to 0.6 meters. The water depth on the south side of the structure is 2.9 meters in the new situation which can be seen in Figure 57, therefore the side walls on the south side will be taken as 3.1 meters with regard to the bottom level.

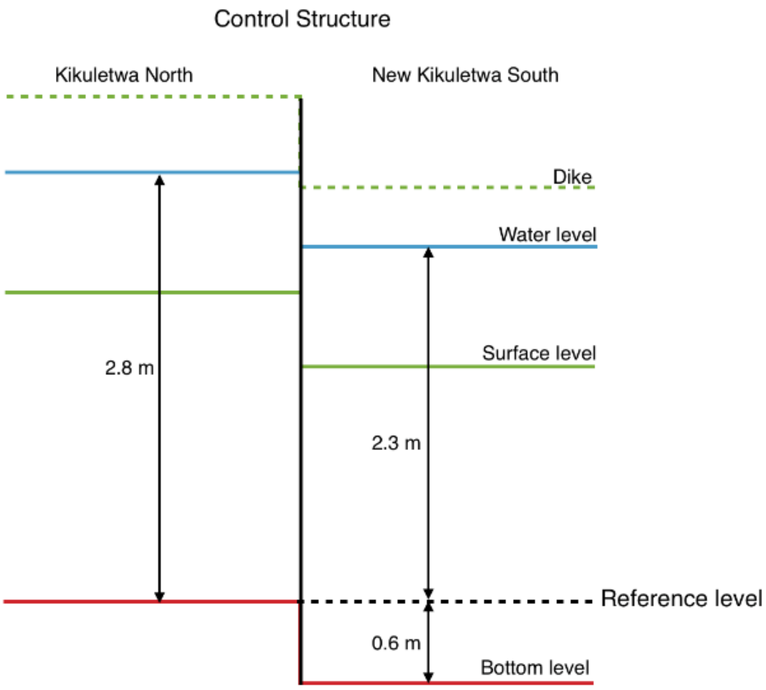

Figure 57: Reference level control structure

Connection New Kikuletwa South

The location of the control structure is shown in Figure 58. The location is different than the one proposed in the initial design. The connection to the Kikuletwa North is now placed to the east side of the current Kikuletwa South Small. There were several reasons for this change, which can be found in Appendix D.6 - Location Control Structure. First of all, it would be possible to build inside the PBWB area. Secondly, it reduces the chance on debris hitting the structure and sedimentation taking place due to it being built perpendicular to the Ronga. Lastly, it would be easy to connect to the Kikuletwa South. The opening of the Kikuletwa South Small should be blocked so that no water will enter that way.

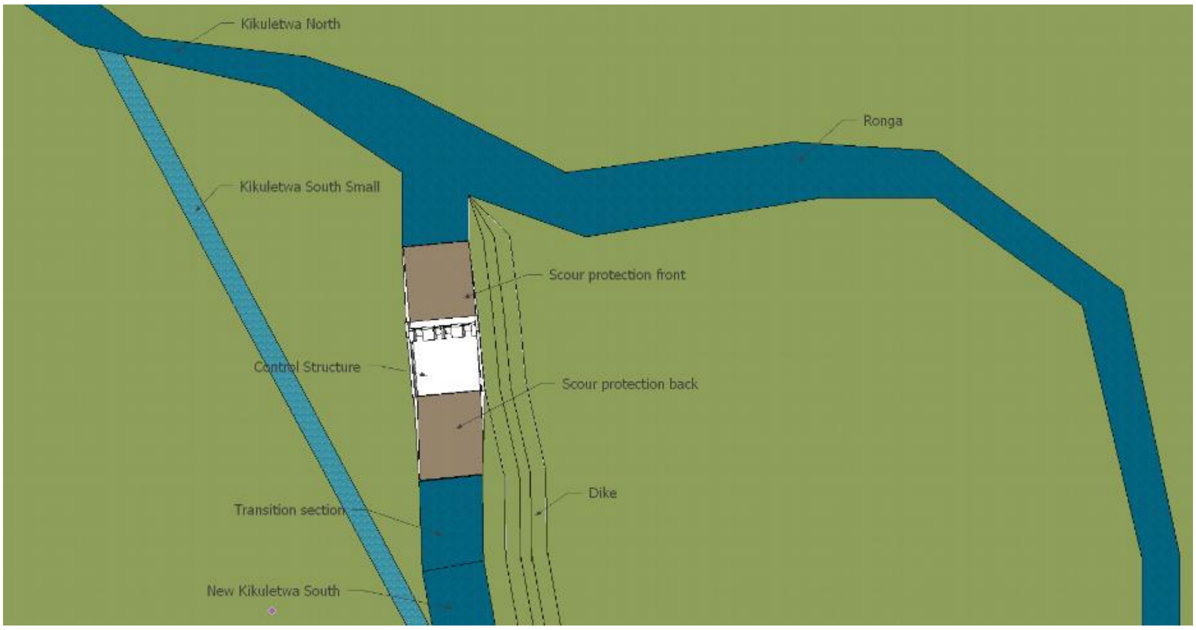

Figure 58: Location control structure

To connect the different cross-sections of the control structure and the New Kikuletwa South a transition section is needed. This transition section starts right after the scour protection and will have a rectangular cross-section at the start which will change gradually to the design cross-section of the New Kikuletwa South.

15.2.2 Design

The dimensions of the integral design are described here, taking into account the new assumptions regarding the reference level. All other dimensions remain the same. The design of the control structure is shown in Figure 59. There are four openings, in each opening three gates can be placed. The flow of water through the structure is indicated with the blue arrows. There will be two platforms in the middle of these openings, which can support cranes. These cranes should be manually operable. Since there will be excavation of the Kikuletwa South Small, there will be a bed level difference as the water flows through the bifurcation.

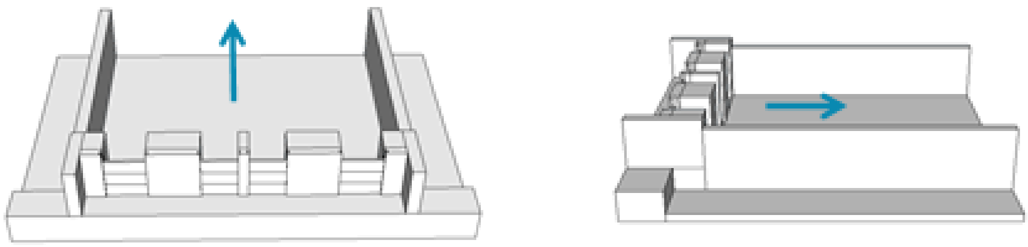

Figure 59: Visualisation of the control structure

The unity checks, which were used in the initial design, were run again with the new dimensions. They all have a positive outcome; see Appendix D.7 - Failure Mechanisms Control Structure for the results. Presented below are the final dimensions of the integral design.

Dimensions

The dimensions of the control structure can be found in the drawings of the front, side and top views presented shortly. These dimensions should not be regarded as detailed building drawings.

Front View

In Figure 60, the front view of the control structure is presented.

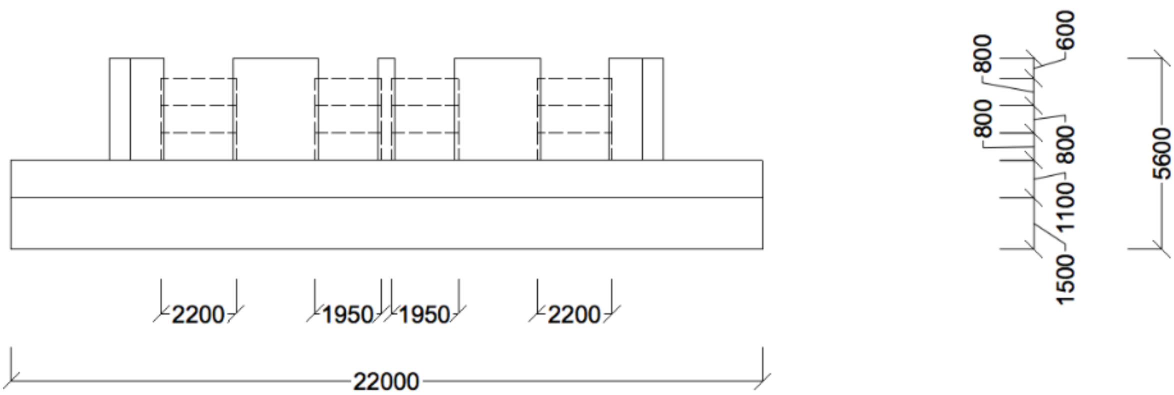

Figure 60: Front view control structure

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Side View

In Figure 61, the side view of the control structure is presented.

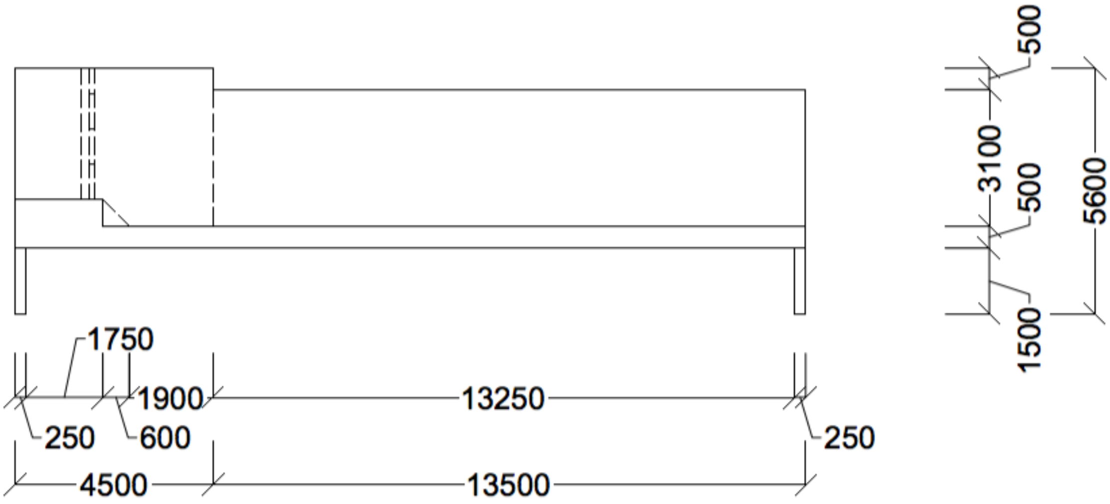

Figure 61: Cross-section control structure

Top View

In Figure 62, the top view of the control structure is found.

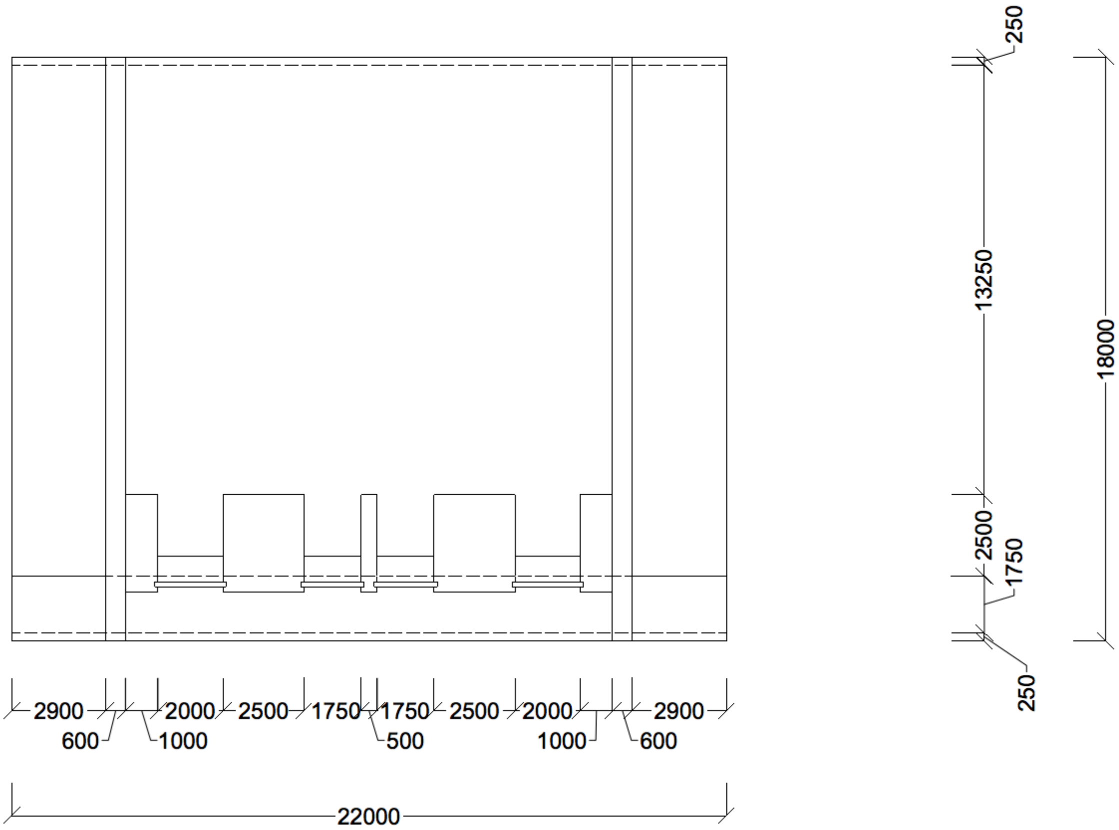

Figure 62: Plan view control structure

Scour Protection

A summary of the scour protection at the control structure is provided here, for a more detailed description the reader is referred to Appendix D.3 - Scour Protection. The appendix mainly focuses on bed protection against bed erosion. Other types of scour are not worked out in detail. Riverbank protection, especially at the entrance should be implemented in the final design.

Hard bed protection at the front and back of the control structure is essential to keep the scour hole that will develop on a safe distance from the structure. If the scour hole gets too close to the structure, a number of problems can occur. Firstly, the leakage length decreases resulting in a higher chance of piping. Secondly, the erosion can continue under the structure risking instability of the structure. Finally, a big hole close to the control structure can influence the flow through the structure.

The protection at the entrance and at the exit of the control structure is very similar. Both are hard protections consisting of a top layer and a filter layer. The filter consists of fine gravel; the same grading for both the entrance and the exit is used. The top layer of the entrance protection has bigger stones; this is the result of higher expected flow speeds.

The protection at the entrance will be 18.5 m long and as wide as the channel towards the control structure. The width of the bed protection is therefore 15 m. The top layer will consist of stones with a median diameter of 100 mm. The filter layer will be 15 cm thick and the top layer will have a thickness of 25 cm. The bed protection at the exit is 19 meters long and 15 meters wide. The top layer will consist of a 10 cm thick layer of coarse gravel. The filter layer is the same as at the entrance.

Lift Installation for the Control Structure

The crane that is chosen to lift the gates of the control structure is a swivel crane with a hand chain hoist. This type of crane can be manually used, which is a requirement since no electricity is available at the location of the control structure. Two cranes will be placed at the control platforms, which can be accessed by a bridge. The gates have two hooks on top and two holes at the bottom. The hooks are used to attach the crane to the gate and the holes make sure that the gates can be put on each other.

A plan view of the lift installation can be seen in Figure 63. For a complete explanation of the crane construction, see Appendix D.8 - Lift Installation for the Control Structure.

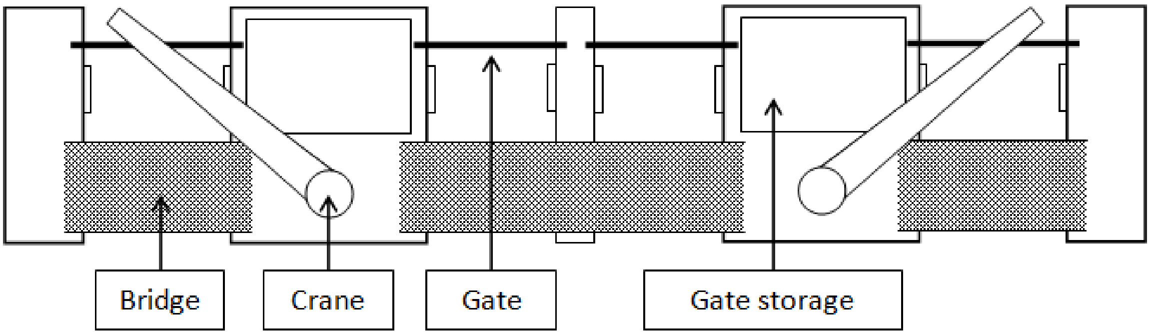

Figure 63: Plan view lift installation

16 Morphological Effects

In this chapter, the morphology of the river system is studied. This is done for the prevalent discharge in the dry season. This discharge is most common and therefore has the longest time to shape the rivers. For a complete morphological study, the variations in discharge throughout the year should be taken into account. Especially the high discharges should be studied, since they can cause a lot of erosion. This is however not essential for this project and can be done in a later stage of the design. Furthermore, the prevalent discharge will give a good indication about the morphological response of the rivers.

First, morphology is explained using the terms erosion and sedimentation. Then locations in the river system where the riverbed is likely to change over time are identified. This is followed by a brief conclusion for each river stretch. In this conclusion, the necessity of preventive measures is considered and possible additional effects are elaborated. Finally, the possibility of an increase of sediment flowing into the reservoir will be elaborated.

16.1 Morphology Explained

Most of the changes that are made in a river system have direct effects on the river. For instance, the flow velocity will increase or decrease. In many cases, these direct effects are the required changes for a successful solution. For example, a higher flow speed is necessary to increase the discharge capacity of the river. Besides the direct effects that the solution has, there are also the long-term effects. In the case of morphology, how will the shape of the river change over time due to the adjustments made?

The shape of a natural riverbed changes constantly due to two processes, erosion and sedimentation. Erosion is the removal of soil, which will be transported by the river. Sedimentation is the opposite, meaning that soil that is being transported by the river will settle. Both processes mainly depend on the flow speed in the river. Simply stated, high flow speeds cause erosion and low flow speeds sedimentation. Erosion and sedimentation in itself are not a big problem as long as it is equally distributed over the length of the river. Therefore, the earlier statement needs to be adjusted to: an increase in flow speed causes erosion and a decrease in flow speed causes sedimentation.

At locations where a lot of erosion occurs the riverbed will be lowered, resulting in a deeper river. This does not necessarily mean that the water depth will be larger, but rather that the bottom of the river lies deeper with respect to ground level. At locations with high sedimentation rates, the riverbed will get higher over time. This results in a smaller cross-sectional area of the river. Both effects can have negative consequences but they should not necessarily be prevented. For each situation the extent of negative effects should be analysed and from this analysis it can be determined if preventive measures are necessary.

16.2 Flow Velocity and Sediment Load

To detect possible erosion or sedimentation locations the flow speed of each stretch in the river system is calculated, see Appendix D.9 - Calculations Morphology. With the flow velocity, the sediment load can be determined; this is explained in detail in the same appendix. The sediment load indicates the amount of sediment the river discharges in that stretch. For the flow speed as well as the sedimentation load holds, if they decrease in the downstream direction sedimentation is likely to occur. On the other hand, if they increase in the downstream direction erosion is likely to occur. In Table 46, the results of the morphological study are summarized. The words little, medium and severe indicate the extent of the erosion or sedimentation. The equilibrium bed slope is also included; this is the bed slope the river will reach after a long time, assuming that the conditions do not change.

Table 46: Equilibrium bed slope and morphological effect

	Equilibrium bed slope	Erosion or sedimentation
Kikuletwa North	0.00103	Little erosion
New Kikuletwa South	0.0012	Medium erosion
Ronga	0.00082	Severe erosion
Ronga Braided	0.00117/0.00119	Medium sedimentation
Ronga South	0.00085	Medium erosion

The locations of the river stretches where erosion or sedimentation will occur can be seen in Figure 64.

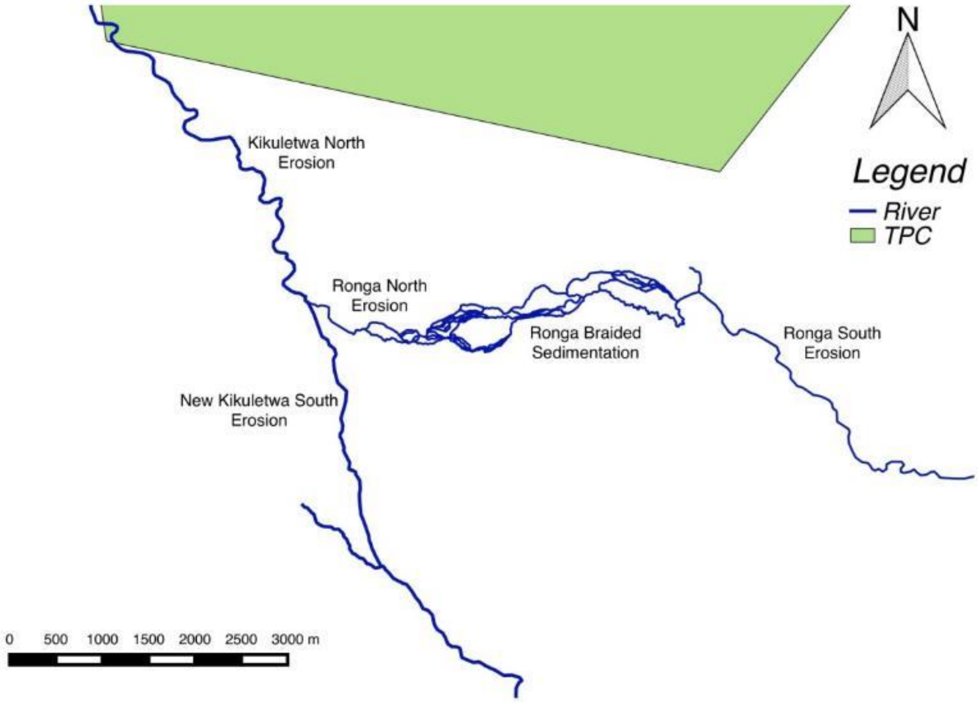

Figure 64: Map of the erosion and sedimentation locations

16.3 Conclusion

The locations where erosion and sedimentation are likely to occur are determined. Both effects should not necessarily be prevented. In this conclusion, each river stretch is highlighted and preventive measures are suggested, where necessary. Furthermore, possible positive effects are identified.

16.3.1 Kikuletwa North

The bed in the Kikuletwa North will have some erosion during the prevalent discharge. This will increase the capacity of the river, which can be seen as an additional benefit. No preventive measures are necessary. The lowering of the bed should be monitored however, to ensure that the water level does not get too low with respect to the ground level. A low water level can prevent the local farmers from irrigating their crops and should therefore be prevented.

16.3.2 New Kikuletwa South

The bed in the New Kikuletwa South will erode. Note that there will be a very low discharge through the river, therefore only the small channel in the centre of the river will erode. This is beneficial to discharge peak flows and can therefore be seen as a benefit. Since no negative consequences follow from the erosion in the centre channel of the New Kikuletwa South, preventive measures are not needed.

16.3.3 Ronga

The erosion of the bed in the small part of the Ronga just before the river starts braiding is expected to be quite severe. The higher flow velocities in this part mainly cause this. The erosion will not have any negative effects and will only increase the discharge capacity.

16.3.4 Ronga Braided

The braided part of the Ronga is the only stretch where sedimentation is expected during the prevalent discharge in the dry season. The sudden drop in flow velocity, which reduces the sediment load, can best explain this. A lower sediment load will result in settlement and thus sedimentation. The sedimentation will lower the discharge capacity of the braided part of the Ronga. This can have a bad influence on the flood safety in the short rains. However, it can be expected that much of the accreted soil will flush with the flood. Furthermore, there are many small channels, without extensive research, the conclusion that the capacity will be too low cannot be drawn. Taking all of this into account, preventing the sedimentation is not necessary. Further research is however advised or the sedimentation in the Braided Ronga should be monitored to ensure the capacity remains high enough.

16.3.5 Ronga South

In the southern part of the Ronga, erosion is expected. The flow velocity increases compared to the braided part and therefore an increase in sediment load is expected. A sudden increase in sediment load results in erosion. Erosion of the bed in the Ronga South has no negative effects on the flood safety and therefore no preventive measures are needed.

16.3.6 Sediment Flowing into the Reservoir

The sediment discharge into the Nyumba ya Mungu reservoir is not expected to increase. There is no increase expected, because the changes made to increase the flood safety are not expected to cause significantly more erosion, compared to the current situation. To obtain certainty about this statement additional research is needed. This includes a full morphological study, which cannot be realized within the time frame of this project.

17 Cost-Benefit Integral Design

In this chapter, the benefits and the costs of the integral design will be discussed. The benefits have not changed compared to the initial design, however, the costs changed due to new information and changes to the design. In Appendix C.1 - Determining the Present Value it is explained how the present value is calculated for the costs and benefits. The used exchange rate from Tanzanian Shilling to US Dollars is TZS 1 = $ 0.000458 (XE: (TSH/USD) Tanzanian Shilling to US Dollar Rate, 2016). Firstly, the benefits will be discussed. Secondly, the different costs are presented. Thirdly, the net present value of the project will be given. Lastly, a sensitivity analysis of the costs is conducted.

17.1 Benefits

The benefits for the design have not changed for the integral design. As discussed in chapter 12 Cost-Benefit Initial Design and Appendix C.2 - Monetary Benefits of the Designs, the total monetary benefits during the lifetime of the structures are $ 36,000,000 (TZS 78,600,000,000) with a present value of $ 11,544,000 (TZS 25,205,500,000*)*. These benefits consist of the prevention of crop loss, increase of farmland utilisation and reducing the flood damage of assets. In comparison with the initial design, the benefits have slightly increased due to different estimated start date of the project.

There are also non-monetary benefits. These are the benefits for which no monetary value can be determined. In Appendix C.3 - Non-Monetary Benefits these are described. They consist of the increase of welfare (health), school attendance, employment opportunities and the decrease of hunger and willingness to make long-term commitments. While they cannot be expressed in a value, they should be kept in mind while considering the total benefits of the solution.

17.2 Costs

An estimation of the lifetime cost for the integral design will be discussed in this chapter. The same assumptions are used as during the initial design stage, unless mentioned otherwise. In Appendix C.4 – Unit Rates and Appendix C.6 - Construction Costs of the Integral Design, these assumptions can be found.

The assumption is made that the construction of the project will start two years from now, in the year 2017. After the end of the long rain season in June/July, there will be a period of four to six months in which it will be dry enough for construction. This start date has been taken into account for determining the cost.

The costs that will be discussed are the project management, design, construction, operation and maintenance costs.

17.2.1 Project Management

In order to properly manage the project, a project team should be created during the preparation phase for the duration of the project. During meetings with local contractors it was estimated that one project leader/civil engineer and one or two assistants are required for the project. In Table 47, the overview of the project management cost is presented.

It is assumed that during the short rain season the preparations start for the project with first only the project leader, who will later be joined by his assistants a month before construction is started. The construction should be done between the end of the long rain season and the start of the short rain season, which is approximately 5 months. An extra month is included for finishing up the details. For the roles of the project leader and the assistants, it is assumed that local people will be hired.

Table 47: Project Management Cost

	Monthly Salary	Preparation Phase [months]	Construction Phase [months]	Costs	
Project leader/ civil engineer	TZS 2,500,000	5	6	TZS 27,500,000	$ 12,600
2x Assistants	TZS 2,400,000	1	6	TZS 33,600,000	$ 15,400
Additional expenses (office, transport, research, tender, etc.)	5.0%	-	-	TZS 216,635,000	$ 99,200
Total				**TZS 277,735,000**	**$ 127,200**

The additional expenses consist of transport for the team, office space, potential cost for a tender and other associated cost. Contrary to the initial design, the percentage has been decreased from 10% to 5% to give a more realistic estimate.

17.2.2 Finishing the Design

Before construction can start, the design will need to be worked out in more detail. This means detailed construction drawings should be made and a bill of quantities needs to be specified.

The cost of finishing the design is defined as a percentage of the total construction costs. In the initial design, the percentage (6%) was based on an interview with one contractor. Since the initial design, more meetings were held with other contractors and different percentages were given. It was also stated that this percentage could always be negotiated. For the integral design, a percentage of 12% will be used.

17.2.3 Construction

The construction costs are based on the calculated quantities of the design, unit rates received from different contractors and several assumptions that were made for the building method. This has been specified in Appendix C.4 – Unit Rates.

The total cost for the different clusters are given in Table 48 and Table 49. In comparison with the cost estimation of the initial design, the estimated construction costs have decreased. This is due to a better estimation of the cost for clearing the vegetation, which is explained in Appendix C.7 - Cost Estimation for Vegetation Clearing. Furthermore, the designs for the Kikuletwa South and Samanga Dike have been optimised, requiring fewer earthworks to take place.

Table 48: Construction cost integral design cluster 1

Cluster 1	Element Costs		% of total construction cost
Clearing vegetation	TZS 1,148,100,000	$ 530,000	26%
Samanga Dike	TZS 729,600,000	$ 340,000	17%
Spillways	TZS 179,200,000	$ 90,000	4%
Total	**TZS 2,056,900,000**	**$ 960,000**	**47%**

Table 49: Construction cost integral design cluster 2

Cluster 2	Element Costs		% of total construction cost
Clearing vegetation	TZS 1,238,200,000	$ 570,000	29%
Control Structure	TZS 534,900,000	$ 250,000	12%
Excavation	TZS 551,500,000	$ 260,000	13%
Total	**TZS 2,324,600,000**	**$ 1,080,000**	**53%**

17.2.4 Realisation Cost

Based on the previous sections the total realisation cost has been estimated which can be found in Table 50. As the project is still in an early design phase it is not possible to give a precise price, but rather an expected range of what the cost will be. This range is the same as used during the initial design and is based on the AACE as explained in chapter 12 Cost-Benefit Initial Design.

Table 50: Realisation Cost Integral Design

	Estimated cost		Expected Range	
			85%	140%
Project Management	TZS 280,200,000	$ 130,000	$ 110,000	$ 180,000
Design	TZS 525,800,000	$ 250,000	$ 210,000	$ 340,000
Construction	TZS 4,381,300,000	$ 2,010,000	$ 1,710,000	$ 2,810,000
Contingency	TZS 1,556,200,000	$ 720,000	$ 610,000	$ 1,000,000
Total	**TZS 6,743,300,000**	**$ 3,090,000**	**$ 2,630,000**	**$ 4,330,000**

The contingency reserve is estimated at 30% of the estimated project management, design and construction cost, which is the same percentage as during the initial design phase. Compared to the initial design phase the estimated realisation cost has decreased with roughly 40%.

17.2.5 Operation and Maintenance

In chapter 21 Implementation, the operation and maintenance have been explained and what is required for them. In total annually, 0.5% of the total construction cost should be reserved for maintenance and at least one supervisor should be hired to oversee both. In Table 51, the operation and maintenance cost for the first year and the entire lifetime are presented.

Table 51: Operation and Maintenance Costs

	Estimated Cost		Expected Range	
			85%	140%
First year	TZS 66,300,000	$ 40,000	$ 30,000	$ 50,000
Lifetime	TZS 2,953,900,000	$ 1,360,000	$ 1,150,000	$ 1,900,000
PV Lifetime	TZS 1,062,700,000	$ 490,000	$ 420,000	$ 690,000

17.2.6 Total Cost Lifetime

The total cost consists of the realisation and the operation and maintenance costs. In Table 52, the total lifetime cost and its present value are presented.

Table 52: Total and PV lifetime cost

	Estimated Cost		Expected Range	
			85%	140%
Total Cost	TZS 9,697,200,000	$ 4,450,000	$ 3,780,000	$ 6,220,000
PV Cost	TZS 7,806,000,000	$ 3,580,000	$ 3,040,000	$ 5,010,000

17.3 NPV and BCR

Based on the present value of the costs and benefits over the entire lifetime the net present value and the ratio between the benefits and the costs were determined. The results are presented in Table 53. Compared to the initial design for 1/15 return period, the NPV and ratio have increased. This is due to the lower realisation costs.

Table 53: NPV and Benefit Cost Ratio

Net Present Value	TZS 17,399,600,000	$ 8,000,000
Benefit Cost Ratio	3.23	

17.4 Sensitivity Analysis

The calculated costs and benefits are based on many variables. With a sensitivity analysis, the relative importance of the various variables will be determined. This will lead to the identification of the variables to which the project is the most sensitive.

For the various variables, the switching values (the percentage/absolute number a variable need to change for the NPV to become zero) and sensitivity indicators (compares the percentage change in variable with the percentage change in the NPV) will be determined. This will help identify the potential risks/threats to the project (Verhaeghe, 2009). To identify the changes, only one variable is changed at the same time, while the rest remains the same.

In Appendix C.9 - Sensitivity Analysis the full analysis can be found. The results are presented in Table 54. All values with (A) behind them are absolute changes to the variable. For example, a +1% for the discount rate means that the discount rate was changed from 8% to 9%. In case (R) is behind the value, it means that it’s a relative change. With other words, the variable is changed by that percentage. For example, a + 1% for the unit rates means that the rate was increased from $1,000,000 to $1,010,000.

Table 54: Results Sensitivity Analysis

Variable	Switching Indicator		Switching Value	
	Variable	NPV	Variable	NPV
Lifetime				
Discount Rate	+ 1% (A)	-15.4%	22.8% (A)	-100%
Inflation	- 1% (A)	-17.6%	0% (A)	-60%
Operation and Maintenance	+ 1% (R)	-0.06%	+1665% (R)	-100%
Value of yield per acre	- 1% (R)	-1.42%	-71% (R)	-100%
Construction				
Unit Rates	+ 1% (R)	-0.41%	+246%	-100%
Start Date	+ 1 year (A)	+4.4%	-	-
Time	+ 1 year (A)	-5.82%	-	-
Time till full farmland utilisation				
Short Rain Season	+ 1 year (A)	-0.4%	-	-
Long Rain Season	+ 1 year (A)	-1,78%	-	-

Not all the variables used in determining the cost and benefits were tested in the sensitivity analysis. Only those that are the most uncertain or are thought to have a large impact on the net present value. These are mainly the long-term effects (increased use of the farmlands) and costs (limited sources). In the analysis, it was determined that the internal rate of return for the project is 22.8%, this is the discount rate for which the NPV becomes zero.

Furthermore, scenarios were made where several variables are changed. This was done to determine the vulnerability of the project in case the agricultural benefits are far less than estimated. In Table 55, the outcome of two scenarios can be found.

Table 55: Result Scenarios

Scenario	Changed Variable	NPV	
		%	Dollar
No extra use of farmland	Time till full farmland utilisation	-50%	$ 3,971,700
High supply of agricultural goods on the market	Value of Yield per acre Inflation Time till full farmland utilisation	-33.3%	$ 5,292,400

It can be observed that even in the case no extra harvests will be done by the farmers during the year, the project will still have a positive net present value.

Concluding, none of the variables have enough influence to cause a negative net present value of the project. The inflation has the largest influence on the net present value and is also the most uncertain. However, only in case of serious continuous deflation, the net present value will become negative. In general, the higher the inflation, the better the net present value.

The discount rate also has a large influence on the net present value, however, it is not expected that a discount rate close to the switching value will ever be chosen.

The benefit of increased utilisation of farmland during the rain seasons has most to gain from encouraging farmers to harvest during the long rain season. However, even in the case that none of the farmers will make extra use of their lands, the NPV will still be positive.

18 Validation of the Integral Design

In chapter 6 Validation, the prefeasibility design was validated for several criteria. The same will be done for the new design to identify if it meets the standards set at the beginning. An overview of this design is presented in Figure 65.

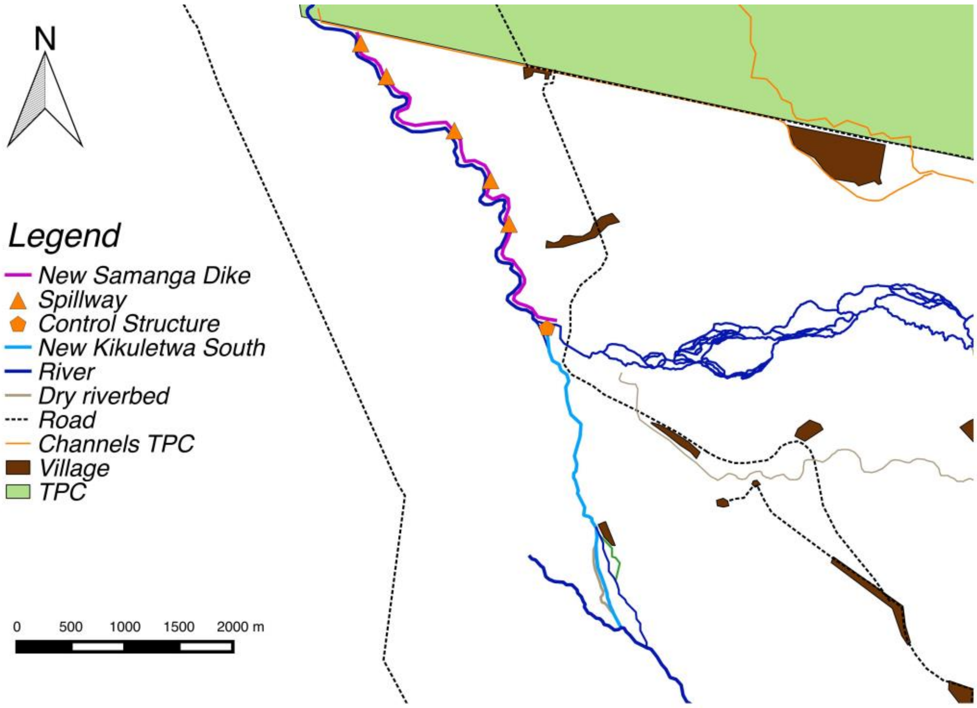

Figure 65: Final Design

The full validation can be found in Appendix D.10 - Validation of the Design, the results are presented in Table 56.

Table 56: Results of the Validation

Criteria	
Risks	Partly, research required
Cost Effectiveness	Yes
Water Management of the Agricultural Land	Yes
Locations of the Designs	Yes
Resources and Construction Method	Yes
Morphological Effects	Partly, research required
Operation	Yes
Maintenance	Yes
Longevity	Yes

The design meets most of the previously set criteria. The criteria that are not completely met are the risks and the morphological effects. This is due the lack of data, which requires more time and effort to obtain than was available during the project. In the recommendation it will be advised what should be done in order to meet these criteria.

19 Conclusions Integral Design

In the detailing of the integral design, several changes and additions were made.
Cluster 1:

- A dike along the Kikuletwa North including manually controllable spillways.
    - Additional guiding dikes are necessary along the path of the spillways in the Samanga dike. This is due to the gap that emerges when the soil is dug up from behind when it is built.
    - For the spillways soft scour protection is needed before the structure and hard scour protection behind the structures.

Cluster 2:

- Widening the narrow part of the Kikuletwa South.
    - A dike on the eastern side should be built to reduce excavation and to provide a place for the excavated soil.
    - The location was changed; the connection with the bifurcation has moved to the east side of the current Kikuletwa South Small after which it connects with the Kikuletwa South Small and follows it until the start of the old riverbed. From here on the New Kikuletwa South will be excavated to the connection with the Kikuletwa South in the shortest possible way where only little vegetation is present.
- A manually controllable control structure in the Kikuletwa South, near the bifurcation.
    - For the control structure, hard scour protection is needed both before and after the structure.
    - Further investigation was done into the crane construction for the gates of the control structure. The crane that is chosen to lift the gates of the control structure is a swivel crane with a hand chain hoist.
    - The location has moved to the east side of the current Kikuletwa South Small because this was concluded to be the most efficient location.

From the morphology study of the prevalent discharge, it was found that most riverbeds would erode over time. Only the braided part of the Ronga experiences sedimentation. The morphology study concluded that the erosion of most riverbeds would increase the capacity of the rivers, which can be seen as an additional benefit. Sedimentation of the Ronga will likely not cause problems, because the floods will flush away accreted sand. No increase in sediment entering the reservoir is expected.

The costs and benefits were determined for the entire lifetime of the project. The costs of this design were determined to be 4.5 million US dollars, with an estimated range between 3.8 and 6.2 million US dollars. The present value of these costs is 3.5 million US dollars. The benefits of the design were estimated at 36 million US dollars with a present value of 11.5 million US dollars. The benefits combined with the costs results in a net present value of 8 million US dollars for the design and the associated cost benefit ratio of 3.23 with an internal rate of return of 22.8%.

Part E – Risks and Implementation

The main goal of this part is to identify the risks and give a recommendation for the implementation phasing of the design presented in the previous part. It is important to realise that this is only a recommendation and that this can change when the detailed design is prepared.

Firstly, the risks are identified in all possible phases of the project. These phases link directly to the implementation of the design. These risks have been put into a risk register, which needs to be updated regularly.

Secondly, the work breakdown structure (WBS) will be treated to provide the reader with an overview of the work that has to be carried out when implementing the design. Thereafter, the work needed for preparation of the project will be elaborated. Hereafter the construction method and the preferable phasing sequence will be treated. Then the operation procedures for the spillways and the control structure will be explained. Finally, the recommendations for the maintenance during the lifetime of the structure will be presented.

The risk register and the implementation plan are dependent on one another. In many cases, the risks identified led to measures in the implementation plan. Vice versa, many of the risks follow from measures taken in the implementation plan.

Figure 66: Picture taken during fieldwork, logjam in the Kikuletwa North

20 Risk Register

The risk register is a document, which should be updated regularly during the entire process of the project. The risk register shows the detected potential risks up until this point in the project. Additional risks that are identified later on in the project should be added to the risk register. The entire risk register can be found in Appendix E – Risks and Implementation
Appendix E.1 - Risk Register. It has not been added to this report, as it is very extensive. Not all of the risks will be explained, only the ones identified as most essential for the project at this stage. The risk register is divided into five parts; general, Samanga dike, spillways, excavation and control structure. For each of these parts the risks during preparation, construction, operation and maintenance as well as any external risks are identified.

During preparation, the most important risks are that incorrect data is gathered, incorrect assumptions are made and that mistakes are made in the calculations. Measures to reduce these risks have been identified as surveying the area, both horizontally and vertically, doing more extensive soil testing and having an engineering company validate these things and make a detailed design and bill of quantities. A second important risk during preparation is that Pangani Basin Water Board refuses to give permission for the project. The measure to reduce this is to keep communication with them intact. Thirdly, there is a risk of loss of information between phases. This is reduced by creating a project team, which oversees the entire operation.

During construction, the most important risks are that the contractor deviates from the design leading to the structures not meeting the standards. To reduce this risk a communication network between design team and contractor is necessary. Additionally, reviews and checks during construction process by the design team should take place and contractors should be chosen based on reputations to ensure they have sufficient quality.

During operation and maintenance, the most important risks lie within the collaboration and communication between the villagers, the farmers and FTK. The first risk is that due to insufficient, visible results in the start, the local farmers stop supporting the solution and do what they think is best. These things can lead to the structures not being maintained properly and being damaged. Additionally, risks are that farmers do not know how to operate the control structure and the spillways and there is no clear division of responsibility for the operation leading to the structures not operating as they should. To reduce these risks, clear operation and maintenance plans should be made and the communities need to be educated on these. Furthermore, a person(s) needs to be appointed to be in charge of the warning system, the control structure and the spillways. There should be checks to ensure that the operation and maintenance are being done appropriately.

External risks that are important include extreme weather conditions and afforestation/deforestation in the Kilimanjaro catchment area, which can lead to damage of the structures or the design being wrongly dimensioned. To reduce these risks, it is recommended to design flexible and robust structures. Another risk is that Msitu wa Tembo district decides to build a dike on the opposite side of the Kikuletwa North. Msitu wa Tembo should be kept informed on the advancements of the project. Lastly and most importantly the farmers could distrust the solutions and do not plant as much as they could. This should be prevented by educating them properly on the solutions.

21 Implementation

The implementation of the design is described here. This implementation plan has been constructed as a separate document, but many of its recommendations follow from the risks identified previously.

21.1 Work Breakdown Structure (WBS)

The WBS is a hierarchical subdivision of the project in smaller components. The main goal of the WBS is to provide the reader with an overview of the work that has to be carried out when implementing the design and during the lifetime of the structure. Different levels have been used to break down the project into smaller parts. The first level shows the total project. The second level shows the four main parts, which are of importance for the implementation and during the life cycle of the structures. The four main parts are each divided into sublevels and elements. The total Work Breakdown Structure is shown in Appendix E.2 - Work Breakdown Structure. Figure 67 shows the first two levels.

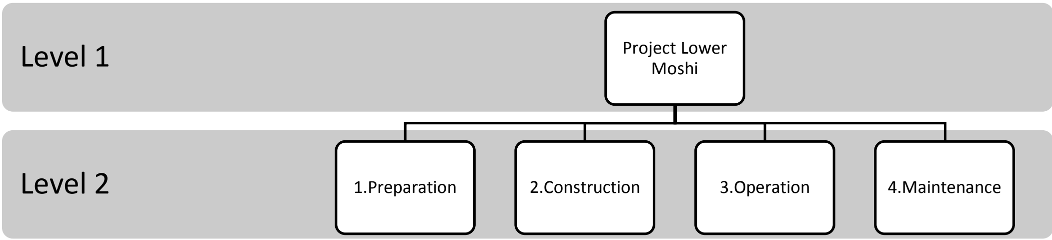

Figure 67: WBS level 1 and level 2

As stated above the four main parts of level 2 each consist of sublevels. These sublevels will be elaborated in the parts below. The headings used in the following parts refer to their sublevels in the WBS and are not headings related to the chapter.

21.2 Preparation

The first phase of the project will be the preparation phase. In this phase the detailed design will be made, the project organisation created and the tender will be held. Figure 68 shows the levels of the work breakdown structure concerning the preparation phase. Following this is an explanation of the shown sublevels.

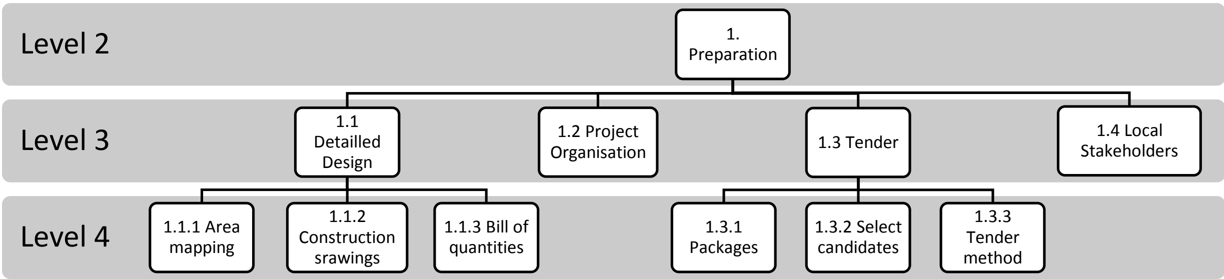

Figure 68: WBS preparation phase

1.1 Detailed Design – external engineering company

As there are still many unknowns at this point, additional research will need to be done and the design will need to be developed in more detail. It is advised to outsource the detailed design to an engineering firm or a contractor who has the required knowledge. Outsourcing the detailed design will also transfer the design risk to that company. For a detailed design, it is important that additional calculations are performed in order to ensure the safety of the design. Calculations for the failure mechanisms, reinforcement, material quantities and the exact dimensions should be done. When the calculations are complete, construction drawings that can be used by the contractor have to be made.

1.1.1 Area mapping – TPC/surveyor company/external engineering company

It is advised to survey the area where the solutions are planned. The surveying will provide data, which makes it possible to determine the exact locations of the dike, spillways, control structure and the excavation. It is also advised to survey the cross-sections of both the Kikuletwa North and Kikuletwa South and the present Samanga dike, so that the accuracy of the calculations can be improved. Furthermore, having an accurate/detailed elevation map of the entire project area will allow improving the model of the required discharge for flooding during the long rains. Additionally, knowing more precisely where the water will flow will increase the effectiveness of the solution. On the long-term, it will allow better management of the irrigation and flooding.

This work could partially be done by TPC if they can spare the surveying crews for horizontal mapping. The vertical mapping will need to be outsourced. An option is also to let the external engineering company arrange the complete mapping of the area. In any case, it is important that the same reference system is used for the locations/heights by the different parties to prevent misunderstandings during design or construction.

1.1.2 Construction drawings – external engineering company

The construction drawings are required for the contractor before he can start the construction work. These drawings will specify the entire design including all details. This part of the detailed design should be done by the external engineering company.

1.1.3 Bill of quantities – external engineering company

To allow contractors to bid on the project/submit their price; a bill of quantities is normally required in Tanzania. In it, the required work and associated quantities are specified. This should also be done by the same external engineering company.

1.2 Project Team – FTK/TPC

It is advised that from the start a project team is set up for the duration of the preparation and construction phase. This project team should have sufficient knowledge of construction in Tanzania. Later on the knowledge gained during these phases should be transferred to the operation and maintenance phase. The reason for such a team is that the loss of knowledge should be prevented. It is important that during all phases it is known why decisions were made and what assumptions are used. Additionally, the work done by the engineering company and contractors will need to be supervised to check if things go as they should. In the beginning, the team can be small, consisting of one person with sufficient knowledge and experience. Later on, the team can be expanded with assistants to help oversee the construction work.

1.3 Tender of Work – FTK/Project team

After the project team has been set up and the detailed design including the bill of quantities has been made the work can be tendered. It is advised to grant the work to different contractors, as there are different kinds of solutions and each contractor has their speciality. In Tanzania, the contractors are divided into classes, based on the value of the contract that they can accept and this should also be taken into account.

1.3.1 Packages – FTK/Project team

In the report four partial solutions have been presented (Samanga dike, Spillways, Widening Kikuletwa South, Control structure) and it is advised to use the same division for dividing the work with an option to create a fifth package for the removal of the vegetation. The reasons for this are the different types of work (soil and concrete), the size of the project (limitation due to classes of contractors), separated construction locations (different sides of the Ronga/Kikuletwa) and limited construction time (dry season).

1.3.2 Select candidates – FTK/Project team

It is important to have a good quality of work, as the solution needs to last for 25 years and the maintenance cost will be lower if the work is done properly. The used selection criteria are dependent on the requirements set by the sponsors. It is advised to invite a selection of contractors based on their reputation and not have an open tender. Knowing that the contractors can be trusted can prevent many potential problems. Another option is to grant the packages without a tender to contractors, which already collaborated with FTK previously. The benefit is that you know the work will be done well, but the downside is that it might cost more as there is no competition.

1.3.3 Tender method/criteria – FTK/Project team

The tender can be held once the work has been divided into packages and the method for selecting the candidates has been determined.

It is advised to use a *"Fixed Price"* contract for all packages. This type of contract entails that the contractor receives a fixed price for the work. The work that needs to be done for this project is of low complexity and they are common types of work in the area. This will also provide a relative certainty about the costs of the project for the client, as they are set beforehand. The downside is that they will need to be specified in detail to prevent that additional work outside of the contract will need to be performed. This will add additional costs to the project. With proper surveying of the soil and area beforehand by the engineering company during the detailed design phase no big surprises should be expected. Furthermore, the work can be specified in detail beforehand by the engineering company as it is rather straightforward.

This type of contract will require close supervision of the contractors during construction to prevent that they will cut corners to save on costs. They could start using cheaper quality materials, perform marginal workmanship or extend the completion date to reduce costs (Nicholas & Steyn, 2012). It would be wise to specify the completion date in the contract, as it has to be done before the rain season starts again.

1.4 Local Stakeholders - FTK

During the preparations for the project, the local stakeholders will need to be managed. The villagers/farmers will need to be informed of what will happen when and how it will benefit them. Moreover, the farmers near the river where the construction will take place need to be compensated by the local community.

Regarding the removal of vegetation, identifying the local options for the removal of the vegetation can significantly influence the cost for the project. It will need to be determined in cooperation with the villagers/farmers what they are able to contribute to the project. Examples of potential roles that the villagers can fill are in Appendix C.7 - Cost Estimation for Vegetation Clearing and Appendix C.8 - Operation and Maintenance Costs.

21.3 Construction

The second phase of the project will be the construction phase and can start when the preparation phase is finished. Figure 69 shows the levels of the work breakdown structure concerning the construction phase. Firstly, the recommended execution order will be treated and thereafter a short explanation of the recommended building method will be given. This is to give an impression of the work that needs to be done. The final building method has to be determined by the executive contractor. A visualisation of the construction concerning the spillways and the control structure is given in Appendix E.3 - Construction Visualisation.

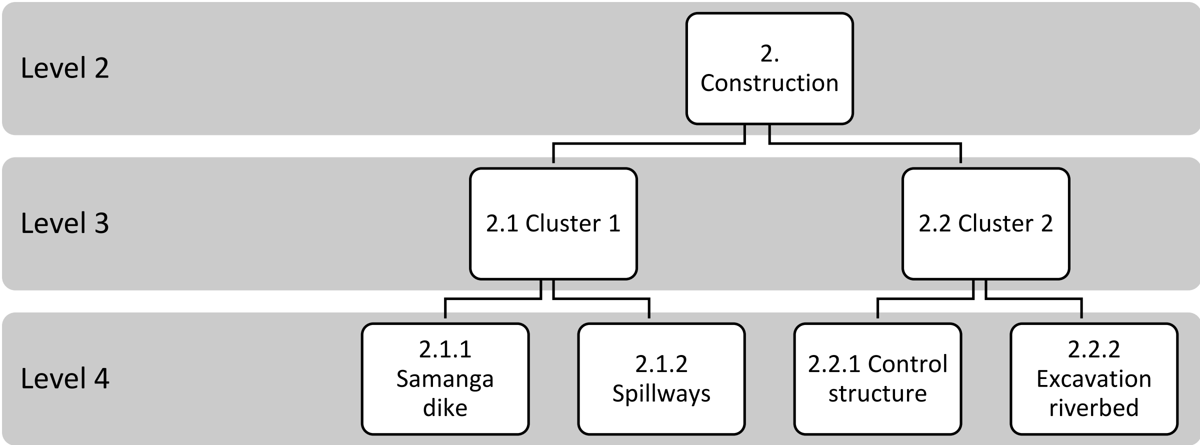

Figure 69: WBS construction phase

21.3.1 Recommended Execution Order

There are two different options for the execution order. The first one is to start the construction of cluster 1 and cluster 2 at the same time, so that the solutions can also start operating at the same time. The second option is to start with the construction of cluster 2 and later on start the construction of cluster 1. This could be done, for example, if there is not yet enough money available to carry out the entire plan at once. It is important that the construction is finished before the end of the dry season, otherwise the construction work will obstruct the river during the rainy season and this can cause severe floods and the building pits can fill with water due to the rain.

It is **not recommended** to start with the construction of cluster 1 and construct cluster 2 afterwards. If this is done, the constructed Samanga dike will ensure an increase of the amount of water that flows through the Kikuletwa North. Consequently, an increased discharge will arrive at the bifurcation point south of the Samanga dike and this will result in very large floods at this point. This has to be prevented and therefore it is not recommended to use this order.

21.3.2 Sublevels

2.1 Cluster 1

The construction of cluster 1 consists of two parts; the Samanga dike and the spillways. First, the construction of the Samanga dike must be started and then the spillways can be built.

2.1.1 Samanga dike

The existing dike has to be adapted and a new part of the dike has to be built. As mentioned in chapter 10 Cluster 1 the construction of the dike will have five steps. The associated steps will be elaborated below (CIRIA, 2013).

First, the site has to be cleared of vegetation. This involves the cutting of woody vegetation, trees and large bushes from the existing dike and removing the small vegetation and grass. The clearing should be done at the existing dike, the new dike footprint and the area behind the dike from which soil shall be taken to build the dike.
Then the organic topsoil layer can be removed. It is important because the organic materials are subject to undesirable consolidation and degradation and will be forming weak and unstable zones under the dike. The topsoil removal can be done with a bulldozer or excavator.
Following this, the embankment has to be constructed and this involves the controlled deposition, spreading, and compacting of earth to the extent required by the detailed design. The earth will be deposited and compacted in small layers of approximately 30 centimetres until the required height is reached. The guiding dikes behind the spillways, as explained in Appendix D.2 - Elaboration on the Guiding Dikes, should be constructed in the same way. The slopes of the dikes should also be compacted to prevent erosion and shearing.
Hereafter a layer of organic soil has to be spread on the slope for the purpose of sowing and supporting Vetiver grass. The Vetiver grass will be kept in nurseries by the local farmers who will take care of the vegetation in a later stage. The previously won topsoil can be reused, as this is already present at the site. At last, the Vetiver grass should be planted.

2.1.2 Spillways

Two spillways are located in the existing dike and three spillways should be constructed in the new dike. For the spillways in the existing dike, an extra step should be taken into account since they need to be replaced with new ones. After the removal of the organic topsoil layer, mentioned in the part above, the construction of the spillways can be started for the last three spillways. The extra step for the spillways in the existing dike is the excavation of the existing dike to remove the old spillways and create space for the new spillways. Hereafter the construction method is the same for all the spillways.

First, the space for the foundation of the spillway should be excavated and a rock layer has to be applied at the bottom of the excavation. Then plastered sheets will be applied to avoid saline intrusion of the concrete. Hereafter, the formwork and reinforcement for the foundation will be placed and the concrete foundation can be poured.
After hardening of the concrete, the foundation formwork can be removed. Then the formwork and reinforcement of the rest of the structure can be placed and the rest of the concrete can be poured.
Simultaneously the steel gates and winding mechanisms can be fabricated in a workshop so that they, after the removal of the formwork, can be placed in the structure.
At last, the required scour protection can be placed behind the spillway.

Then the rest of the dike will be made on top of the foundation plate as mentioned in the part above.
Following this the guiding dikes can be constructed.

2.2 Cluster 2

The construction of cluster 2 consists of two parts; the control structure and the excavation of the riverbed. The construction of the two parts can be started simultaneously; however, the connection between the control structure and the new riverbed can only be made when the control structure is finished.

2.2.1 Control structure

First the building pit should be made. The construction work will be executed in the dry season and to avoid water from the river from entering the building pit a minimum distance of 3 meters should be present between the river and the building pit excavation.

When the building pit is constructed, a rock layer at the bottom of the excavation has to be made. Then the formwork and reinforcement for the foundation will be placed and the concrete foundation can be poured.

After hardening of the concrete, the foundation formwork can be removed. Following, the formwork and reinforcement of the rest of the structure can be placed end the rest of the concrete structure can be poured.

Simultaneously the concrete gates, bridges, ladders and cranes can be fabricated in a workshop so that they, after the removal of the formwork, can be placed in the structure.

Following this, the required scour protection should be placed in front and behind the structure.

Hereafter the building pit will be closed and soil will be placed around the structure. Finally, the control structure will be connected to the Ronga so water can start flowing through the structure.

2.2.2 Excavation riverbed

The recommendation for the excavation of the New Kikuletwa South riverbed is to start downstream and then work toward the control structure upstream. This is recommended because in this way the groundwater can immediately flow away to the downstream part of the river. Another benefit of starting downstream is that the control structure will be finished when the excavation work reaches the upstream part. This will make it easier to connect the control structure with the excavated New Kikuletwa South.

First, the vegetation of the top layer has to be removed. After that, the riverbed can be excavated layer by layer until the designed cross-section is reached. The excavated soil can be used to construct the dike at the east side of the New Kikuletwa South. The construction involves the controlled deposition, spreading, and compacting of earth to the extent required by the detailed design. The earth will be deposited and compacted in small layers until the required height is reached. The slopes of the dike should also be compacted to prevent erosion and shearing. Vetiver grass should be planted on the dike in the same way as for the Samanga dike. Finally, the excavated riverbed can be connected to the control structure upstream, so that water can start flowing through the New Kikuletwa South.

21.4 Operation

After the construction of the solution, it is important that the gates in the structures are operated in order to ensure the wanted flooding during the long rains and prevent the unwanted flooding during the short rains. The structures are designed in such a way that the local farmers, with a certain amount of training by FTK, can operate the structures manually. It is still very important that FTK assign one supervisor who checks if the structures are used properly in order to achieve the desired effects. Figure 70 shows the levels of the work breakdown structure concerning the operation phase. The operation of the spillways is different from that of the control structure and therefore they will be treated separately in the part below. A visualisation of the operation procedures concerning the spillways and the control structure is given in Appendix E.4 - Visualisation Operation Phase.

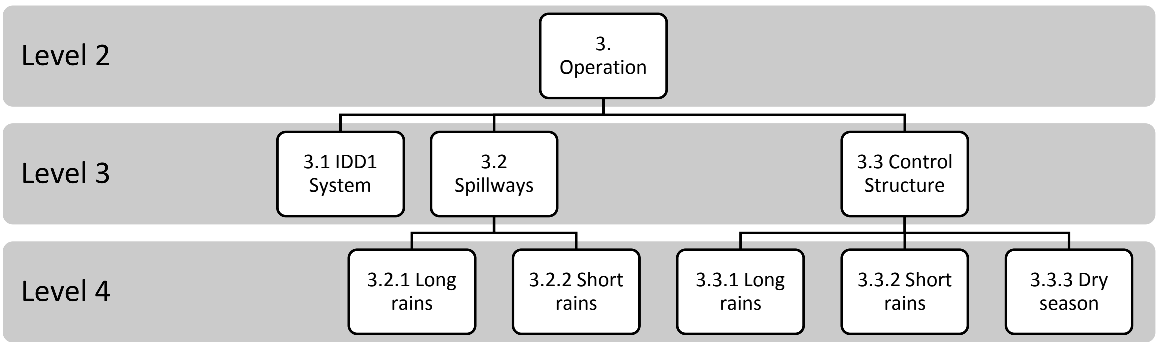

Figure 70: WBS operation phase

3.1 IDD1 Warning System

A warning system during the rainy seasons that is situated at IDD1 is needed to ensure the proper working of the spillways and control structure. At this moment, the water levels are measured twice a day at the location of the IDD1. Due to lack of knowledge on how long it will take to change the configuration of the gates and how long it takes a flood peak to arrive at the spillways and control structure a recommendation cannot be given on the frequency of measurements. It can be said, however, that twice a day is probably insufficient as the flood peaks travel quite rapidly and it will take some time to change the configuration of the gates once the warning signal has been distributed.

The warning system will make use of this data, but the extra part is that the supervisor of the project will get this data and a warning when the water levels reach the point that the configuration of gates in the structures should be adapted. For this, a collaboration with Pangani Basin Water Board is needed as they are in charge of IDD1. Based on the data the supervisor can make a choice about the opening configuration of the gates in the spillways and the control structure. The supervisor needs to communicate his decision to the local farmers, which then can open or close the gates. In Table 57 the water levels at IDD1 for the extreme events are given. If these water levels are reached the recommended operation procedures as described below should be executed immediately for the spillways and control structures, so that the right amount of gates is opened before the flood wave arrives at the structure. The working of the warning system can be found in Figure 71.

Table 57: Water levels at IDD1 for starting operation procedures

		Discharge[m^3/s]	Water Level [m]
Long rains	Maximum	230	4.5
	Minimum	45	1.5
Short rains	Maximum	75	2.0
	Minimum	15	0.5

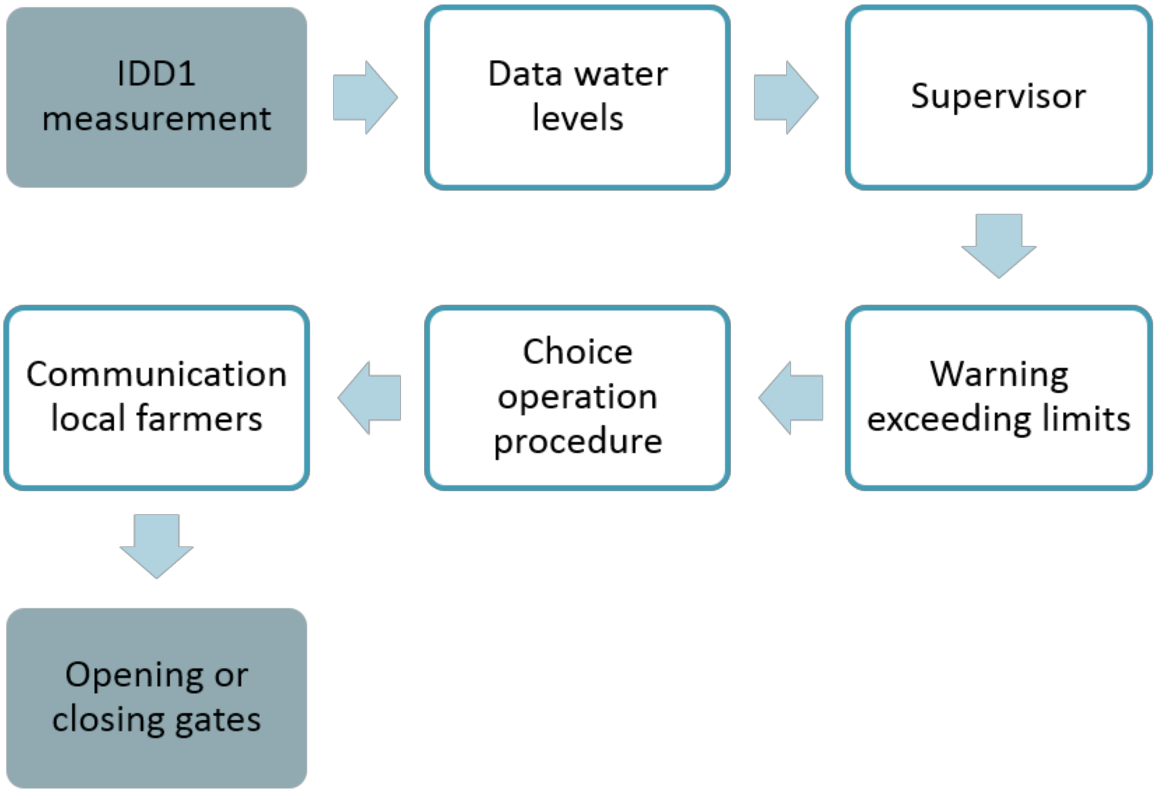

Figure 71: Working warning system

3.2 Spillways

The recommendations given below are based on estimations. It is possible that the model will change when a more accurate elevation map is present and this will change the opening times of the spillways. Furthermore, there is a very large variation in the duration and height of the rain peaks. The recommendations given below are for the extreme events and so the opening duration can differ between these extremes. It is thus recommended that the village leaders will communicate with each other. So that when the last village of the floodplain (Kirungu at the current elevation map) has flooded, the village leader of this village will contact the farmers at the spillways to tell them to close the spillways. The role of the supervisor is to oversee whether this communication is going well and to intervene when problems arise.

3.2.1 Long rains

During the long rains, it is wanted that the Samanga area behind the dike is flooded. Therefore, it is necessary to open the spillways so that water can flow through the spillways onto the Samanga area. The duration of the opening of the spillways is different for minimum and maximum discharge.

Minimum discharge

When there is not so much rain during the long rain season the discharge reaches as low as the bank full capacity of the river. With this minimum discharge, the spillways need to be opened for two days. This amount of days should be sufficient to flood the Samanga area and flush the soils, see Appendix B.2 - Design Calculations Spillways. It is important that the supervisor ensure that the spillways are opened this amount of days, otherwise the water will not flood the whole Samanga area.

Maximum discharge

When there is a lot of rain during the long rain season, the discharge exceeds the bank full capacity of the river. At this moment, it is sufficient to open the spillways for just one day in order to flood the Samanga area and to flush the soils.

3.2.2 Short rains

During the short rains, the flooding of the Samanga area is unwanted. This means that during the short rain season the spillways should be closed in order to prevent flooding. However, irrigation of the farming land can be wanted during the dry periods in the short rain season. In this case, the spillways can be opened so that the farmers can use the gates to pump water on their farming land.

3.3 Control Structure

The recommendations given below are based on estimations. It is possible that the model will change when data that is more accurate is present and this will also change the amount of opened gates in the control structure. Furthermore, there is a very large variation in the duration and height of the rain peaks. The recommendations given below are for the extreme events and so the amount of open gates can differ between these extremes. It is also possible to partly open the gates depending on the water level, however, it has to be determined based on experience when this would be required. Table 58 shows the water levels at IDD1 during the extreme events and the amount of gates that should be opened in the control structure, determined with the present available data.

Table 58: Amount of opened gates control structure

	Range Water Level IDD1 [m]	# Opened Gates Required [-]
Long rains	< 2.0	1
	2.0 - 4.8	2 - 3
	> 4.8	4
Short rains	< 1.2	1
	1.2 – 2.0	2 - 3
	> 2.0	4

3.3.1 Long rains

During the long rains, it is essential that the Ronga floods every year to ensure fertile lands. Consequently, different measures are needed at the control structure for minimum and maximum discharge.

Minimum discharge

When there is a minimum discharge, only one of the gates should be opened in order to ensure the wanted flooding of the Ronga area. When the Ronga area has flooded enough the other gates in the control structure can be opened so that more water will flow through the Kikuletwa South and the flooding of the Ronga will decrease.

Maximum discharge

When there is a maximum discharge all of the gates should be opened in order to ensure that the excessive amount of water is transported by the Kikuletwa South. When the discharge decreases one of the gates can be closed if it is still wanted for the Ronga area to flood.

3.3.2 Short rains

During the short rains the Ronga should not flood. During peak discharges of short rains, the Ronga cannot handle the discharge arriving at the bifurcation. This excess discharge should go to the Kikuletwa

South. However, when there is a low peak during the short rains there should be sufficient water flowing through the Ronga for irrigation.

Minimum discharge
During minimum discharge, it is required to open one of the gates so that a minimum discharge can enter the Kikuletwa South to avoid the running dry of the Kikuletwa South while the rest of the flow is going to the Ronga for irrigation purposes.

Maximum discharge
The Ronga may not flood so the excess water should flow into the Kikuletwa South. For that reason, it is necessary to open the middle two gates. When the discharge is very high there can be chosen to open the third and/or the fourth gate to ensure that more water is going into the Kikuletwa South to avoid flooding of the Ronga. The IDD1 warning system is necessary to make sure that enough gates will be opened to avoid flooding of the Ronga.

3.3.3 Dry season

In the dry season, there should be enough discharge through the control structure to prevent that the Kikuletwa South runs dry. Thus one of the gates should be opened so that a minimum discharge of $1 m^3/s$ can enter the Kikuletwa South.

21.5 Maintenance

In chapter 6 Validation, a time horizon for the project of 20-25 years is proposed. This means that for that time span, the design should function for the set criteria. Regular maintenance during the years is needed in order to ensure the longevity of 20-25 years of the structures. The maintenance can be divided into three parts: the rivers, the dikes and the structures. Each part of the solution needs a different maintenance plan.

Figure 72 shows an overview of the elements that should be in the maintenance plan. The recommended maintenance plans are further elaborated in the part below.

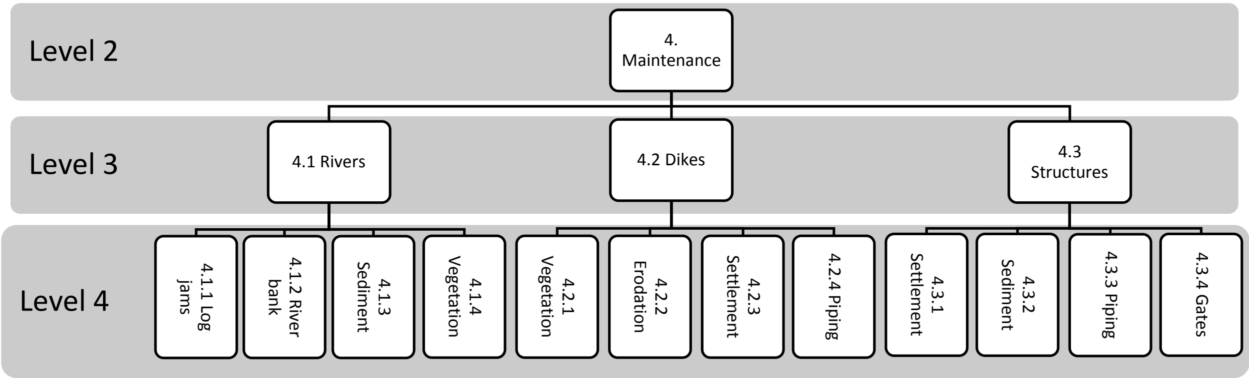

Figure 72: WBS maintenance phase

4.1 Rivers

4.1.1 Logjams – Local farmers

The fieldwork showed that the accumulation of trees during and after high water could cause erosion of the riverbank in a very short time span. It is therefore very important that the rivers are regularly checked for the presence of these kind of logjams. It is not enough to only observe the logjams, but if such a logjam is observed it is necessary that the trunks are removed within a few days. It is recommended that the farmers will be responsible for the observation and removal of the logjams, but a general supervisor from FTK will have to check if the farmers are acting by the agreements.

4.1.2 Riverbank – Local farmers

For the discharge capacity of the river, it is important to maintain the riverbanks. This means that after a heavy rain it should be checked that no parts are eroded. Therefore, it is advised that an inspection of the riverbank be performed simultaneously with the checking for the logjams. When eroding of the riverbank is identified, it is important that the riverbank be restored within a few days.

4.1.3 Sediment – TPC/supervisor

It is hard to predict beforehand what the morphologic effects of the changes to the river will be, see chapter 16 Morphological Effects. TANESCO does not want the Nyumba ya Mungu reservoir to silt up and so it is important to monitor the change in sediment transport in the river. This monitoring consists of checking the river depth twice per year by someone from TPC or the general supervisor of the project.

4.1.4 Vegetation – Local farmers

Maintaining the designed flow speed is essential for the discharge capacity of the rivers. The growth of vegetation in the rivers will lower the flow speed and this is unwanted. It is necessary to remove the vegetation that grows in the river during the dry season when the minimum discharge flows through the rivers. The removal of the vegetation can be performed by the local farmers. Here it is also important that there is supervision from FTK to make sure the farmers are acting by the agreements.

4.2 Dikes

4.2.1 Vegetation – Local farmers

Vegetation on the slopes of a dike is needed in order to prevent eroding of the slopes. Vetiver grass is the best sort of vegetation for a dike slope and it was advised by TPC to let the local farmers make nurseries for their own Vetiver grass. It is therefore important that the local farmers will maintain the Vetiver grass on the slopes after the planting and remove other vegetation such as banana plants and bushes.

4.2.2 Eroding – Local farmers/supervisor

Maintaining the designed slopes is essential for the stability of the dikes. The eroding of the slopes will decrease the stability of the dike and is thus unwanted. Regular checking of the dike slopes is needed in order to note eroding in time. When eroding is determined, it is necessary to repair the dike as fast as possible.

At the front of the spillways large amount of erosion is expected. Due to the choice for soft scour protection the eroded soil needs to be replenished as soon as possible. This can be done by local farmers.

4.2.3 Settlement – TPC

Uneven settlement of the dike could cause cracks and this is unwanted because it reduces the water retaining function of the dike. The height of the dike should be monitored in order to notice settlements of the dike. If significant settlement is observed, the dike should be heightened. The monitoring can be done by the TPC surveying crew.

4.2.4 Piping - Supervisor

Piping is the process of pipe formation under river dikes. During high water levels, the piping process manifests itself by the formation of sand boils landwards of the dike. It is important to check regularly for these sand boils in order to notice them before the critical piping length that causes failure has occurred. When piping occurs, there is not much that can be done. Therefore, it should be prevented by taking measures in the design.

4.3 Structures

4.3.1 Settlement – Supervisor/engineer

Uneven settlement of the structures can cause cracking of the concrete. Cracking of the concrete is unwanted because water can enter the concrete through the cracks and can cause corrosion of the reinforcement. It is therefore important to survey the level of the structures and check the concrete for cracks. When cracks are seen it is necessary to fill these cracks before water can enter the crack. The checking and surveying of the structures should be done by someone who understands the structures; such as the supervisor, a contractor or preferably an engineer.

4.3.2 Sediment – Local farmers

Sediment can cause silting up of the structures and this decreases the discharge capacity of the structures. Thus, it is important to regularly remove the sand and or waste that has been collected in the structure in order to maintain the designed discharge capacity. The removal of sand and waste can be performed by the farmers who also manually operate the gates.

4.3.3 Piping – Supervisor

Piping is the process of pipe formation under river dikes. During high water levels, the piping process manifests itself by the formation of sand boils landwards of the structure. It is important to check regularly for these sand boils in order to notice them before the critical piping length that causes failure has occurred. When piping occurs, there is not much that can be done. Therefore, it should be prevented by taking measures in the design.

4.3.4 Gates – Supervisor/engineer

The manually controllable gates are important because they regulate the amount of water that is flowing through the structure. It is important that the gates be checked between the rain periods for irregularities. Irregularities may cause the doors to not close well anymore or that they eventually cannot be opened or closed at all. When any irregularities are observed it is necessary that the gates are repaired or even replaced before the beginning of the next rainy period. The inspection of the gates is preferably performed by the supervisor or an engineer, because they have the required knowledge to observe unwanted irregularities.

22 Conclusions Risks and Implementation

Based on the identified risks and the suggested implementation plan several conclusions can be drawn.

Due to the uncertainty and limitation of the available data used for the design, additional steps need to be taken before the current design can actually be used. First of all, more vertical and horizontal surveys need to be done of the project area. Secondly, an external engineering company will need to validate and further detail the design before construction drawings and a bill of quantities can be made.

To ensure the quality of the project, a project management team should be created in the preparation phase for the duration of the project. The team will supervise the work done by the external companies to ensure no corners are cut and to ensure no information is lost between the project phases. For the selection of the contractors, it is advised to divide the work in packages and tender it among contractors with a good reputation or to grant it to known contractors. It is suggested to use a fixed price contract as the work is of low complexity and familiar to local contractors.

It is important to educate the local farmers about the solution to ensure they will increase the use of their lands and help with the maintenance and operation of the solution. To guarantee proper operation and maintenance of the solution, a supervisor should be hired to oversee these activities. Furthermore, a warning system in IDD1 should be created to provide sufficient time for the control structure to be configured.

In case the Msitu wa Tembo region decides to take measures against the flooding on their side of the Kikuletwa North, this will negatively impact the effectiveness of the solution.

Part F – Conclusions and Recommendations

In the following chapter, the conclusions that can be drawn from the preceding report are stated. These are followed by the limitations of these conclusions. Finally, recommendations are given for the next steps.

Figure 73: Picture taken during fieldwork, riverbed of Kikuletwa South

23 Conclusions

The goal of this project was to improve the welfare in the Lower Moshi area by developing a technical solution that prevents the short rain flooding and regulates the long rain flooding which is socially acceptable, feasible, and durable. The following conclusions can be drawn from the preceding report concerning the goal of the project.

23.1 Analysis

In the analysis phase was concluded that the design of the prefeasibility study had to be changed. The following decisions were made:

- Three return periods for discharges were defined, 1/5, 1/10 and 1/15 years.
- Two clusters from the prefeasibility design will be used, these are:
  - *Cluster 1:*
    - A dike along the Kikuletwa North including passive spillways.
  - *Cluster 2:*
    - A passive control structure in the Kikuletwa South, just after the bifurcation.
    - Widening the narrow part of the Kikuletwa South.
- Passive structures cannot handle the peak variations in discharges and therefore need to be reconsidered.
- The soil was observed to be silt during field visits; this should be included in the design.

23.2 Initial Design

In the initial design, three designs were made based on the return periods. All designs were capable of preventing the short rain floods while controlling most of the long rains floods depending on the used return period. Each design had varying dimensions depending on the return period. It was concluded that the only option was to design manually controllable structures for both the spillways and the control structure; therefore, the designs consisted of the following elements:

Cluster 1:

- A dike along the Kikuletwa North including manually controllable spillways.

Cluster 2:

- Widening the narrow part of the Kikuletwa South.
- A manually controllable control structure in the Kikuletwa South, just after the bifurcation.

Various interviews were held with local contractors and engineers to get insight into the ability to build the design and the associated costs. These concluded that the design looked realistic to build with local available materials and equipment.

It was decided to continue with the 1/15 year solution after a cost benefit analysis had been conducted and a progress meeting with the stakeholders had been held.

23.3 Integral Design

In the integral design**,** the 1/15 year solution was worked out into more detail. Several changes and additions were made:

Cluster 1:

- A dike along the Kikuletwa North including manually controllable spillways.
  - Soft scour protection is needed before and hard scour protection after the spillways.
  - Guiding dikes are necessary along the path of the spillways behind the dike.

Cluster 2:

- Widening the narrow part of the Kikuletwa South.
  - A dike should be built on the eastern side of the Kikuletwa South.
  - The location was changed; The connection with the bifurcation has moved to the east side of the current Kikuletwa South Small
- A manually controllable control structure in the Kikuletwa South, near the bifurcation.
  - Hard scour protection is needed before and after the control structure.
  - A swivel crane with a hand chain hoist needs to be placed on the control structure.
  - The location was changed; The connection with the bifurcation has moved to the east side of the current Kikuletwa South Small

A morphology study for the prevalent discharge in the dry season was conducted. From this study can be concluded that most riverbeds will erode over time. This increases the capacity of the rivers, which can be seen as an additional benefit. Only the braided part of the Ronga experiences sedimentation. However, it is not likely that this will cause problems, because the floods will flush away accreted sand. No increase in sediment entering the reservoir is expected.

The costs and benefits were determined for the entire lifetime of the project. The costs of this design were determined to be 4.5 million US dollars, with an estimated range between 3.8 and 6.2 million US dollars. The present value of these costs is 3.5 million US dollars. The benefits of the design were estimated at 36 million US dollars with a present value of 11.5 million US dollars. The benefits combined with the costs results in a net present value of 8 million US dollars for the design and the associated cost benefit ratio of 3.23 with an internal rate of return of 22.8%.

23.4 Risks and Implementation

Based on the identified risks and the suggested implementation plan several conclusions can be drawn for the different project phases.

- Horizontal and vertical surveying of the project area is required.
- An external engineering firm will need to make the construction drawings and bill of quantities.
- A project team should be created in the preparation phase for the duration of the project.
- The work should be divided into packages and tendered or granted to contractors with a good reputation to ensure the quality.
- In no situation cluster 1 should be built before cluster 2.
- The local farmers need to be educated to ensure an increased use of the farmland and support for the operation and maintenance of the structures.
- A warning system should be created to provide sufficient time for the control structure and spillways to be configured.
- In case the Msitu wa Tembo region decides to take measures against the flooding on their side of the Kikuletwa North, this will negatively impact the effectiveness of the solution.

24 Limitations

The conclusions drawn in this report have significant limitations, which will be described here. These limitations can have a substantial impact on the success of the project and should be taken into serious consideration.

24.1 Preparation

There are many uncertainties in the data gathered and the assumptions made for the area. The data gathered was only for a few locations, and assumptions were made for the remaining stretches. The assumptions made for the profile of the river, profile of the floodplains, bed roughness and slope of the rivers and floodplains and various other components were used for the calculations throughout the report. If these assumptions are very different from the reality, it will have a significant influence on the safety and accuracy of the design as well as the cost estimation.

Many assumptions have been made to determine the costs. For one, they depend on unit rates given by local contractors and engineers who may have an interest in giving optimistic rates as they might hope it would increase their chances on being granted the work. Additionally, the cost of clearing the vegetation forms a major part of the construction cost, which is based on satellite imagery. This estimation is rough at best.

The estimation for the maintenance and operation expenses is based limited data. The local contractors and engineers spoken to had little information or experience with the associated costs.

For the benefits, the precise area that is affected by the solution is unknown due to inaccurate height maps. Likely the affected area is larger than that is currently assumed as in a far larger area farming takes place in the project area.

The inflation rate has been based on historic data while this gives no guarantee for the future. This has consequences for all estimated cash flows if this changes.

The calculations made in this report were approached in an academic way. The team performing the current study has limited working experience and the methods used could be inaccurate as a result of this.

The team performing the current study will not be present during the construction or operation of the project. The design needs to have a responsible party so that when construction works are initiated any issues regarding the design can be addressed. Additionally, a party needs to hold responsibility of the design.

The approval of the project by Pangani Basin Water Board is necessary for the project to take place.

It is important that the design will be constructed as it has been designed. The decision on which the contractor will perform the work is essential in this case. Additionally, proper supervision and the type of contract are important factors, which could limit the quality of work. Contractors can cut corners to save on costs. They could start using cheaper quality materials, perform marginal workmanship or extend the completion date to reduce costs.

24.2 Construction

The order of construction needs to be considered carefully, this is because if cluster 1 is built first it could have devastating impacts on the areas downstream of cluster 1, if the structures of cluster 2 are not yet present.

24.3 Operation and Maintenance

After the construction of the solution, it is important that the gates in the structures are operated in order to ensure the wanted flooding during the long rains and prevent the unwanted flooding during the short rains. During operation and maintenance, collaboration and communication between the villagers, the farmers and FTK is very important for the solution to work. If operation and maintenance are not performed as they should it could lead to severe damage or failure of the structures.

A warning system is necessary for the controllable structures to operate as they have been designed. Due to lack of knowledge on how long it will take to change the configuration of the gates and how long it takes a flood peak to arrive at the spillways and control structure it is not known how frequently the water level should be measured. Twice a day, as it is currently measured, is probably insufficient as the flood peaks travel quite rapidly and it will take some time to change the configuration of the gates once the warning signal has been distributed. The flood wave could arrive at the structures before they have been configured properly, thus making the design not work as it should.

24.4 External

The design should take into account that unforeseen circumstances may have influence on the design. For instance, extreme weather conditions and afforestation/deforestation in the Kilimanjaro catchment could considerably affect the incoming discharge or morphology of the river.

It has been assumed that Msitu wa Tembo does not mind their plains being flooded. However, if they change their minds this could have significant consequences.

Lastly and most importantly, the farmers could distrust the solutions and as a result do not cultivate as many crops as they could.

25 Recommendations

The following recommendations follow from the earlier drawn conclusions and limitations and provide a guide to the next steps that should be taken to fully realise the project.

25.1 Preparation

To reduce the uncertainties in the gathered data and assumptions made, it is recommended to survey the area, both horizontally and vertically. This needs to be done to determine the profiles of the river, get an accurate elevation map and to determine the amount of vegetation. Additionally, it is advised to do more extensive soil testing.

To ensure the calculations made are done accurately it is advised to have an engineering company validate them. This engineering company should include the data gathered from the surveying and soil testing. They should produce a detailed design and a bill of quantities. Moreover, an engineering company is necessary, as there needs to be a party responsible for the design since the team performing the current study will not be present during the realisation of the project.

For the approval of the project, it is essential to keep Pangani Basin Water Board in the loop of all activities.

It is advised that for clearing of the area the local community be consulted upon as well as TPC building department. Their participation in clearing vegetation could significantly reduce the costs.

More local contractors and engineers could be approached to validate the used unit rates and to get a better estimation of the required work for maintenance.

For the selection of a contractor, it is advised to grant the work to different contractors, as there are different kind of solutions and each contractor has their speciality. Therefore, the work should be divided into packages. It is advised to grant or tender the packages to contractors with a good. This will reduce the chance that the design quality is not met, although it will likely increase the costs.

It is advised to use a *"Fixed Price"* contract for all packages as the work is relatively straightforward and contractors have experience with the type of work. In order for the construction to meet the design standards, a communication network between the design team and the contractor is necessary. Additionally, reviews and checks during construction process by the design team should take place.

It is advised that from the start a project team be set up for the duration of the preparation and construction phase. Later on the knowledge gained during these phases should be transferred to the operation and maintenance phase. This team should supervise the work done by the engineering company and contractors to check if things go, as they should.

25.2 Construction

It is recommended that either cluster 1 and 2 are constructed at the same time or that cluster 2 is constructed first and then cluster 1.

25.3 Operation and Maintenance

To ensure that the structures are properly operated and maintained, clear operation and maintenance plans should be made and the communities need to be educated on these. There needs to be a clear division of responsibility for the operation and maintenance of the structures. Furthermore, a person(s) needs to be appointed to be in charge of the warning system, the control structure and the spillways to determine if they are used properly in order to achieve the desired effects. There should be checks to ensure that the operation and maintenance are being done appropriately.

For the operation to work successfully a warning system needs to be present. It is therefore advised, when accurate data has been collected through surveying, to determine how long it takes for a flood wave to arrive at the different locations. In addition, it should be determined how long it takes to configure the gates of the control structure. Based on these things a warning system can be designed, including how many times the water level should be measured per day. Based on the data the supervisor can make a choice about the opening configuration of the gates in the spillways and the control structure. The supervisor needs to communicate his decision to the local farmers, which then can open or close the gates before the flood wave has arrived.

25.4 External

To account for unforeseen circumstances, it is advised to design flexible and robust structures.

To make sure that the Msitu wa Tembo does not take drastic measure to counteract the planned construction, they should be kept informed on the advancements of the project.

For the success of the project as a whole, it is desired that the local communities are educated properly on the solutions so that they can optimize their cultivation of crops.

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Appendix A – Analysis

Appendix A.1 - Validation of the Prefeasibility Design

The validation of the prefeasibility study for the criteria is described in this appendix. In chapter 6 Validation, the conclusions were presented of this validation. First, the criteria that affect the entire design will be discussed; hereafter the relevant criteria for each individual cluster will be explained.

Total Design

Risks

The risks of the design and project have not specifically been taken into account. These will need to be identified as they can have an impact on the design.

Cost Effectiveness

In the prefeasibility study, no analysis was made for the cost and benefits for the different return periods of the floods. It is therefore not known what return period for the design would be most optimal.

Morphological Effects

The long-term morphological effects have not been taken into account. These will need to be studied and the design will be evaluated based on the findings.

Maintenance

This has been briefly touched in the prefeasibility report, however, it will need to be further determined what will be the best ways to maintain the system and keep it functioning. The impact of trees floating in the river on the structures, or the loss of land before the dikes are examples of matters that need to be taken into account.

Longevity

No attention has been paid to what happens with the dike/structures in case the design discharge is exceeded. This affects both the longevity of the design as well as the required maintenance to restore potential damage.

Cluster 1: Samanga Dike and Spillways

Water Management of the Agricultural Land

The spillways have been designed to allow flooding of the area behind the dike. However, irrigation of the land behind the dike and the drainage of the land have not been taken into account.

Locations for the Designs

The locations for the dike and spillways have been very roughly estimated. The locations will need to be defined more accurately, before definitive locations can be chosen.

Resources and Construction Method

The designs have a low complexity of execution. For the earthworks, the local resources might not be available as was designed. During the meeting with a local contractor, doubts were expressed about the available soil types. Furthermore, for the equipment to compact the soil of the dike at least 2.5 meters' width of the crest of the dike is required.

Operation

The solution of the prefeasibility was specifically designed to operate without constant attention. However, this makes the flow of water from the river to the land behind the dike uncontrollable. A more thorough study is required to determine if this is the best solution. Especially since manually operated spillways are already used in the area.

Longevity

The spillways were designed to be adaptable to changing circumstances. Therefore, the longevity of the spillways is acceptable. For the dike too much is still unknown.

Cluster 2: Kikuletwa South Small and Control Structure

Risks

The solution allows extra discharge to flow through the Kikuletwa, which might have an impact on the river crossing downstream. It will need to be evaluated if the impact is worth the benefits.

Water Management of the Agricultural Land

At first glance, there will be no influence on the drainage or irrigation.

Locations for the Designs

During fieldwork, questions were raised about the planned location for the control structure. There would need to be sufficient space for the construction of the structure and good accessibility.

Operation

For the control structure, the question arises if it will always function as intended with the distribution of the river discharge, as it is not easily adaptable to the changing discharges in the river during the year.

Longevity

The control structure has an option to be adaptable on the long-term. For short-term changes it has no option to quickly adapt.

Cluster 3: Ronga River Dikes

Water Management of the Agricultural Land

No attention has been paid to the impact of the Ronga river dikes on the drainage or irrigation.

Locations for the Designs

The locations of the dikes depend on the location of control structure of the Chem Chem channel. This implies that if the connection between the Ronga and the Chem Chem channel is further away the dikes must be longer.

Construction

The soil types will need to be revaluated for the design of the dikes.

Longevity

The original design height of the dikes is 25 cm; it can be argued that the local farmers will not have enough respect for this height. It might be possible that farmers will walk over the dike and thus damage it. This is also related to the maintenance.

Cluster 4: Chem Chem Channel and Control Structure

Water Management of the Agricultural Land

The river will provide extra water and drainage capacity for the area.

Locations for the Designs

During the first fieldtrip, it was observed that the planned location of the new channel is not possible. A village is located at the start of the route of the channel, requiring a new location to be found for the new channel.

During the fourth fieldwork day, it was observed that the braided part of the Ronga seems different than was described in the feasibility report. Furthermore, the old Chem Chem riverbed is wider, which can cause problems with having sufficient flow velocity in the channel. Additionally, the height of the Chem Chem riverbed compared to the Ronga is unknown. It may be possible that there is insufficient height difference for natural flow. Another possible problem is that the old Chem Chem riverbed passes through the middle of a town, which will be effected once water starts flowing through the river.

Construction

The structure and channel can be built with locally available equipment and materials.

Operation

The cluster would be easily operable.

Longevity

The control structure has an option to be adaptable on the long-term. For short-term changes it has no option to quickly adapt.

Appendix A.2 - Weight Criteria and MCA Scores

The reasoning behind the scores and the weights for the different criteria will be elaborated here. An alternative was created because of the problems that were identified in the validation of the prefeasibility design. The scoring for the options was done by each of the project team members and later combined to get a final score. The scores were awarded on a scale from one to five, with one being the lowest and five the highest. These scores for each criterion were then multiplied by their respective weights. The weighs and criteria are described below.

Weights Criteria

The weights of the criteria are on a scale from one to five. Five is the most important and one the least important.

Weight 5

Risks

One of the goals is to reduce the number of uncertainties and possible risks for this project. It is important to reduce these as much as possible, within the limited time that is available.

Locations of the Design

The location will have a large impact on the cost for the design as well as the side effects of it. At this moment, there are roads, river crossings and large number of farms around the places where things need to be built. It is thus important that the location is realistic and does not create too many problems.

Water Management of Agricultural Land

It is important that the irrigation, drainage and floods can be managed by the options. These three subjects form the corner stone of the project, and the goal is to improve the situation regarding these three elements.

Weight 4

Resources and Construction Method

Building should be possible with the equipment/resources that are locally available. The same holds for the construction method as what is possible in the Netherlands might not always be possible here. However, it is not as important as the risks or locations as they have more impact on the outcome of the project. It is easier to change material type than a location.

Weight 3

Maintenance

The amount of effort it takes to maintain both options can have significant long-term effects for the effectiveness of the solution. However, the elements of both options are similar, thus the work would be the same and only the effort would differ. This is why it gets a lower weight.

Weight 2

Cost effectiveness

Both options will need to be adjusted to take into account the different return periods. For the moment it will be limited to the ease of which it can be adjusted and the associated benefit.

Weight 1

Morphological Effects

At this moment there is little to say about both options for the morphological effects, this will be taken into account during a later stage of the project. An educated guess could be made at best.

Operation

Operation is at this moment of little importance for the decision which option is better.

Longevity

At this moment there is little to say about both options for the longevity, this will be taken into account during a later stage of the project.

Option 1: Prefeasibility Design

Risks

There are still a lot of uncertainties for this alternative. It is not known if the Ronga will have sufficient discharge in the braided part to guarantee sufficient flow in the new riverbed. Furthermore, the river will go through a village, creating a chance that the village will flood. Additionally, it is not known if the height of the riverbed of the Chem Chem is lower than the Ronga. Finally, the southern connection between the old riverbed and the Ronga is not designed yet, it is uncertain what to expect there.

Cost Effectiveness

At several locations, adjustments would need to be made to handle different return periods. As the solution is more complex, it will cost relatively more, for the same benefits when compared to the alternative design.

Water Management of Agricultural Land

This solution will make irrigation in the Chem Chem area easier, as well as drainage, this is a positive effect.

Locations of the Design

The option will go through a village, it is not an easy construction location to reach and it will create an extra barrier in the Chem Chem area that is hard to cross when high water levels occur.

Resources and Construction Method

There would be six building locations required for this alternative, making it more complex. However, similar structures will be built, making it easier to repeat.

Morphological Effects

At this point it is hard to determine the effects, it is estimated that this solution will have more effect on the long-term as there will be more changes along the river.

Operation

Since this solution consists of more elements, it will be harder to operate. The elements are similar though and require no active operation.

Maintenance

An elaborate maintenance plan would be required for this option as it covers a large area and it has many elements.

Longevity

In the design, no attention was paid to the effects of discharges that exceed the design discharges or to the design lifetime.

Option 2: Alternative Design

Risks

There would be an increased chance of flooding of the Kikuletwa South area. Furthermore, there is a chance that the river crossing of the Kikuletwa South will be unavailable more often due to increased discharge.

Reliability

This solution would be easily adaptable for different return periods for the discharges. The benefits for both options will stay the same, while the cost should be lower because fewer elements are present and less area is required.

Water Management of the Agricultural Land

There will be no impact on irrigation or drainage.

Locations of the Design

This option will have fewer building sites, making the logistics easier. Furthermore, no extra barriers will be created by the construction.

Resources and Construction Method

There would be three building locations for this solution. The control structure at the bifurcation will be larger, increasing the complexity.

Morphological Effects

It is expected that this solution will have less impact than option 1, as there are fewer locations where the design will influence the river.

Operation

Fewer elements will be part of the solution, making it easier to manage.

Maintenance

There will be fewer elements and a smaller area to maintain, however, the control structure will be larger and more complex to maintain.

Longevity

In the design, no attention was paid to the effects of discharges that exceed the design discharges or to the design lifetime.

MCA Scores

In Table 59 and Table 60 the scores for the different options for the criteria can been seen. Option 1 scores **2.6** for the MCA, while Option 2 scores **3.3**. It can be concluded that according to the MCA option 2, the alternative design, is a better option and it shall be used for the project.

Table 59: MCA of Option 1

Option 1	Weight factors	Percentage [%]	Project group					Weighted average
			K	M	A	O	W	
Risk	5	19	3	3	2	2	1	**0.4**
Cost effectiveness	2	7	3	2	3	2	3	**0.2**
Water management of the agricultural land	5	19	5	4	5	5	5	**0.9**
Locations of the designs	5	19	2	1	2	1	1	**0.3**
Resources and construction method	4	15	2	2	2	3	2	**0.3**
Morphological effect	1	4	2	3	2	3	2	**0.1**
Operation	1	4	2	4	3	4	3	**0.1**
Maintenance	3	11	2	2	2	2	2	**0.2**
Longevity	1	4	3	4	4	3	3	**0.1**
Score	**27**	**100**						**2.6**

Table 60: MCA of Option 2

Option 2	Weight factors	Percentage [%]	Project group					Weighted average
			K	M	A	O	W	
Risk	5	19	4	3	3	3	4	**0.6**
Cost effectiveness	2	7	4	3	4	3	3	**0.3**
Water management of the agricultural land	5	19	2	2	3	2	2	**0.4**
Locations of the designs	5	19	4	5	4	4	4	**0.8**
Resources and construction method	4	15	4	4	3	3	3	**0.5**
Morphological effect	1	4	2	3	2	3	3	**0.1**
Operation	1	4	3	2	3	3	2	**0.1**
Maintenance	3	11	3	4	3	3	4	**0.4**
Longevity	1	4	3	3	4	3	3	**0.1**
	27	**100**						**3.3**

Appendix A.3 - Current River Dimensions

Here the river data collected in the prefeasibility study will be summarised. This will be done per stretch; the stretches are defined in chapter 7 The Rivers. For every stretch, the most important parameters will be included in this report. For more details, the reader is referred to the prefeasibility report, appendices E and F.

Stretch 0: IDD1

In Table 61 the most important dimensions of stretch 0 are given.

Table 61: Dimensions stretch 0

Width top [m]	32.0
Width bottom [m]	28.0
Total depth [m]	4.0
Bed slope [-]	0.0011
Roughness coefficient [$s/m^{1/3}$]	0.05

For more details, see cross-section J. (Figure 150 appendix E prefeasibility report)

Stretch 1: Kikuletwa (N)

In Table 62 the most important dimensions of stretch 1 are given.

Table 62: Dimensions stretch 1

Width top [m]	25.0
Width bottom [m]	24.5
Total depth [m]	2.0
Bed slope [-]	0.0011
Roughness coefficient [$s/m^{1/3}$]	0.05

For more details, see cross-section Kikuletwa N near dike breach. (Figure 44 prefeasibility report)

Stretch 2: Kikuletwa (S) Small

In Table 63 the most important dimensions of stretch 2 are given.

Table 63: Dimensions stretch 2

Width top [m]	3.0
Width bottom [m]	2.25
Total depth [m]	2
Bed slope [-]	0.0016
Roughness coefficient [$s/m^{1/3}$]	0.05

For more details, see cross-section H. (Figure 148 appendix E prefeasibility report)

Stretch 3: Kikuletwa (S) Large

In Table 64 the most important dimensions of stretch 3 are given.

Table 64: Dimensions stretch 3

Width top [m]	26
Width bottom [m]	8
Total depth [m]	4.8
Bed slope [-]	0.0016
Roughness coefficient [$s/m^{1/3}$]	0.04

For more details, see cross-section G. (Figure 147 appendix E prefeasibility report)

Stretch 4: Ronga (N) Before Braiding

In Table 65 the most important dimensions of stretch 4 are given.

Table 65: Dimensions stretch 4

Width top [m]	10
Width bottom [m]	1
Total depth [m]	4.8
Bed slope [-]	0.0014
Roughness coefficient [s/m$^{1/3}$]	0.05

For more details, see cross-section A. (Figure 141 appendix E prefeasibility report)

Stretch 5.1: Ronga (N) Braiding 1

In Table 66 the most important dimensions of stretch 5.1 are given.

Table 66: Dimensions stretch 5.1

Width top [m]	9
Width bottom [m]	7.8
Total depth [m]	1.4
Bed slope [-]	0.0014
Roughness coefficient [s/m$^{1/3}$]	0.05

For more details, see cross-section D. (Figure 144 appendix E prefeasibility report)

Stretch 5.2: Ronga (N) Braiding 2

In Table 67 the most important dimensions of stretch 5.2 are given.

Table 67: Dimensions stretch 5.2

Width top [m]	13
Total depth [m]	1.8
Bed slope [-]	0.0014
Roughness coefficient [s/m$^{1/3}$]	0.05

For more details, see cross-section E. (Figure 145 appendix E prefeasibility report)

Stretch 6: Ronga (S)

In Table 68 the most important dimensions of stretch 6 are given.

Table 68: Dimensions stretch 6

Width top [m]	15
Total depth [m]	3.2
Bed slope [-]	0.0011
Roughness coefficient [s/m$^{1/3}$]	0.05

For more details, see cross-section C. (Figure 143 appendix E prefeasibility report)

Appendix A.4 - Design Discharges

This part will treat the design discharges that enter the project area from the North. Different discharges will be calculated for the long and short rains.

Long Rain Peak Discharges

To determine the discharge for a certain return period the highest discharge of every year is recorded in a graph, see Figure 74. There are 36 years of records. From this graph, the discharges corresponding to set return periods can be determined. In Table 69, the results are displayed. For the discharge corresponding to the 1/15 return period a higher return period would be possible, for instance 1/25 years. However, it would be incorrect to use this return period in this project, because of the limited amount of data available. This can be explained by presuming a new measurement in the coming year that exceeds the set 230 m^3/s. When this occurs the return period that was assumed to be 1/25 years will suddenly become much less, approximately 1/17 years. Therefore, a safe 1/15 years will be used for this project.

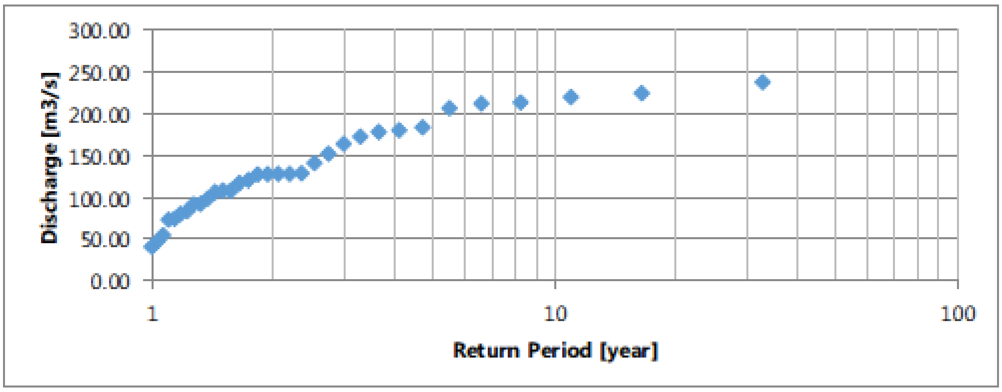


Figure 74: Return period of the annual maxima measured at IDD1 (3) figure 39

Table 69: Design discharges long rains, determined from the annual maxima graph

Return period [years]	Design discharge [m^3/s]
1/5	190
1/10	220
1/15	230

An alternative way is to determine the return period using equation 2 (Ankum, 2002).

$$T = \frac{N+1}{m} \tag{2}$$

Where: - T is the 'estimated' return period, in years
- N is the number of years recorded,
- m is the order number of the event, where $m = 1$ for the largest value.

Using this equation, the return periods for the discharges mentioned before is almost the same as when read from the graph in Figure 74. The results can be seen in Table 70. The only difference is the return period of 1/15 years, with the formula 1/18 years. The choice for 1/15 years, can be explained by using the same reasoning as before. If there is a year with a higher discharge the return period becomes much lower i.e. 38/3 ≈ 13 years. Therefore, a value between 1/13 and 1/18 is chosen, resulting in 1/15 years as indicated in the table.

Table 70: Design discharges long rains, determined with the return period equation

Design discharge [m^3/s]	Return period, from equation [years]	Return period [years]
190	37/7 ≈ 5	1/5
220	37/4 ≈ 10	1/10
230	37/2 ≈ 18	1/15

Lowest Peak Discharge during the Long Rains

To determine the lowest possible peak discharge during the long rains, Figure 74 is used again. Here the discharge that is exceeded every year must be determined. This is the discharge on the left of the graph at the 1 year return period. This indicates that this discharge is exceeded every year for as long as the measurements are recorded in the graph. The discharge that will be used is set at 45 m^3/s.

Short Rain Design Discharges

In figure 41 of the prefeasibility study a bar graph is presented, indicating the short rain annual maxima, see Figure 75. Using this bar graph a new graph can be plotted that is similar to the graph used for determination of the long rain return period. Only there are fewer points available because the number of points is equal to the number of bars in the bar graph.

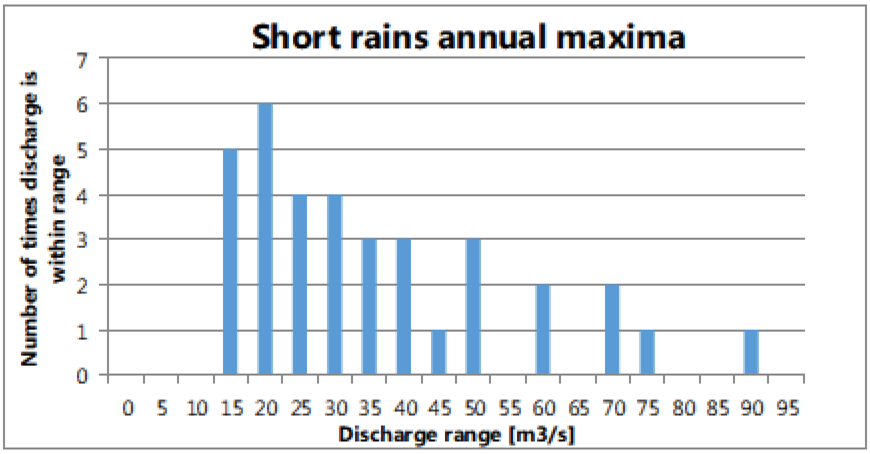


Figure 75: Bar graph of the short rains annual maxima, **from the prefeasibility study figure 41 (Lower Moshi (2015))**

In Figure 76 the graph is shown, that is used to determine the discharges corresponding to the set return periods.

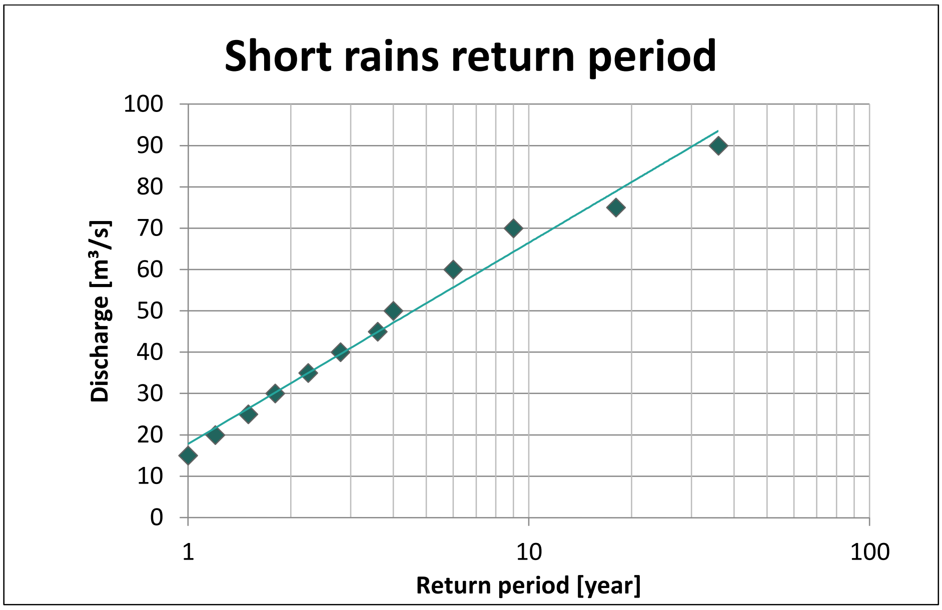


Figure 76: Short rains return period, plotted using the bar graph

In Table 71, the results are displayed. The two different methods to determine the long rain return period showed the same results. Therefore, only the first method is used to determine the short rain return period, assuming that the results of the second method will be the same.

Table 71: Design discharges short rains

Return period [years]	Design discharge [m^3/s]
1/5	52
1/10	68
1/15	75

Appendix A.5 - Design Discharge Cluster 1

Manning's Equation

The discharge for the river and the floodplain are formulated using Manning's equation (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015). Rewriting the equation results in equation 3:

$$Q = A \frac{R^{\frac{2}{3}}\sqrt{i_b}}{n} \tag{3}$$

Where: - Q is the discharge [m^3/s]
- A is the cross-sectional area [m^2]
- R is the hydraulic radius [m]
- n is the Manning roughness [$s/m^{1/3}$]
- i_b is the slope of the floodplain/river [-]

Approach

For both the river and the floodplain, the flooding height is unknown and equal. Furthermore, the design discharge is known for 1/5, 1/10 and 1/15. Solving these equations thus results in a design flooding height and the distribution of discharge between the river and the floodplain, see equation 4.

$$Q_{river} + Q_{floodplain} = Q_{design} \tag{4}$$

Assumptions

For the discharge of the river, the following assumptions are made. The data is taken from the prefeasibility study (Lower Moshi (2015)):

- The width is uniform along the river; 25 meters.
- The banks are 2 meters high and vertical.
- The bed slope is 0.0011.
- The roughness coefficient is 0.05 $s/m^{1/3}$.

For the discharge capacity of the floodplains, the following assumptions are made:

- The floodplain has a length of 300 meters.
- The floodplain has an average capacity depth of 0.25 meters.[1]
- The bed slope is 0.0011.
- The roughness coefficient is 0.07 $s/m^{1/3}$.

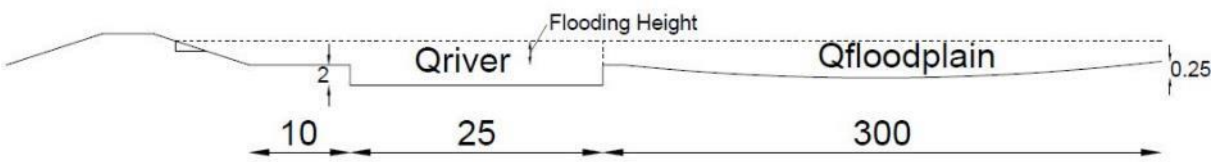


Figure 77: Schematization flooding

Additionally, the dike is assumed to be situated 10 meters away from the river. The flooding is schematized in Figure 77. The flooding height is the height of flooding above the riverbanks.

[1] The maximum depth is 0.5 m, therefore an average for the whole stretch is taken as 0.25m.

Resulting Discharges

Long Rains

The resulting flooding heights for cluster 1 are listed in Table 72.

Table 72: Design flooding heights cluster 1 long rains

	Q (1/5)	Q (1/10)	Q (1/15)
Flooding height [m]	0.65	0.76	0.79

This design height of flooding for the long rains for each design discharge is the starting point for the design height of the dike. The resulting discharge distributions resulting from these are listed in Table 73.

Table 73: Design discharge distribution long rains

	Q (1/5)		Q (1/10)		Q (1/15)	
	River	Floodplain	River	Floodplain	River	Floodplain
Discharge [m^3/s]	71	119	77	143	79	151
Distribution [%]	37	63	35	65	34	66

Short Rains

The resulting flooding heights for the short rains are listed in Table 74.

Table 74: Design flooding height short rains

	Q (1/5)	Q (1/10)	Q (1/15)
Flooding height [m]	-0.0085	0.0988	0.1407

The design flooding height for the discharge of 1/5 years is negative because the floodplain has enough capacity to absorb the flood. The resulting discharge distributions are listed in Table 75.

Table 75: Design discharge distribution short rains

	Q (1/5)		Q (1/10)		Q (1/15)	
	River	Floodplain	River	Floodplain	River	Floodplain
Discharge [m^3/s]	39	13	43	25	45	30
Distribution [%]	74	26	64	36	60	40

Uncertainties

Variations are likely because many assumptions are made. The variations that have the largest impact on flooding height and discharge distribution are the cross-sectional area of the river, the cross-sectional area of the floodplain and the slope of both. The width and depth of the river are likely not uniform, and observations during field visits revealed the river to be narrower rather than wider. A change in width and/or depth of the river would not have significant effects on the height of the flooding. It would affect the discharge distribution however; the discharge entering the floodplain will increase in case of a smaller cross-sectional area of the river or decrease in the case of a larger area. A larger floodplain cross-section would have positive impacts on the flooding height, as it will decrease. A decrease of the floodplain cross-section however, could significantly increase the flooding height and distribution of discharge. The slope also has a major impact on the resulting values; a less steep slope would increase to flooding height and decrease the discharge in the river significantly. A higher slope would have the opposite effect. In Table 76 the assumed values and expected deviations are listed.

Table 76: Expected deviations

	Assumed value	Expected deviation
River width [m]	25	10
River depth [m]	2	0.5
River roughness [$s/m^{1/3}$]	0.05	0.005
Floodplain length [m]	300	200
Floodplain depth [m]	0.25	0.25
Floodplain roughness [$s/m^{1/3}$]	0.07	0.005
Slope [-]	0.0011	0.001

These variations lead to the minimum and maximum values for the long rains, listed in Table 77. Only the effects on the long rains have been calculated, as these are normative for the design of the dike.

Table 77: Variations flooding height

	Q (1/5)	Q (1/10)	Q (1/15)
Flooding height [m]	0.65	0.76	0.79
Minimum [m]	0.45	0.53	0.55
Maximum [m]	1.84	2.05	2.12

In Table 78 the minimum and maximum distribution of discharges, resulting from the long rains can be seen (only the river discharge is shown).

Table 78: Distribution of the discharge

	Q (1/5)	Q (1/10)	Q (1/15)
Discharge river [m^3/s]	71	77	79
Distribution [%]	37	35	34
Minimum discharge [m^3/s]	45	48	50
Distribution [%]	23	22	22
Maximum discharge [m^3/s]	104	115	120
Distribution [%]	55	53	52

These variations are rather large and highly unlikely. To get rid of these uncertainties, the area would need to be surveyed.

Appendix A.6 – Floodplain Calculation

In this appendix, the discharges needed to flood the Samanga area and Ronga area are determined.

Discharge Calculation Samanga Area

During the long rains, the Samanga area should be flooded for at least one month to ensure that the ground remains fertile. The total discharge needed for the Samanga area is determined using the Manning equation. The Manning equation is used to determine the maximum discharge such that the soil of the Samanga area does not erode. This will give a good approximation of the possible discharge over the floodplain. However, to determine the exact flow velocities and water levels on the floodplain a detailed study for a number of discharge scenarios is necessary. It is not possible to do this within the time frame of this project and it is possible that a detailed study is not essential for the success of the project. The consideration should be made between the additional resources needed for a detailed study and the risk of not performing such a study.

The Samanga Area

The total Samanga area that needs to be flooded is approximated at 5 km^2. The area is divided in two stretches to make the discharge approximations more precise. See Figure 78 for a visual impression of the two stretches. For each stretch a length, width and slope is determined. The slope follows from the difference in elevation over the length. Figure 79 shows the locations from where the elevation is taken and the length. The roughness used in the Manning equation is assumed to be the same for the entire area and is set at 0.035, see Figure 80. The value is a combination of different types of floodplains. According to the fieldwork in this area, the Samanga area can be described as a cultivated area with no crops (during long rains there are no crops) with light brush and trees.

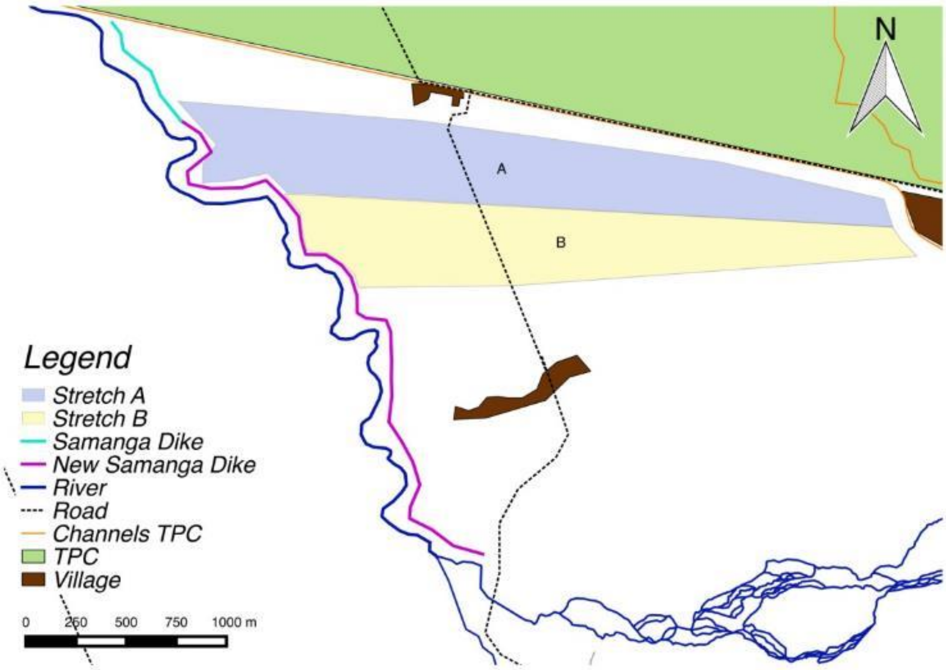


Figure 78: Samanga area with stretches

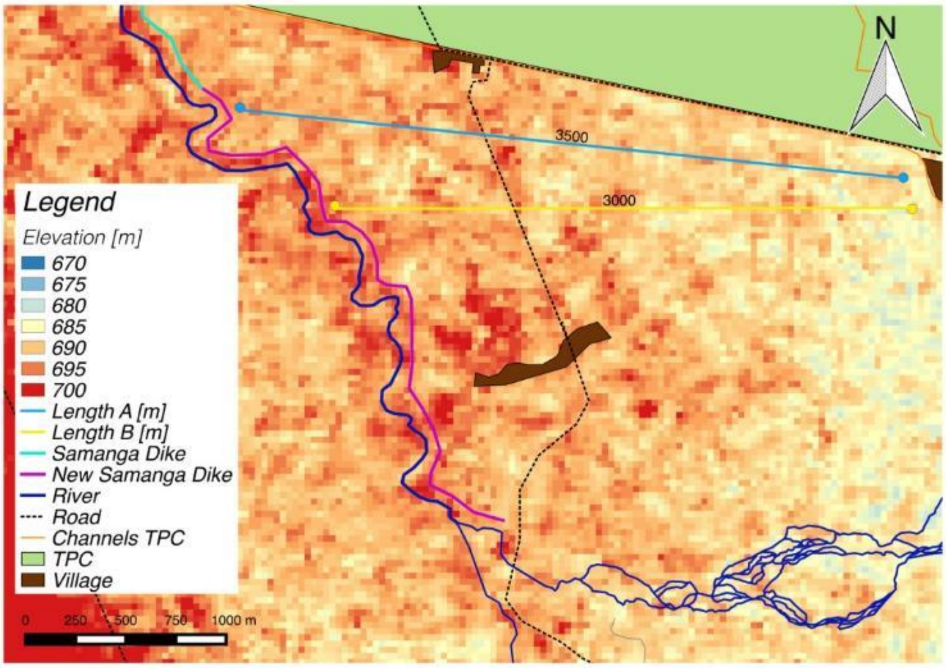


Figure 79: Elevation Samanga area

3. Floodplains	Minimum	Normal	Maximum
a. Pasture, no brush			
1.short grass	0.025	0.030	0.035
2. high grass	0.030	0.035	0.050
b. Cultivated areas			
1. no crop	0.020	0.030	0.040
2. mature row crops	0.025	0.035	0.045
3. mature field crops	0.030	0.040	0.050
c. Brush			
1. scattered brush, heavy weeds	0.035	0.050	0.070
2. light brush and trees, in winter	0.035	0.050	0.060
3. light brush and trees, in summer	0.040	0.060	0.080
4. medium to dense brush, in winter	0.045	0.070	0.110
5. medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. dense willows, summer, straight	0.110	0.150	0.200
2. cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. same as 4. with flood stage reaching branches	0.100	0.120	0.160

Figure 80: Manning's n for floodplains (Chow, 1959)

Flow Velocity

The critical depth-averaged velocity is determined; this is the highest flow speed possible before the ground starts eroding. The critical depth-averaged velocity depends on the soil type that is present on the floodplain and on the water depth. The soil on the floodplain mainly consists of silt. For silt a critical depth-averaged velocity is between 0.20 and 0.15 m/s, see Figure 81. This is a base value, for a water depth of 1 meter. For lower water depths, a correction factor K_i must be multiplied with the base value. This correction factor is 0.8 for water depths of 0.3 meter, see Figure 81. The actual water depth might be even lower in reality, but the difference is negligible. Using a base critical depth-averaged velocity of 0.175 m/s and a correction factor of 0.8 the critical depth-averaged velocity is set at 0.14 m/s.

Material	Sieve size, *D* (mm)	Critical velocity *V* (m/s) for *h* = 1 m
Very coarse gravel	200–150	3.9–3.3
	150–100	3.3–2.7
Coarse gravel	100–75	2.7–2.4
	75–50	2.4–1.9
	50–25	1.9–1.4
	25–15	1.4–1.2
	15–10	1.2–1.0
	10–5	1.0–0.8
Gravel	5–2	0.8–0.6
Coarse sand	2–0.5	0.6–0.4
Fine sand	0.5–0.1	0.4–0.25
Very fine sand	0.1–0.02	0.25–0.20
Silt	0.02–0.002	0.20–0.15

Depth, *h* (m)	0.3	0.6	1.0	1.5	2.0	2.5	3.0
Ki (-)	0.8	0.9	1.0	1.1	1.15	1.2	1.25

Figure 81: Critical depth-averaged velocity for 1 meter water depth and correction coefficient (CIRIA, 2013)

Manning Discharge

Now that the roughness coefficient, bed slope and the critical depth-averaged velocity are determined, the Manning equation can be used to calculate the water height on the floodplain, see equation 5. This is done using the information that the width of the floodplain is very large compared to the water height.

$$h = \left(\frac{V * n}{\sqrt{i_b}}\right)^{\frac{3}{2}} \tag{5}$$

Where: - V is the critical depth-averaged velocity [m/s]

- n is the Manning roughness [s/m$^{1/3}$]
- i_b is the slope on the floodplain [-]

Table 79: Water height on floodplain

	Stretch A	Stretch B
V [m/s]	0.14	0.14
n [s/m$^{1/3}$]	0.035	0.035
i_b **[-]**	0.001	0.001
h [m]	0.05	0.06

In order to determine the discharge, the height is multiplied by the flow speed and the flow width, see equation 6. The flow width is the total width of the stretch multiplied by a correction factor (c_f). The correction factor is used because the water will not flow over the entire width in reality. There are always lower and higher parts in the landscape and the water will only flow in the lower situated areas, see Figure 82. The correction factor is estimated at 0.7 assuming that initially 70% of the land will be flooded. The discharge of each stretch is shown in Table 80.

$$Q = h * V * B * c_f \tag{6}$$

Where: - Q is the discharge [m^3/s]
- h is the water height on the floodplain [m]
- V is the critical depth-averaged velocity [m/s]
- B is the width of the floodplain [m]
- c_f is the correction factor assumed [-]

Figure 82: Lower and higher areas

Table 80: Discharge per stretch

	Stretch A	Stretch B
h [m]	0.05	0.06
V [m/s]	0.14	0.14
B [m]	1000	1000
c_f **[-]**	0.7	0.7
Q [m^3/s]	4.6	6.0

Conclusion

The maximum discharge that is needed to flood the Samanga area is 10.4 m^3/s. In order to flood the Samanga area spillways will be placed in the Samanga dike. This maximum discharge will be used to design the spillways.

Discharge Calculation Ronga Area

Just like the Samanga area the Ronga area should be flooded during the long rains. The discharge that is needed to flood the area is looked at in two ways. The first one is by using the Manning equation and the second one is by using the current situation. The Manning equation is used to determine the maximum discharge such that the soil of the Ronga area does not erode. The current situation will give an indication of the minimum discharge that flows into the Ronga during the long rains. This will give a good impression on how much discharge the farmers currently have to deal with.

The Ronga Area

This total area is approximately 3 km^2 and the maximum width is approximately 1000 m. The slope of the area is determined with the elevation difference over the length, see Figure 83. The roughness coefficient is assumed to be the same for the whole area, namely 0.05, and is determined with Figure 80. The value is a combination of different types of floodplains. According to the fieldwork in this area the Ronga area can be described as a cultivated area with no crops (during long rains there are no crops) with light to medium brush and trees.

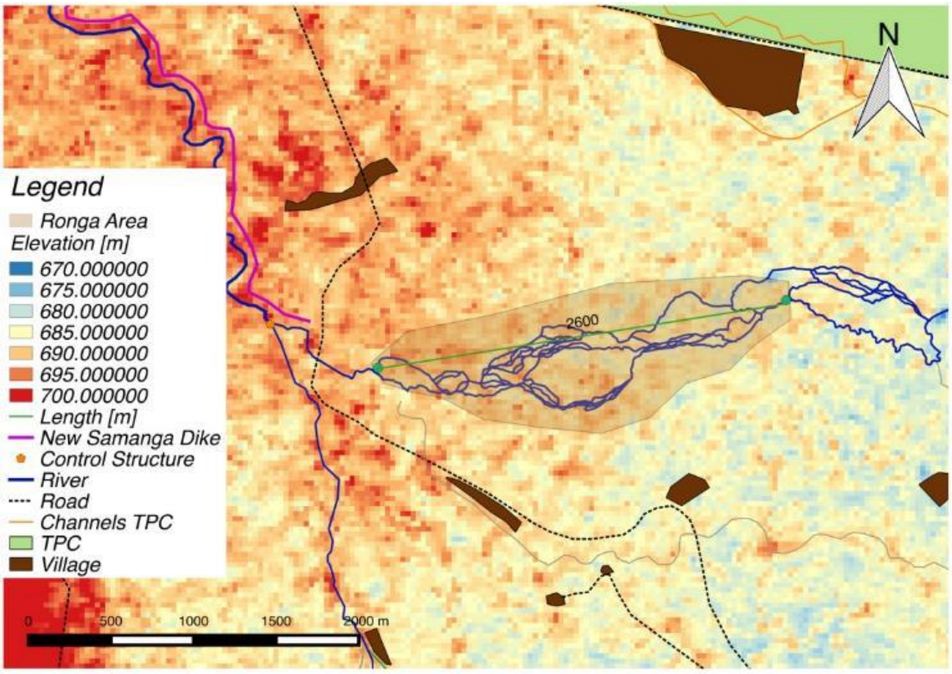


Figure 83: Elevation Ronga area

Flow Velocity

The soil type of the area is mainly silt. Using the same approach as for the Samanga area the base critical depth-averaged velocity for the Ronga area is set at 0.2 m/s. This velocity is multiplied with the correction factor of 0.8 and therefore the critical depth-averaged velocity is 0.16 m/s.

Manning Discharge

The water height on the floodplain is calculated with the Manning equation, see equation 5. The results are shown in Table 81. The discharge can now be determined with equation 6 and the results are shown in Table 82. The width of the floodplain is assumed to be the maximum width of the Ronga area multiplied by a correction factor (c_f). The correction factor is set at 0.7, assuming that 70% of the Ronga area is flooded. The calculated discharge has to be added up to the discharge capacity of the Ronga Braided, so the maximum discharge of the Ronga Braided can be determined, see Table 83.

Table 81: Water height Ronga area

	Ronga area
V [m/s]	0.16
n [s/m$^{1/3}$]	0.05
i_b **[-]**	0.001
h [m]	0.11

Table 82: Additional discharge Ronga area

	Ronga area
h [m]	0.11
V [m/s]	0.16
B [m]	1000
c_f **[-]**	0.7
Q [m^3/s]	13

Table 83: Maximum discharge Ronga area

	Ronga area
Discharge capacity [m^3/s]	25
Q [m^3/s]	13
Maximum discharge [m^3/s]	38

Discharge Current Situation

From the discharge measurements of the past years, see Appendix A.4 - Design Discharges; it can be concluded that during the long rains the minimum discharge that enters the project area is 45 m^3/s. Since the capacity of the Kikuletwa North is 48 m^3/s, see chapter 7 The Rivers, this discharge of 45 m^3/s enters the bifurcation. In the current situation, the Kikuletwa South Small has a capacity of 2 m^3/s, so a minimum discharge of 43 m^3/s enters the Ronga during the long rains. This is larger than the capacity of the Ronga and therefore the Ronga area floods. Most of the times, however, the discharge that enters the Kikuletwa North during the long rains is higher than 45 m^3/s. Although the Kikuletwa North floods during higher discharges and therefore the incoming discharge does not reach the bifurcation entirely, it can still be assumed that the discharge entering the Ronga will be higher than 43 m^3/s.

Conclusion

On the one hand, the maximum discharge is calculated using the Manning equation. This discharge ensures that the soil will not erode. On the other hand, the minimum discharge that currently enters the Ronga is determined. The minimum discharge that currently enters the Ronga is higher than the maximum calculated discharge.

The discharge that will be used as the design discharge of the Ronga is 43 m^3/s. This value has been chosen because this is the minimum discharge that currently enters the Ronga and the maximum discharge that is calculated with the Manning equation is close to 43 m^3/s.

Appendix B – Initial Design

Appendix B.1 - Design Calculations Dike

In this appendix the dike will be tested for a number of failure mechanisms. In Figure 84 an overview of the possible failure mechanism of a dike can be seen (CIRIA, 2013).

Main ultimate limit states (ULS)

Scale	Type	Typical ULS failure mechanisms
Intergranular, phreatic and hydrostatic forces at global scale	Shear failure	Overall stability during construction or levee raising[1]
		Overall stability during a flood – rotational, non-circular, translational, sliding, uplift (see below) etc
		Stability after a flood (rapid draw-down)
		Seismic stability without pressure rise
	Hydraulic heave	Hydraulic uplift of soil at landward toe (may contribute to loss of overall stability)
		Hydraulic separation between the levee and rigid embedded structure caused by the seepage pressure exceeding the total contact pressure
Hydrodynamic forces at global scale	Static or dynamic liquefaction	Sand boiling in the vicinity of the landward toe
		Liquefaction (primarily due to seismic event)
Hydrodynamic forces at local scale	Erosion (deterioration that may trigger instability)	Internal erosion
		External erosion by overflowing
		External erosion by scouring

Figure 84: Failure mechanisms dike

Mass Instability

Mass instability (shear failure) is characterized by sliding, and therefore collapsing of the slope of a dike. This is due to shear strength of the dike not being able to resist the forces acting on it. The stages that this instability is of importance are (CIRIA, 2013):

1. Fast loading during the short rains.
2. Slow loading during long rains.
3. Post-flood situations, including rapid draw down and reverse flood loading.
4. The vulnerable stages during the construction process in the dry season.

To determine if the designed dike is stable for these different situations D-Geo Stability has been used. This program uses Bishops method to determine the minimum safety factor of a soil structure by performing a slip plane calculation based on equilibrium of horizontal forces and moments. D-Geo Stability performs the calculation on several slip planes and determines the slip plane with the lowest safety factor.

Assumptions

These assumptions have been made prior to receiving the results of the laboratory testing of the soil.

Unit Weight

The layers of the dike will be modelled with the same material characteristics. During field observations, the soils were determined to be silt with variations in traces of fine sand and clay. Therefore, it is assumed all layers have the same properties. The dry weight of silt is taken as 16 kN/m^3 and the wet weight as 17 kN/m^3 (Command, 1986).

Cohesion and Friction Angle

The cohesion and friction angle have been determined based on different loading conditions. For long-term conditions (in which the dike is able to drain), the friction angle should be taken as the dominant factor and the cohesion can be set to zero. For short-term conditions (in which the dike is unable to drain quickly enough), the cohesion is the dominant factor and the friction angle can be set to zero. Since the short rain floods are flashy, the short-term conditions are applied. The long rain floods last longer and therefore the long-term approach can be taken. These loading conditions were identified with the help of a geotechnical engineer. The friction angle has been found to be 33 degrees (Command, 1986). The cohesion has been determined by using the lowest value found with the pocket vane shear tester during field-testing. This value is set at 15 kPa. This can be verified by the near horizontal riverbanks observed in the area. In Table 84, the results are displayed.

Table 84: Loading scenarios

Loading scenario	Condition	Friction [°]	Cohesion [kPa]
Short rain	Undrained	0	15
Long rain	Drained	33	0
Post flood	Undrained	0	15
Dry season	Drained	33	0

Ground Water Table (GWT)

Based on measurements taken by piezometers in the lower fields near the river at TPC the GWT is assumed one meter below surface level during all loading scenarios.

Loading Scenarios

- During the short rains, undrained conditions apply. The design height is applied for the river water level, see Figure 85.

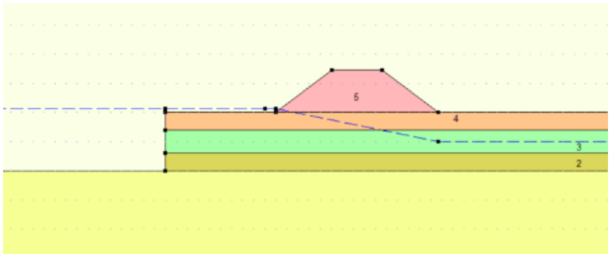


Figure 85: Schematization short rain floods

- During the long rains, drained conditions apply. The design height is applied for the river water level, see Figure 86.

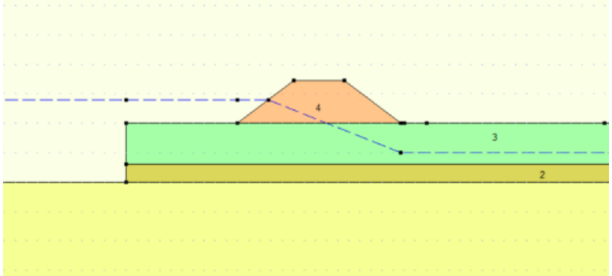


Figure 86: Schematization long rain floods

- During post-flood situations, including rapid draw down and reverse flood loading the phreatic water level is modelled higher in the dike than the river level at that moment. This is due to the undrained conditions from the flashy floods. The water level is modelled as 1.5 meters in the river. The post flood situation has been modelled for the long rains, as the difference is larger, see Figure 87.

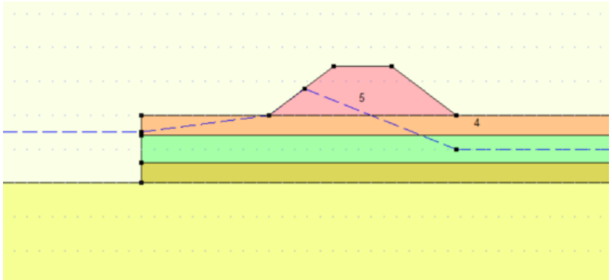


Figure 87: Schematization rapid draw down

- During the vulnerable stages during of construction process, the water level is assumed to be one meter in the river. Additionally, a compactor of 2.5 m width with uniform weight of 15 kN/m^3 is applied on top of the dike, see Figure 88.

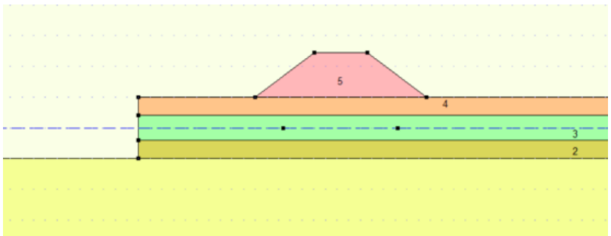


Figure 88: Schematization construction during dry season

Results

The different scenarios were run through the D-Geo Stability program for the new dike. The resulting values can found in Table 85.

Table 85: Factors of Safety D-Geo

Loading scenario	Q (1/5)	Q (1/10)	Q (1/15)
Short rain	3.0	2.9	2.8
Long rain	2.2	2.2	2.3
Post flood	2.5	2.4	2.3
Dry season	2.2	2.3	2.3

The safety factors, which should be reached, can be found in Figure 89.

	Applicable stability conditions and required factors of safety (FS)			
Type of slope	*End-of- construction*	*Long-term (steady seepage)*	*Rapid draw-down*[a]	*Earthquake*[b]
New levees	1.3	1.4	1.0 to 1.2	(see notes)
Existing levees	–	1.4[c]	1.0 to 1.2	(see notes)
Other dikes and embankments[d]	1.3[e,f]	1.4[c,f]	1.0 to 1.2[f]	(see notes)

Figure 89: FOS (CIRIA, 2013)

The factors of safety determined by D-Geo stability are larger than these values; therefore, the dike is stable for all scenarios.

Seepage

The permeability of a dike should be sufficient and not allow for seepage through or below the structure. Seepage can occur suddenly or be on going deterioration. Seepage of water through the dike can lead to internal erosion or slope failure of the inner slope. As can be seen in Figure 90, if the structure is too permeable (b) then seepage will occur.

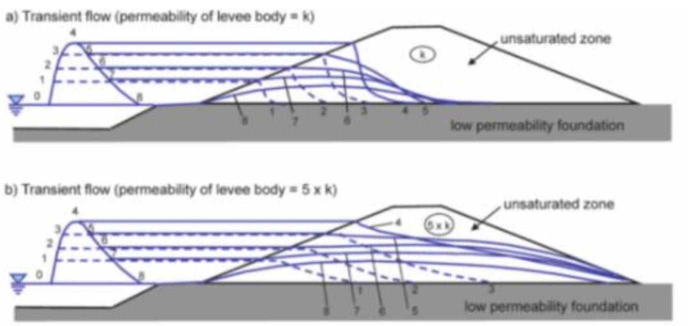


Figure 90: Seepage through dike (CIRIA, 2013)

Secondly, the period of exposure of the dike to the design water level of the flood is of importance. The dike at Samanga is not constantly subjected to high water levels, therefore constant seepage pressures are probably not present, and thus not the main cause of failure. This is visualised in Figure 91.

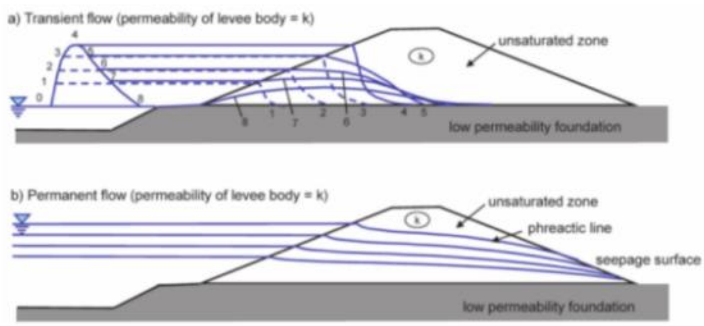


Figure 91: Seepage through dike (CIRIA, 2013)

Since a seepage analysis will not be performed taking preventative measures is advisable for both the new and existing dike. To prevent seepage an impervious layer (e.g. clay) could be used as well as a stabilizing berm, cut-off walls, toe drains or relief wells. An impervious layer seems like the best option with the available local capabilities. Clay would be a good solution; however, this is likely not available in the area. Seeing as the leakage length according to Bligh (see internal erosion) is sufficient and the current dike does not seem to have any problems using no measures is possible. Additionally, the high water levels are not constant. The existing dike has a clayey layer beneath it, which could be preventing seepage.

External Erosion

External erosion is important for the exposed fill materials. Due to high water levels, overtopping can occur causing erosion of the inner slope of the dike. Furthermore, channel scour is also possible on the riverside due to currents. Observations during field visits showed that erosion along the riverbed is common, especially at outer bends as shown in Figure 92. The dike has been dimensioned against overtopping so this should not occur.

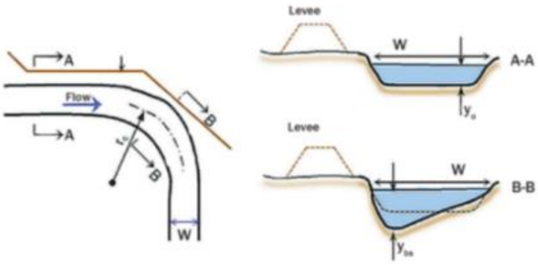


Figure 92: Erosion outer bend (CIRIA, 2013)

The velocities of the river during the design discharges can be found in the table below. These have been determined by dividing the design discharge of the river by the cross-sectional area of the river, see Table 86.

Table 86: Velocities Kikuletwa North

	Q (1/5)	Q (1/10)	Q (1/15)
Velocity long rains [m/s]	0.97	0.99	1.00
Velocity Short rains [m/s]	0.78	0.81	0.82

The allowable limits of critical velocity for silt, according to the International Levee Handbook (CIRIA, 2013), are between 0.2 and 0.15 m/s for a water depth of 1m. The expected water depths are 2 meters for the river and between 0.65 and 0.79 for the dike. Using the correction factors result in critical velocities for the riverbank (k=1.15) of 0.23 and 0.17 m/s and for the dike (k=0.9) of 0.18 and 0.14 m/s. These are all much lower than the expected velocities. Therefore, surface protection is recommended.

Surface protection can prevent external erosion. Materials such as sand/gravel, clay/grass, armour stone (riprap), gabions or mattresses including geotextiles, placed concrete blocks including tied block mattresses and continuous asphaltic paving can be used. These can be placed on the waterside, crest or inner slope. The current dike has a lot of vegetation growing on it, including grass. The grass binds the top soil together to resist external erosion. Grass is thus a reasonable preventive measure against external erosion for the new dike too, which is cheap and robust. An example of a grass cover can be seen in Figure 93. Vetiver grass is available in Tanzania.

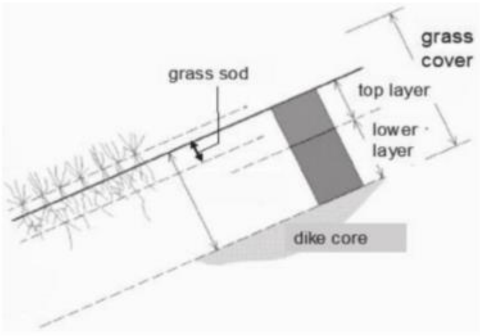


Figure 93: Grass protection (CIRIA, 2013)

Additionally, to make sure erosion of the riverbanks does not affect the dike, the new dike should be placed a certain distance from the riverbanks.

Internal Erosion

Internal erosion consists of a gradual loss in fill material due to seepage through or below the dike. The permeability of the dike and of the foundation are thus of importance. Internal erosion can have the following forms (CIRIA, 2013):

- Backward erosion – detachment of fill materials due to seepage leading to pipes.
- Concentrated leak erosion – detachment of fill materials due to existing holes.
- Suffusion/Contact erosion – detachment of fill materials through coarser materials.

Backward erosion (piping) is the most common failure mechanism. Bligh's method can be used to see if the length of the dike is larger than the seepage length, see equation 7.

$$L_H > \gamma \times \ \Delta H \times C_{bligh} \tag{7}$$

Where - L_H is the leakage length [m];
- γ is a safety factor [-];
- ΔH is the flooding height in [m];
- $C_{bligh}$is the creep factor of Bligh [-] (18 for silt).

Since the high water level is not constant, a safety factor of 1 is sufficient. The results can be seen in Table 87.

Table 87: Bligh

	Q (1/5)	Q (1/10)	Q (1/15)
Minimum leakage length [m]	11.7	13.7	14.2
Width [m]	12.7	13.9	14.7

The widths of the dikes are therefore safe against backward erosion. The width of the current dike is on average 8.2 meters. This is insufficient.

Concentrated leak erosion happens if there are changes made by, for instance, farmers in the dike. This should be monitored to make sure no holes are made in the dike. The dike is expected to be made of a homogeneous material similar to that of its foundation. Therefore, it is unlikely that suffusion or contact erosion will occur.

Settlement

Settlement is problematic for foundations consisting of clay and peat. This is not the case here as the foundation in made of silt. Additionally, the dike that will be built is not very high (between 1-1.5 meters) so settlement should not occur in large magnitudes. This could be surveyed periodically.

Appendix B.2 - Design Calculations Spillways

The new spillways have a design that is similar to the existing spillways. This means that they are designed as manually operated vertical gates under submerged underflow. This appendix will firstly treat the possibility of irrigation through the spillways and the discharge that is possible through the spillways. Secondly, the calculations that have been executed in order to determine the required dimensions will be shown. Thirdly, the loads and moments that are acting on the structure are presented. Lastly, the calculations that are required to ensure safety against the most common failure mechanisms are shown.

Irrigation through the Spillways

To make sure that irrigation is possible throughout the year a certain minimum water depth in the Kikuletwa North will be determined. The idea is to make the spillways lower than the most common low water level, to ensure that the farmers have access to the water in almost every scenario. The approach is to use the water depth at the IDD1 to estimate the water depth in the Kikuletwa North. In Figure 94, the water depths at IDD1 for the last four years can be seen. After elimination of some incorrect measurements, the water depth that is exceeded 97.7% of the days could be determined. This water depth is 0.7 meter at the IDD1 measuring station.

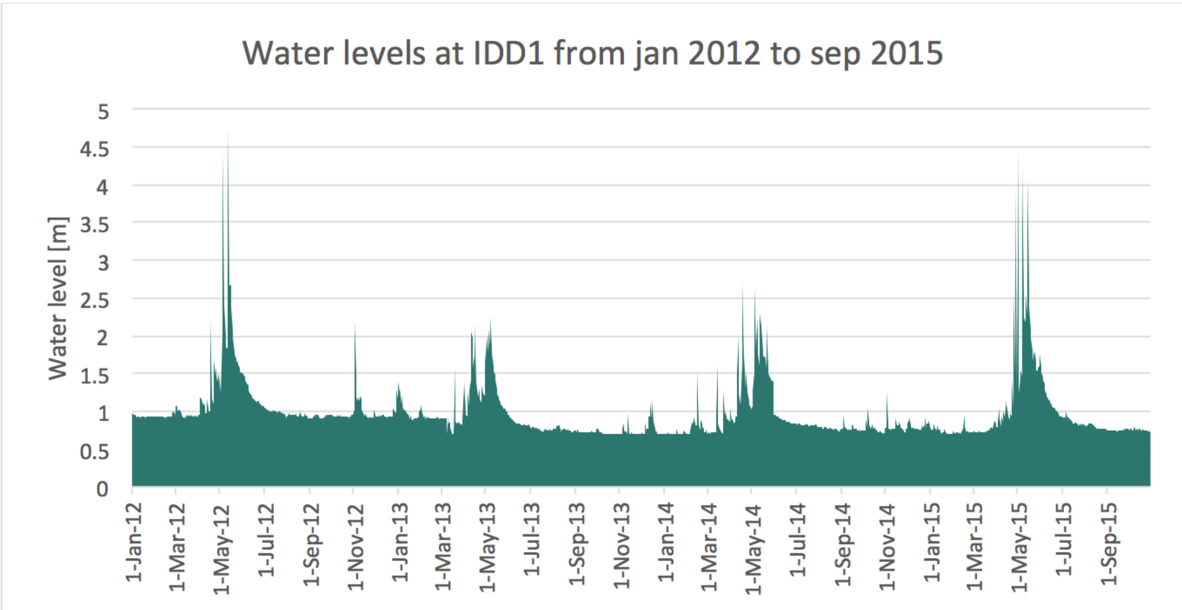


Figure 94: Graph of the water levels at IDD1 from January 2012 to September 2015

Water Depth during the Dry Season

The Manning equation is used to estimate the resulting water depth in the Kikuletwa North during the dry season, see equation 9 (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015). This is done assuming that the discharge at the location of the IDD1 station is equal to the discharge in the Kikuletwa North, see equation 8.

$$Q_{IDD1} = Q_{kn} \tag{8}$$

Where: - Q_{IDD1} is the discharge at the IDD1 station [m^3/s]
- Q_{kn} is the discharge in the Kikuletwa North [m^3/s]

$$Q = A * \frac{R^{\frac{2}{3}} * i_b^{\frac{1}{2}}}{n} \tag{9}$$

Where: - Q is the discharge [m^3/s]
- A is the cross-sectional area [m^2]
- R is the hydraulic radius [m]
- i_b is the bed slope [-]
- n is the Roughness coefficient [s/m$^{1/3}$]

Substituting equation 9 into equation 8 gives a water height in the Kikuletwa North. The other unknowns can be found in Appendix A.3 - Current River Dimensions. The resulting water depth is 0.75 meters in the Kikuletwa North.

Conclusion

The Kikuletwa North is on average 2 meters deep and therefore the spillways should be placed 1.3 meters lower in the ground to ensure access to water throughout the year. This is possible and can be made in such a way that it will work; the only problem is that after the dike the water is still 1.3 meter below ground level. A deep and complex irrigation network can distribute the water across the Samanga area. This is however a very expensive and unrealistic solution. Therefore, this idea will not be used in the spillway design.

Discharge Passable through the Spillways

In order to flood the Samanga area during the long rains a discharge is needed through the spillways. Two design situations are recognised, and for both a discharge through the spillways will be determined:

1. Lowest possible water level during the long rains.
2. Highest possible water level, during the discharge of the 1/15 year return period.

It is very important that for every possible long rain situation flooding of the Samanga area can occur. With these two situations, the extremes are tested, so if both are considered to be acceptable every water level between the two extremes will be acceptable as well.

Discharge during Lowest Possible Water Level

During the long rains with the lowest possible discharge, the Kikuletwa will just reach its bank full discharge. This means that in a normal situation the river cannot flood. However, the spillways will be constructed 0.6 meters below the top of the existing riverbanks. Therefore, flooding is still possible, see Figure 95. The red line indicates the location of the spillway and the dashed line the bottom of the irrigation channel. In this situation, it is assumed that the Kikuletwa North has reached its bank full capacity.

The channel from the river to the spillway and via the spillway to the other side of the dike can be modelled as an irrigation channel. The discharge possible through the channel will be calculated using the method provided by (Food and Agriculture Organisation of the United Nations).

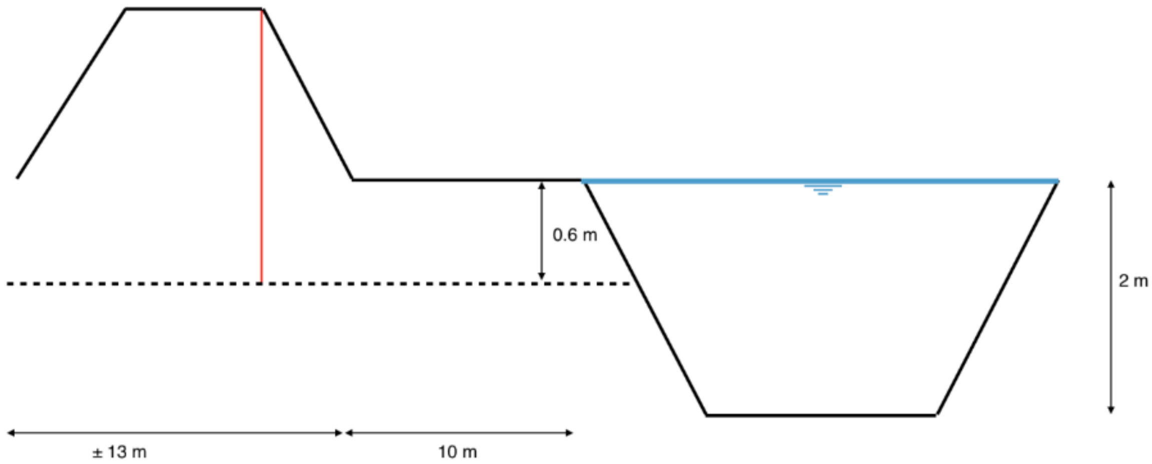


Figure 95: Cross-section Kikuletwa North and Samanga dike, with proposed channel

Assumptions made:

- The water level will be until the top of the Kikuletwa riverbank.
- Therefore, the water depth in the channel will be 0.6 meters deep.
- The width of the channel is 1.8 meters, see Figure 96.

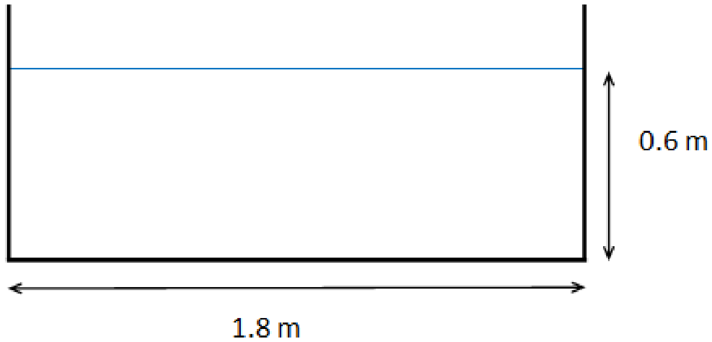
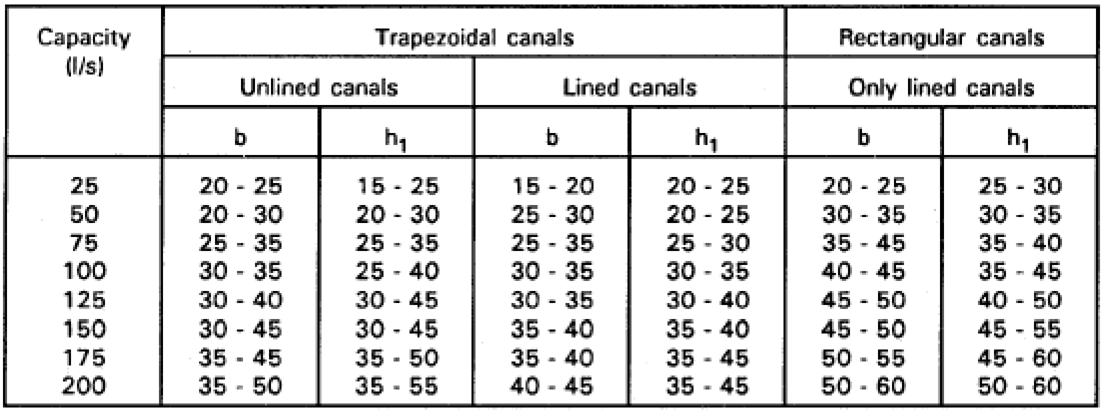


Figure 96: Cross-section of the assumed channel

Using the table from the (Food and Agriculture Organisation of the United Nations), see Figure 97 the discharge through the channel can be set at 200 l/s, 0.2 m^3/s. This is using a depth of 0.6 meters and a width of 0.6 meters and a slope of 0.15%.



Capacity (l/s)	Trapezoidal canals				Rectangular canals	
	Unlined canals		Lined canals		Only lined canals	
	b	h_1	b	h_1	b	h_1
25	20 - 25	15 - 25	15 - 20	20 - 25	20 - 25	25 - 30
50	20 - 30	20 - 30	25 - 30	20 - 25	30 - 35	30 - 35
75	25 - 35	25 - 35	25 - 35	25 - 30	35 - 45	35 - 40
100	30 - 35	25 - 40	30 - 35	30 - 35	40 - 45	35 - 45
125	30 - 40	30 - 45	30 - 35	30 - 40	45 - 50	40 - 50
150	30 - 45	30 - 45	35 - 40	35 - 40	45 - 50	45 - 55
175	35 - 45	35 - 50	35 - 40	35 - 45	50 - 55	45 - 60
200	35 - 50	35 - 55	40 - 45	35 - 45	50 - 60	50 - 60

Figure 97: Discharge possible through irrigation channel

Now using the information that indicates that through a channel of 0.6x0.6 m^2 a discharge of 0.2 m^3/s is possible. A channel of 1.8 meters wide can discharge 3 x 0.2= 0.6 m^3/s.

The slope will be calculated using the distance from the riverbank to the other side of the dike, see chapter 10 Cluster 1. This 10 + 12 = 22 meters up to 10 + 14.7 = 24.7 meters depending on the return period. This results in a height difference between 0.033 and 0.037 meters.

Assuming five spillways will be constructed the total discharge from the Kikuletwa North into the Samanga area will be 3 m^3/s. A quick check is done to see if the total discharge will be enough to flood the Samanga area, using a fast and relatively easy calculation. This calculation can only give an indication of the reality because a lot is unknown, such as; water absorbed by the ground in the area, the amount precipitation and evaporation and the exact area flooded. However, the calculation will still give a reasonable indication of the time needed to flood the area with the given discharge, because these unknowns can cancel each other out. Still, if the exact flooding time and amount of water needed needs to be determined, a more detailed study must be performed.

The total area that needs to be flooded is multiplied by an average preferred flooding height to get a volume. Then this flooding volume is divided by the incoming discharge to obtain the time needed to flood the area sufficiently. The area is 5 km^2 and an estimation of a preferable average flooding height is 10 cm, see also Appendix A.6 – Floodplain Calculation. This results in a flooding volume of 1 million m^3. The time needed to flood this volume is approximately 2 days.

To see if the flood will stay at a high level for at least 2 days the water level during the long rains is studied. This is done based on water level measurement at IDD1, see Figure 98. The water level at IDD1 for the long rain season of 2014 is shown. This is only an example of water levels during the long rain season, but after looking at graphs of different years, it could be concluded that for every year the duration of the peaks will always amount more than 4 days in total. For the example in Figure 98, this is approximately 14 days. This is indicated in the graph as the parts above the 2-meter line. Note that this is the water level at IDD1, but it is only used to indicate the duration of the peaks and this will be the same in the Kikuletwa North.

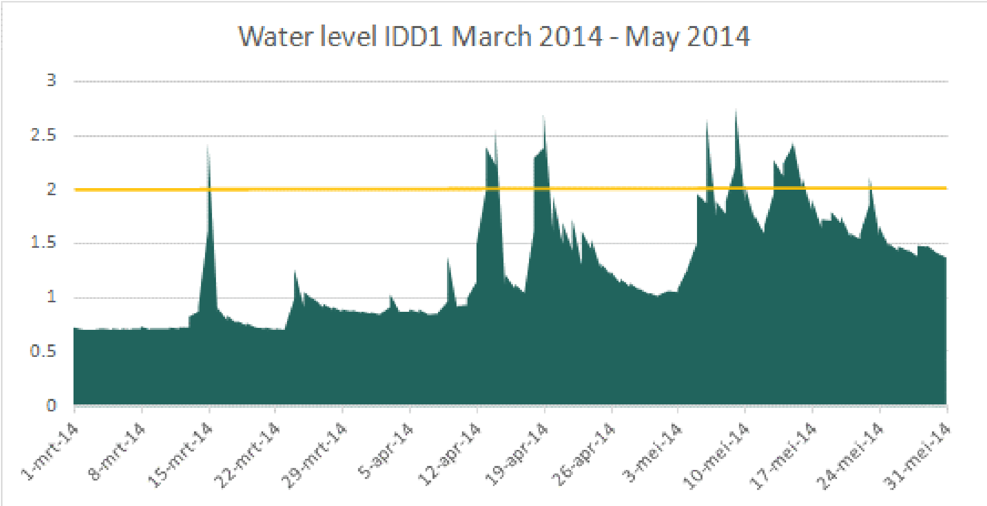


Figure 98: Duration of high water peaks, based on measurements at IDD1 (long rain season 2014)

Discharge during Highest Possible Water Level

The highest possible water level is 2.79 meters, see chapter 8 Design Discharges. The situation can be schematized as indicated in Figure 99, this situation can be recognised as an underflow gate. Equation 10 can be used to determine the discharge through the underflow gate (Ankum, 2002).

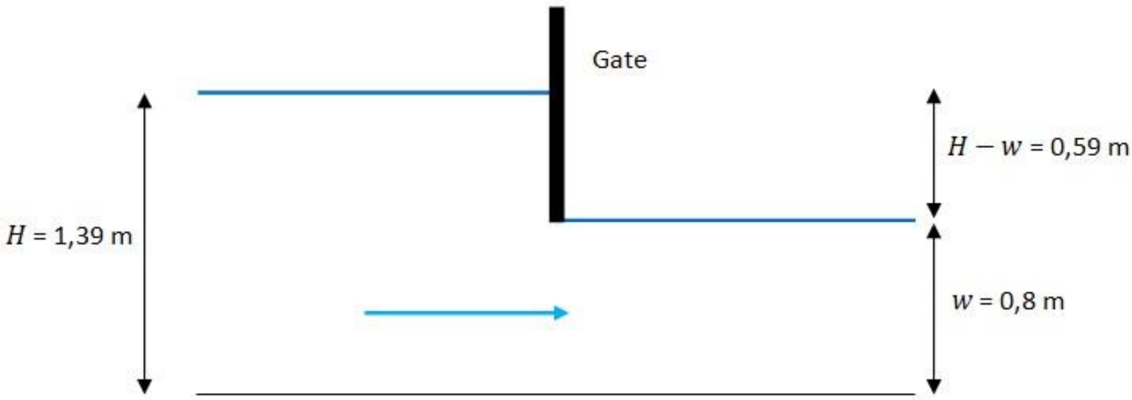


Figure 99: Schematization of discharge through the spillway

$$Q = \mu * b * w * \sqrt{2g * (H - w)} \tag{10}$$

Where: - Q is the discharge through the gate [m^3/s]
- μ is the contraction coefficient, indicating loss of energy [-]
- b is the width of the gate [m]
- w is the height under the gate [m]
- g is the gravitational acceleration, set on 9.78 [m/s^2]
- H is the water height for the gate [m]

The contraction coefficient μ can be determined using equation 11. There is no flow streamliner present, therefore d can be taken equal to zero.

$$\mu = 0.51 + 0.1 * \sqrt{23 - \left(\frac{d}{w} - 4.7\right)^2} \tag{11}$$

Where: - μ is the contraction coefficient, indicating loss of energy [-]
- w is the height of the gate [m]
- d is the diameter of the flow streamliner [m], not present in this case.

First the contraction coefficient is calculated, resulting in a μ of 0.6. Now the discharge through the gate is calculated, resulting in a Q of 2.94 m^3/s. With an average flow speed of approximately 2 m/s.

To check if the channel through the dike is able to discharge the calculated discharge, the maximum flow speed in the channel is calculated. The maximum flow speed depends on the water depth in the channel and is limited by the requirement that the flow velocity should not get super critical. This can be expressed by the Froude number being smaller than 0.5, therefore avoiding standing waves. The maximum velocity is expressed in equation 12 and is equal to 1.4 m/s for a water depth of 0.8 meter.

$$v < 0.5\,(g * y)^{0.5} \tag{12}$$

Where: - v is the maximum flow velocity [m/s]
- g is the gravitational acceleration, set on 9.78 [m/s^2]
- y is the water depth [m]

Because the maximum flow velocity is smaller than the velocity calculated earlier, a new discharge will be determined. This is done using the maximum flow velocity and multiplying it with the cross-sectional area of the channel. This results in a discharge of 1.4*0.8*1.8 = 2 m^3/s. The maximum discharge that can flow through the channel is lower than the discharge through the gate. Therefore, the discharge through the channel is normative.

Assuming five spillways, the total discharge from the Kikuletwa North into the Samanga area will be 10 m^3/s. This is during the highest possible water level in the river and it therefore assumed to be the highest possible discharge through the spillways. The time needed to flood the area is calculated in the same way as before, using the total area to be flooded and the preferable average flooding height. This results in a flooding time of 1 day and 4 hours. This is fast enough considering that the water level peaks maintain their height for approximately 14 days, see Figure 98.

Dimensions

The dimensions of the spillway are determined by looking at the dimensions of reference projects. By performing unity checks on the possible failure mechanisms later on, it is possible to determine whether the assumed dimensions are feasible. The only calculation that has to be performed is in order to determine the thickness is of the steel door, this calculation is shown below.

Steel Door

First, the distributed load on the concrete wall is determined, which is a water load in this case, see equation 13.

$$q = 0.5\rho g h^2 \text{ [kN/m]} \tag{13}$$

The moment is calculated using this value, using equation 14.

$$M_{max} = \frac{1}{8} q l^2 \text{ [kNm]} \tag{14}$$

The maximum moment that the steel can take is then given by equation 15 below.

$$M_{max} = \sigma_{steel} \times W = \sigma_{steel} \times \frac{1}{6} t h^2 \text{ [kNm]} \tag{15}$$

The yielding strength of steel is 235,000 kN/m^2
Therefore, the thickness can be determined with equation 16.

$$t = \frac{M_{max}}{\frac{1}{6}\sigma_{steel} * h^2} \text{ [m]} \tag{16}$$

The resulting dimensions are found in Table 88.

Table 88: Dimensions steel door spillway

	Length/thickness [m]	Width [m]	Height [m]
Steel door	0.01	1.8	0.80

Loads

To determine the failure mechanisms, the loads acting on the dike have to be determined. The constants that are used can be found in Table 89.

Table 89: Constants used by determining loads

Constant	Unit	Value (12)
Gravitational acceleration	[m/s^2]	9.78
Water density	[kg/m^3]	1020
Steel density	[kN/m^3]	78
Concrete density	[kN/m^3]	25
Soil density	[kN/m^3]	16

The loads have been determined per meter width. The loads have been calculated by using the dimensions and multiplying them by their respective densities. The most unfavourable situation has been taken, in which the water level is at a maximum during the long rain season. In other words, at the riverside the water level is maximum and at the landside the water level is equal to zero.

Horizontal Loads

The horizontal loads acting on the structure are:

- Water pressure from the river

The values are found in Table 90.

Table 90: Total horizontal load spillway

	Q (1/5)	Q (1/10)	Q (1/15)
Total Horizontal load [kN/m]	12.0	13.7	14.2

Vertical Loads

The vertical loads acting on the structure are:

- Self-weight
  - Foundation slab and coffers
  - Spillway walls
  - Steel door
- Soil
- Water

The values can be found in Table 91.

Table 91: Total vertical load spillway

	Q (1/5)	Q (1/10)	Q (1/15)
Total vertical load [kN/m]	175.5	190.2	203.1

Moments

The moments have been determined by multiplying the loads with the distance of their centre of gravity to the centre of the foundation slab. The resulting values are found in Table 92.

Table 92: Total moment spillway

	Q (1/5)	Q (1/10)	Q (1/15)
Total moment [kNm/m]	194.7	187.4	180.7

These loads and moments acting on the spillway as schematized in Figure 100.

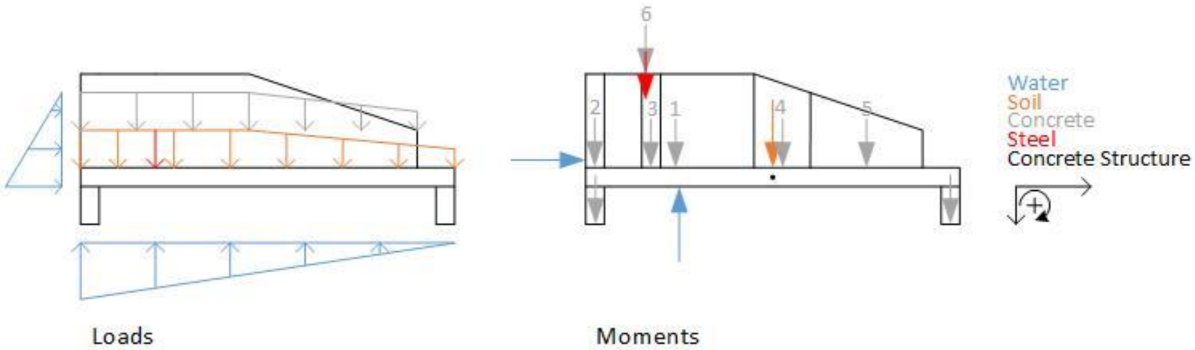


Figure 100: Loads and moments acting on spillway

Failure Mechanisms

The following failure mechanisms have been determined using the Hydraulic Structures Manual (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015).

Horizontal Stability

The horizontal stability is calculated by comparing the total horizontal load to the total vertical load, see equation 17 and 18.

$$\sum H \leq f \sum V \tag{17}$$

$$f = \tan\left(\frac{2}{3}\varphi\right) \tag{18}$$

Where: - φ is the angle of internal friction [°]. This was determined to be 33° in drained conditions, see Appendix B.1 - Design Calculations Dike.

The resulting values are found in Table 93.

Table 93: Horizontal stability check

Quantity	Q (1/5)	Q (1/10)	Q (1/15)
f [-]	0.4	0.4	0.4
$\sum H$ **[kN/m]**	12.0	13.7	14.2
$f \sum V$ **[kN/m]**	70.9	76.8	82.1
Unity check	**5.9**	**5.6**	**5.8**

The unity check is larger than one; therefore, the structure is safe against horizontal stability.

Rotational Stability

The rotational stability is calculated with equation 19. By dividing the total moment by the total vertical load and comparing the result to the length of the foundation divided by 6.

$$\frac{\sum M}{\sum V} \leq \frac{L}{6} \tag{19}$$

Where: - L is the length of the foundation [m]

The resulting values are found in Table 94.

Table 94: Rotational stability check

Quantity	Q (1/5)	Q (1/10)	Q (1/15)
$\frac{\sum M}{\sum V}$ **[m]**	1.1	1.0	0.9
$\frac{L}{6}$ **[m]**	1.7	1.7	1.7
Unity Check	**1.5**	**1.7**	**1.9**

The unity checks are larger than one; therefore, the structure is safe against rotation.

Vertical Stability

The vertical stability has been tested by comparing the load acting on the soil with the bearing capacity of the soil. The bearing capacity ($\sigma_{k,max}$) of silt is estimated to be 70 kPa (Council, 2007), see equation 20.

$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{bl} + \frac{\sum M}{\frac{1}{6}lb^2} \leq 70 \tag{20}$$

The minimum bearing capacity can also be determined with equation 21. This minimum value should be larger than zero.

$$\sigma_{k,min} = \frac{F}{A} - \frac{M}{W} = \frac{\sum V}{bl} - \frac{\sum M}{\frac{1}{6}lb^2} \geq 0 \tag{21}$$

Where: - F is the vertical force [kN]
- A is the cross-sectional area [m^2]
- B is in this case the length [m]
- L is the width in [m]

The moments and loads are already taken per meter width therefore, L can be neglected.

Table 95: Vertical stability check

Quantity	Q (1/5)	Q (1/10)	Q (1/15)
$\frac{\sum V}{b}$ **[kN/m^2]**	17.6	19.0	20.3
$\frac{\sum M}{\frac{1}{6}b^2}$ **[kN/m^2]**	11.7	11.2	10.8
σk,max [kPa]	29.2	30.3	31.2
σk,min [kPa]	5.9	7.8	9.5
Unity Check max	**2.4**	**2.3**	**2.2**

Table 95 shows that the unity check for σk,max is larger than one and that σk,min is larger than zero. Therefore, the structure is safe against vertical stability.

Piping

Bligh

Bligh's method can be used to see if the structure is safe against piping, see equation 22.

$$L_H > \gamma \times \Delta H \times C_{bligh} \tag{22}$$

Where: - L_H is the leakage length [m]
- γ is a safety factor [-]. Since the high water level is not constant, a safety factor of one is sufficient.
- ΔH is the flooding height [m]
- C_{bligh} is the creep factor of Bligh [-]. This is 18 for silt.

Table 96: Piping check

	Q (1/5)	Q (1/10)	Q (1/15)
Seepage length [m]	11.7	13.68	14.22
Actual length [m]	15	15	15
Unity check	**1.28**	**1.10**	**1.05**

Table 96 shows that the actual length is larger than the seepage length of Bligh. Therefore, the structure is safe against piping.

Lane

In equation 23 Lane states that:

$$L > \gamma \times \Delta H \times C_{Lane} \quad (23)$$

However, L is not an addition anymore of the horizontal and vertical path, but is given with equation 24:

$$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor} \quad (24)$$

Where: - L is the leakage length [m]
- γ is a safety factor [-]. Since the high water level is not constant, a safety factor of one is sufficient.
- ΔH is the flooding height [m]
- C_{Lane} is the creep factor of Lane [-]. This is 8.5 for silt.

The results can be found in Table 97.

Table 97: Results Lane

	Q (1/5)	Q (1/10)	Q (1/15)
Seepage length [m]	5.5	6.5	6.7
Actual length [m]	8.3	8.3	8.3
Unity check	**1.5**	**1.3**	**1.2**

The actual length is larger than the seepage length of Lane. Therefore, the structure is safe against piping.

Scour

The structure is safe against scour if the coffers lengths are four times the bottom thickness (Ankum, 2002). Since the coffer lengths are 0.8 m and the bottom thickness is 0.2 m the structure is safe against scour.

Appendix B.3 - Design Calculations Kikuletwa South

In this appendix, the calculations using Manning's equation for the initial design of the New Kikuletwa South are explained.

Manning's Equation

Manning's equation is used to determine the dimensions of the New Kikuletwa South and is showed in equation 25 (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015).

$$Q = V * A = \frac{R^{2/3} * i_b^{1/2}}{n} * A \tag{25}$$

Where: - Q is the discharge [m^3/s]
- V is the velocity [m/s]
- A is the cross-sectional area [m^2]
- R is the hydraulic radius [m]
- i_b is the bed slope [-]
- n is the Manning's roughness coefficient [$s/m^{1/3}$]

The given values in this equation are the discharge, taken from Appendix A.4 - Design Discharges and the roughness coefficient, which is explained below.

Cross-section

The river consists of two sections, the deep section and the main section, see Figure 101.

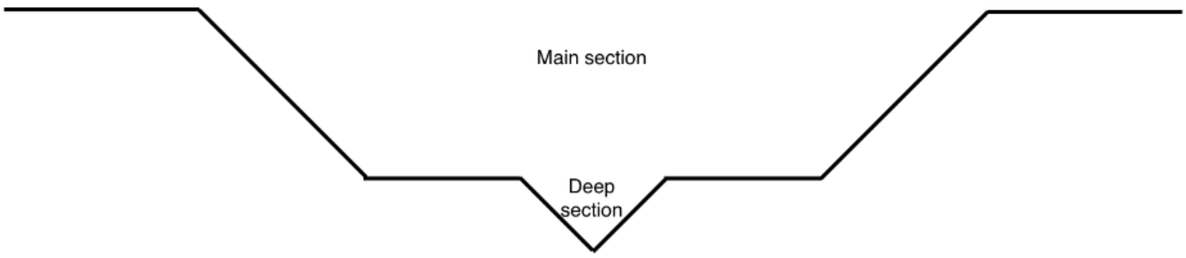


Figure 101: Sections New Kikuletwa South

Manning's Roughness Coefficient

The roughness coefficient of the New Kikuletwa South is determined using Figure 102. It is assumed that the new river is dragline-excavated or dredged and that the channel is not maintained with a clean bottom and brush on the sides. According to this description, the value for the roughness coefficient is set at 0.04 $s/m^{1/3}$ for the deep section and 0.05 $s/m^{1/3}$ for the main section and the deep section together.

4. Excavated or Dredged Channels	Minimum	Normal	Maximum
a. Earth, straight, and uniform			
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. gravel, uniform section, clean	0.022	0.025	0.030
4. with short grass, few weeds	0.022	0.027	0.033
b. Earth winding and sluggish			
1. no vegetation	0.023	0.025	0.030
2. grass, some weeds	0.025	0.030	0.033
3. dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. earth bottom and rubble sides	0.028	0.030	0.035
5. stony bottom and weedy banks	0.025	0.035	0.040
6. cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. no vegetation	0.025	0.028	0.033
2. light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. smooth and uniform	0.025	0.035	0.040
2. jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. dense weeds, high as flow depth	0.050	0.080	0.120
2. clean bottom, brush on sides	0.040	0.050	0.080
3. same as above, highest stage of flow	0.045	0.070	0.110
4. dense brush, high stage	0.080	0.100	0.140

Figure 102: Manning's n for excavated channels (Chow, 1959)

Bed Slope

First, the determination of the bed slope is explained. The bed slope of the New Kikuletwa South is determined using the elevation at the bifurcation and at the expected connection between the New Kikuletwa South and the Kikuletwa South. In Figure 103, the steps of the determination of the bed slope are shown. In the first step the surface level at the bifurcation is shown on the left and on the right the bottom level of the Kikuletwa South is given. Then in the second step, the elevation levels are lowered by the height of the New Kikuletwa South, h. In this way, the bed slope of the New Kikuletwa South is the same for different heights. This is, however, not possible because the bottom levels at the connection between the New Kikuletwa South and the Kikuletwa South have to be the same. Therefore, in step three the height of the New Kikuletwa South is only lowered at the bifurcation and at the connection the elevation stays the same. In this way, the bed slope is different for different heights of the New Kikuletwa South.

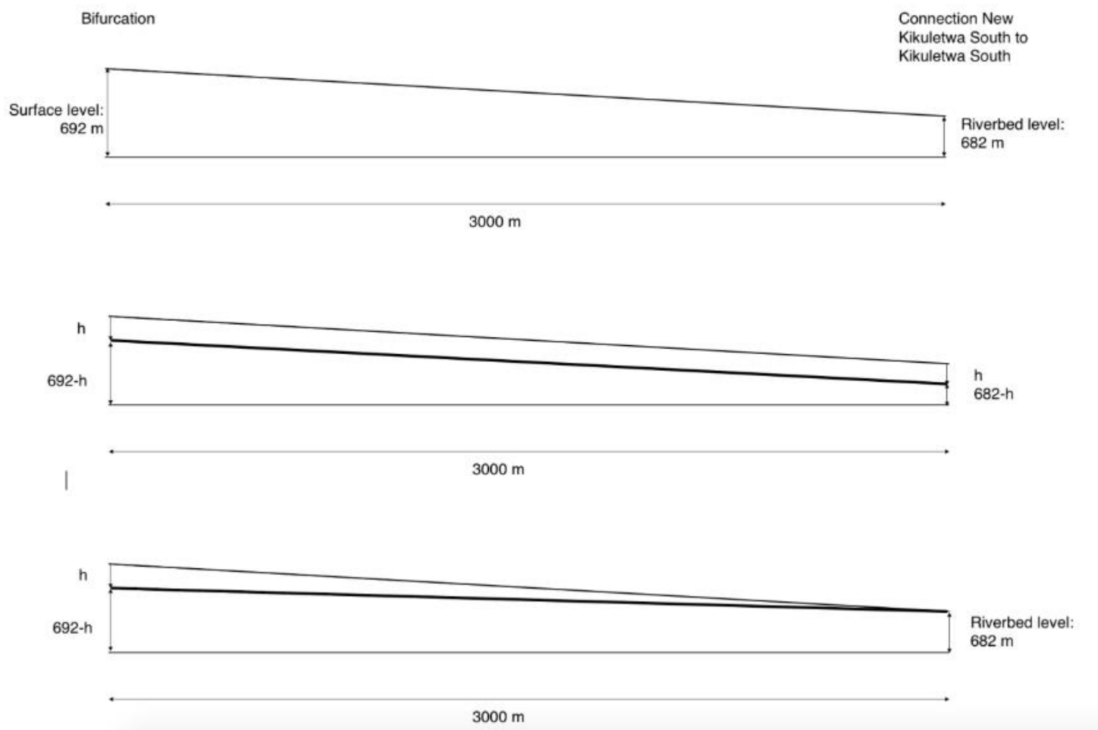


Figure 103: Determination bed slope

Calculations

The width and height of the new river are calculated using Manning's equation. The hydraulic radius and the cross-sectional area both depend on the width and height of the river. The bed slope only depends on the height of the river. The dimensions of the river are determined iteratively.

First, the dimensions of the deep section are determined. This section has to be designed for a discharge of 1 m^3/s. The assumed values and the determined dimensions of the deep section are presented in Table 98.

Table 98: Dimensions deep section

	Q (1/5)	Q (1/10)	Q (1/15)
i_b [-]	0.0022	0.0021	0.0021
n [s/m$^{1/3}$]	0.04	0.04	0.04
h [m]	1.0	1.0	1.0
b [m]	3.2	3.2	3.2
P [m]	3.8	3.8	3.8
A [m^2]	1.6	1.6	1.6
R [m]	0.4	0.4	0.4
U [m/s]	0.67	0.65	0.64
Q [m^3/s]	1.1	1.0	1.0

The dimensions of the main section have to be such that it can discharge the design discharge of the long rains of each return period, see Design discharge. For this section the slope of the riversides is chosen to be 1:2 (vertical: horizontal). The calculated height is the maximum water level that will occur during the long rains. To be safe an extra 0.2 meter has been added to this height, see Figure 104. Therefore, the top width of the river increases as well. The dimensions of the total cross-section (main section and deep section) are given in Table 99.

Table 99: Dimensions total cross-section

	Q (1/5)	Q (1/10)	Q (1/15)
i_b [-]	0.0022	0.0021	0.0021
n [s/m$^{1/3}$]	0.05	0.05	0.05
h (maximum water level) [m]	3.1	3.5	3.6
h (top main section) [m]	3.3	3.7	3.8
b (bottom main section) [m]	6.2	5.2	5.2
b (maximum water level) [m]	14.6	15.3	15.6
b (top main section) [m]	15.4	16.1	16.4
P [m]	16.1	17.1	17.4
A (maximum water level) [m^2]	23.4	27.5	28.6
A (total river) [m^2]	26.4	30.6	31.8
R [m]	1.5	1.6	1.6
U [m/s]	1.2	1.3	1.3
Q [m^3/s]	28	35	36

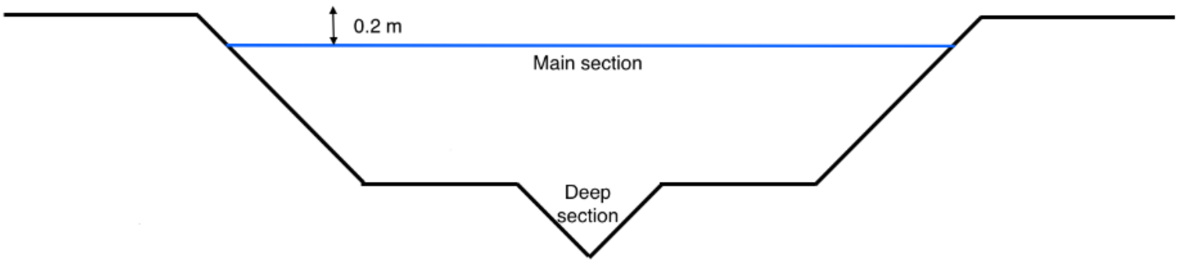


Figure 104: Maximum water level New Kikuletwa South

Appendix B.4 – Design Drafts Control Structure

As previous explained, the design of the total solution has been changed compared to the solution of the prefeasibility study, see Chapter 6 Validation. Consequently, the choice has been made to make a new design for the control structure at the bifurcation of the Kikuletwa South and the Ronga, which meets the requirements of the new solution. This appendix will elaborate the possible design drafts for the control structure. Different design possibilities will be treated, such as placing the control structure in different branches and making a passive or manually adaptable control structure. A flow chart of the design possibilities is shown in Figure 105.

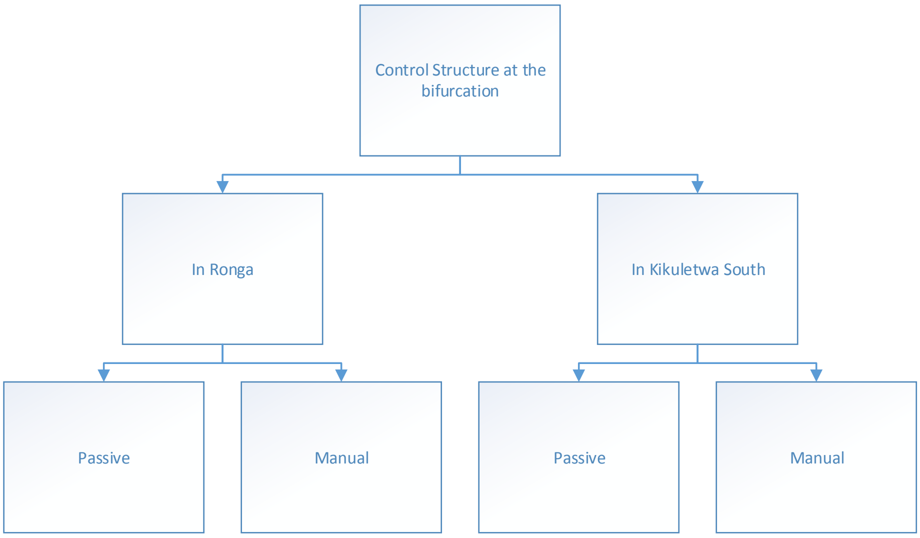


Figure 105 Flow chart Design Possibilities Control Structure Bifurcation

Requirements for the Control Structure

The control structure has two types of requirements, the discharge requirements and practical requirements. Both will be listed here ensuring a solution that meets the requirements.

Discharge Requirements

The bifurcation of the Kikuletwa South and the Ronga is an important location in the river system. Therefore, it is necessary that the control structure be designed in such a way that it is meeting the requirements for each period. The requirements for the dry, short rain and long rain period are given below.

Dry Season

In the dry period, there has to be an as large as possible discharge in the Ronga to enable agriculture along the Ronga, while there remains a minimum discharge of 1 m^3/s in the Kikuletwa South to avoid drying up of the riverbed. The drying up of a river will have as a result that sediment is deposited at the dry riverbed, this must be prevented in order to ensure a long service life of the solution.

Short Rain Season

In the short rain period, there is a certain discharge limit that may go through the Ronga in order to prevent flooding of the land along the Ronga. As a result, the Kikuletwa South must therefore be able to provide for the remaining discharge.

Long Rain Season

In the long rain season, there is a certain discharge that has to go into the Ronga to ensure flooding of the land along the Ronga. The flooding of this land is necessary to flush the soils in order to ensure the success of agricultural activities. Because the discharge in the long rain season is highly variable it is difficult to make sure enough discharge is available for the Ronga. In this case, the years that the peak discharge is low are important for the design. The Ronga should flood even in long rain seasons with low peak discharges. The Kikuletwa South should transport the remaining discharge.

Practical Requirements

For the practical requirements, constructability and operation are important. A local contractor with available material should build the control structure. Complex regulating structures will not be considered. For the operation, it is important that local farmers or other villagers can operate potential regulating structures.

Discharge Information

To arrive at the correct solution an indication of the design discharges is given in Table 100. These are the discharges that would flow through the Kikuletwa South and the Ronga for the short and long rain periods after the solution. The indication is based on the design discharges calculated in chapter 8 Design Discharges.

Table 100: Design discharges Ronga and Kikuletwa South

	Returning Period	Kikuletwa South	Ronga	Arriving at the Bifurcation
Discharge Long Rains [m^3/s]	1/5	28	43	71
	1/10	34	43	77
	1/15	36	43	79
Discharge Short Rains [m^3/s]	1/5	14	25	39
	1/10	18	25	43
	1/15	20	25	45

The discharge in the long rain season is highly variable. The years with low peak discharges should be taken into account when designing to ensure that the Ronga floods during these years. To determine the discharge that is normative in the described situation the long rain return period graph is used, see Appendix A.4 - Design Discharges. It is essential that the Ronga floods every year to ensure fertile lands. Therefore, a discharge of 45 m^3/s is used, this discharge occurs at least once every year. This is the discharge measured at the IDD1 station, but with this discharge the Kikuletwa North does not flood and therefore the same discharge arrives at the bifurcation.

In the dry season, there will be enough discharge to prevent that one of the riverbeds runs dry. This is the case assuming that there is a way for the water to flow into the river branches. A low water level in the Kikuletwa North should be taken into account.

Different Options for the Control Structure

Now some options for the control structure will be listed following the flow chart, see Figure 105. Creativity and inventiveness will be key elements in the design. Some options might be unrealistic but it is important to consider as many options as possible. For every solution, the requirements met and requirements not satisfied will be listed following the requirements listed before.

Ronga Passive

For this variant a simple sill with pipes at the bottom of the sill is considered, see Figure 106.

Requirements met:

- During the dry season, flow is possible through the pipes, preventing the river from drying up.
- The height of the sill should be designed for the design discharge of the short rains. This control structure will prevent flooding in the Ronga during the short rains.
- The control structure can be built by local contractors with local material. This was concluded after meeting with local contractors.
- The construction is passive so no complex regulating structures are present.

Requirements not satisfied:

- During the long rains, it cannot be guaranteed that the Ronga will receive enough discharge to flood. This is because the Kikuletwa South is not controlled. There is no easy solution possible for this problem because the peak discharge of the long rains is highly variable.

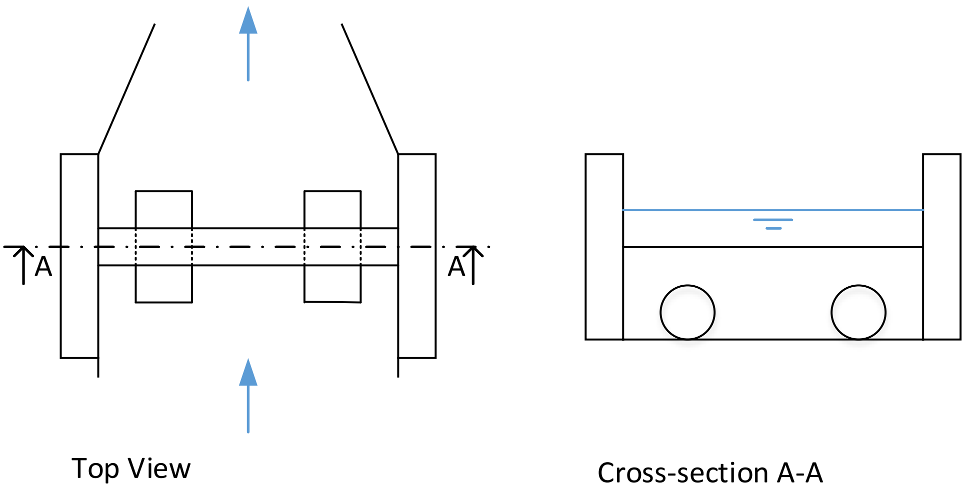


Figure 106: Design drawings passive control structure in the Ronga

Ronga Manual with Doors

For this variant a big sill with doors at the side is considered, see Figure 107.

Requirements met:

- During the dry season, flow is possible through the doors, which can be opened, preventing the river from drying up.
- The height of the sill should be designed for the design discharge of the short rains. This control structure will prevent flooding in the Ronga during the short rains.

Requirements not satisfied:

- During the long rains, it cannot be guaranteed that the Ronga will receive enough discharge to flood. This is because the Kikuletwa South is not controlled. There is no easy solution possible for this problem because the peak discharge of the long rains is highly variable.
- It is probably not possible to build this structure with the knowledge of the local contractors.
- The regulating structures are complex and it might not be possible to close the doors while water is flowing through the gaps.

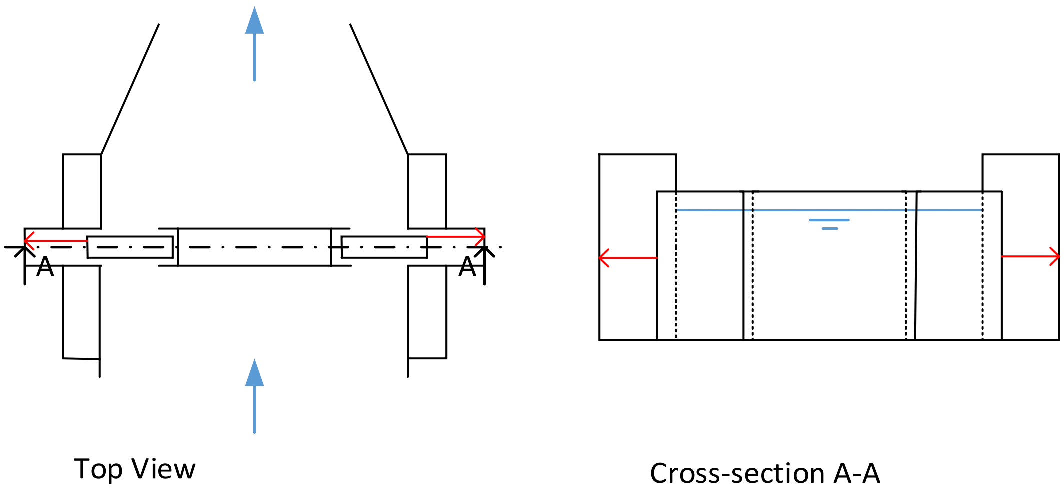


Figure 107: Design drawings manual control structure with doors in the Ronga

Ronga Manual with Weirs

For this variant a big sill with vertical slide gates at the side and a triangular gap in the centre is considered, see Figure 108.

Requirements met:

- During the dry season, flow is possible through the weirs, which can be opened, preventing the river from drying up.
- The height of the sill should be designed for the design discharge of the short rains. This control structure will prevent flooding in the Ronga during the short rains.
- The control structure can be built by local contractors with local material. This was concluded after a meeting with local contractors.

Requirements not satisfied:

- During the long rains, it cannot be guaranteed that the Ronga will receive enough discharge to flood. This is because the Kikuletwa South is not controlled. There is no easy solution possible for this problem because the peak discharge of the long rains is highly variable. Although the triangular gap will ensure that more water will go through the Ronga, for very low peak flows in certain years the Ronga will not overflow in the long rain season.
- The regulating structures are not that complex but it might not be possible to close the doors while water is flowing through the gaps.

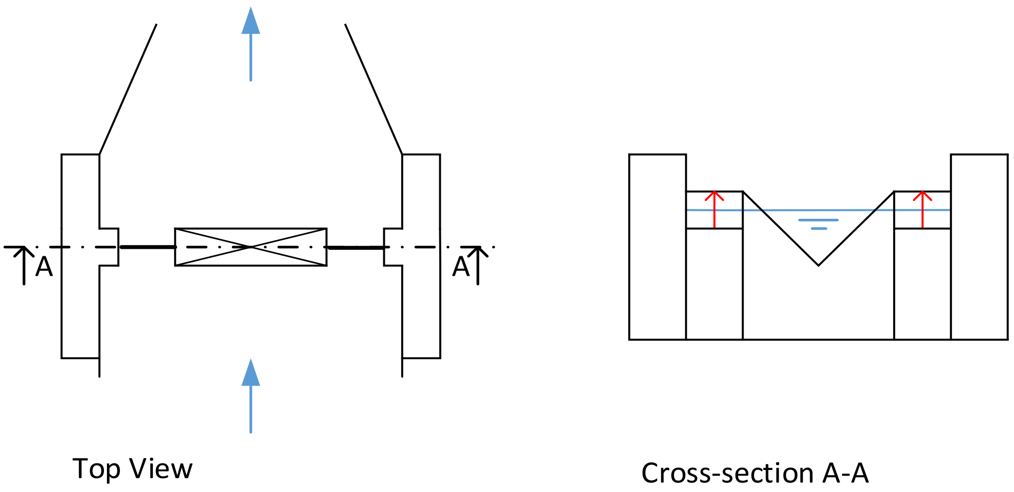


Figure 108: Design drawings manual control structure with weirs in the Ronga

Kikuletwa Passive

For this variant a big sill with a triangular gap in the centre is considered, see Figure 109.

Requirements met:

- During the dry season flow, is possible through the v shaped sill, preventing the river from drying up.
- The height of the sill should be designed for the design discharge of the short rains. Even with the control structure, the Kikuletwa should be able to discharge enough of the incoming water. This in order to prevent flooding in the Ronga during the short rains. To prevent the Ronga from being flooded in extreme years during the long rains even bigger discharges should pass the control structure, see Figure 109.
- Local contractors with local material can build the control structure. This was concluded after a meeting with local contractors.
- The construction is passive so no complex regulating structures are present.

Requirements not satisfied:

- With this structure, there is a trade-off. If it is designed to have enough capacity to prevent flooding during the short rains, flooding during the long rains will not be guaranteed. If it is designed to make sure that the Ronga floods every year, it is also possible that the Ronga will flood during the short rains. This is because the long rain discharge in low discharge years is equal or almost equal to the design discharge during the short rains. To clarify, the discharge (arriving at the bifurcation) for years with low peak discharge is 45 m^3/s like mentioned before and the design discharge during the short rains with a return period of 1/15 years is also 45 m^3/s. This imposes a problem that cannot be solved with a passive structure in the current situation.

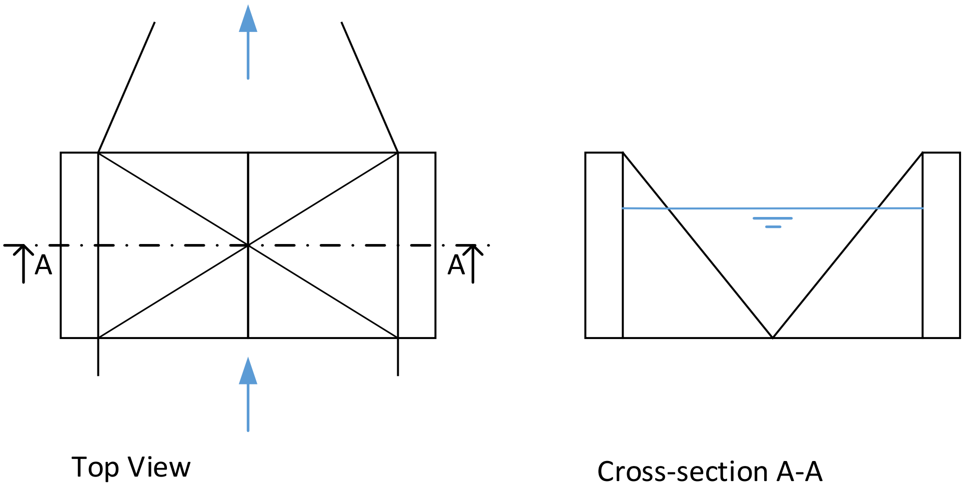


Figure 109: Design drawings passive control structure in the Kikuletwa

Kikuletwa Manual Moving Gate

There are a lot variations possible for a manual control structure with a moving gate here we will consider two. A variant with a hole in the gate, see Figure 110and a variant with a v shaped top, see Figure 111.

Requirements met:

- During the dry season, flow is possible through the hole in the gate or over the v shaped gate when the gate is positioned correctly, preventing the river from drying up.
- The height of the gate can be adjusted this makes it possible to design for both the long and the short rains for different discharges. During the short rains or high discharge long rains, the gate can be opened to make sure that the Ronga does not flood.
- During low discharge long rains, the gate can be closed to make sure that the Ronga has its flooding period.

Requirements not satisfied:

- It is probably not possible to build this structure with the knowledge of the local contractors. These are very complex designs where modern technology and equipment are required.
- The regulating structures are complex and it might not be possible to close the doors while water is flowing through the gaps. Besides that, the technology to open and close the gate might not be available locally.

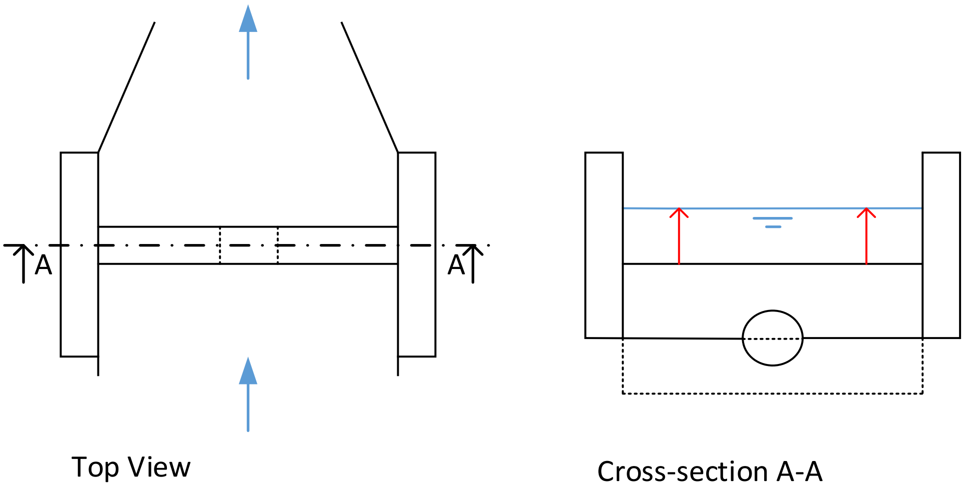


Figure 110: Design drawings manual control structure in the Kikuletwa, variant with hole

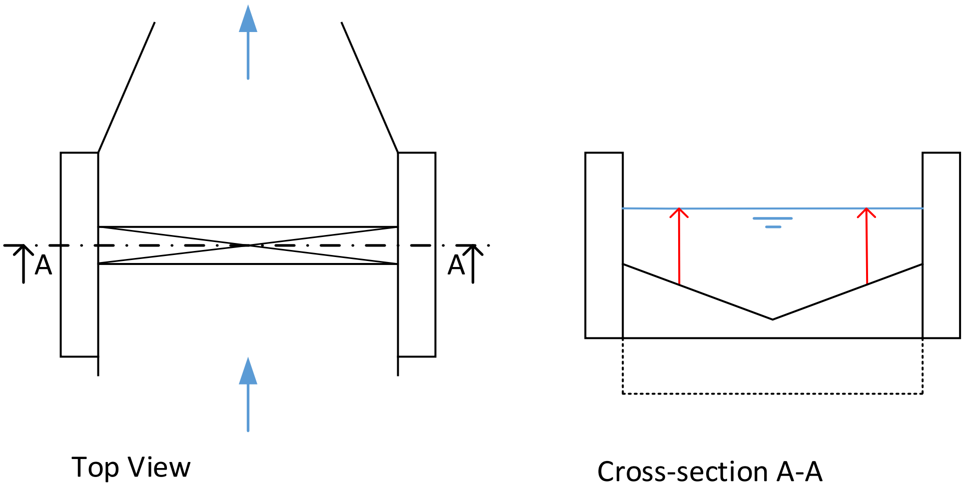


Figure 111: Design drawings manual control structure in the Kikuletwa, variant with v shape

Appendix B.5 - Total Gate Width Control Structure

In order to determine the total width of the gates in the control structure some calculations are done. The calculations are described here. For the determination of the width, the maximum discharge is normative. Therefore, the maximum discharge to the Kikuletwa South for every return period will be used. The discharges can be found in Table 100.

Flow Speed Calculation

First, the flow speed through the control structure must be determined. The calculation is based on the hydraulic head balance; the situation is visualized in Figure 112. The hydraulic head balance can be seen in equation 26.

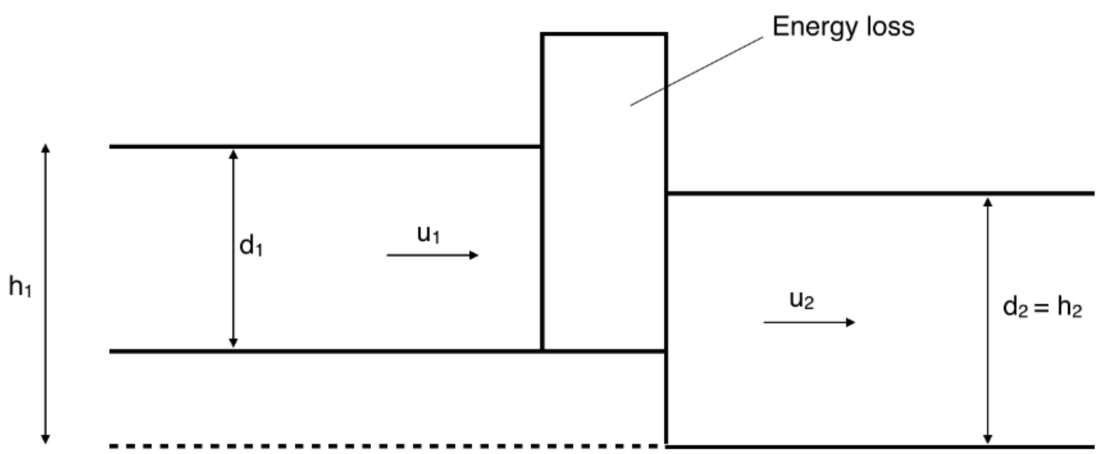


Figure 112: Visualization of the hydraulic head balance

$$h_1 + \frac{u_1^2}{2g} = h_2 + \frac{u_2^2}{2g} + z \qquad (26)$$

Where: - h_1 is the elevation head on the left side, including the water depth and the height difference [m]
- u_1 is the flow velocity on the left side [m/s]
- h_2 is the elevation head on the right side (equal to the depth) [m]
- u_2 is the flow velocity on the right side and through the control structure [m/s]
- z is the total headloss [m]

The elevation head on the left side depends on the water level in the Kikuletwa North and the height difference of the bottom, see Table 101. The height difference is formed due to the excavation works in the Kikuletwa North. In this stadium of the design, the height difference is estimated.

Table 101: Elevation head Kikuletwa North

Return period [1/years]	Water level [m]	Height difference bottom [m]	Total [m]
1/5	2.65	1	3.65
1/10	2.76	1.4	4.16
1/15	2.79	1.4	4.19

The flow velocity on the left side can be determined using the discharge, the water level and the width of the river. The width of the river is assumed to be the width of the Kikuletwa North, which is 25 meters. The discharge is divided by the cross-sectional area of the river to obtain the flow velocity, see Table 102.

Table 102: Flow speed Kikuletwa North

Return period [1/years]	Cross-sectional area [m^2]	Discharge [m^3/s]	Flow speed [m/s]
1/5	66.25	28	0.4
1/10	69	34	0.5
1/15	69.75	36	0.5

The elevation head on the right side is assumed to be equal to the maximum water level in the Kikuletwa South, which can be found in chapter 11 Cluster 2.

The total head loss over the control structure can be determined using equation 27.

$$z = \left(c_{in} + c_{out} + c_f\right) * \frac{u_2^2}{2g} \tag{27}$$

Where: - c_{in} is the headloss coefficient for inlet [-]

- c_{out} is the headloss coefficient for outlet [-]
- c_f is the headloss coefficient for friction [-]

The inlet and outlet coefficients can be determined using Figure 113. The rectangular transition is used for this situation. The head loss coefficient for friction can be determined using equation 28 (Ankum, 2002).

Type of transition	headloss coefficients over inlet	headloss coefficients over outlet
• warped transition	$c_{in} = 0.1$	$c_{out} = 0.2$
• rounded transition	$c_{in} = 0.2$	$c_{out} = 0.4$
• straight-line transition	$c_{in} = 0.3$	$c_{out} = 0.5$
• rectangular transition	$c_{in} = 0.5$	$c_{out} = 1.0$

Figure 113: Head loss coefficients for inlets and outlets (Ankum, 2002)

$$c_f = x * \frac{2gL}{k^2 R^{\frac{4}{3}}} \tag{28}$$

Where: - x is the number of gates [-]

- L is the length of the structure [m]
- k is the Strickler coefficient for concrete [m$^{1/3}$/s]
- R is the hydraulic radius [m]

The number of gates is estimated on four and the length of the structure on 1.5 meter. The Strickler coefficient for concrete is 70 m$^{1/3}$/s (Ankum, 2002). The hydraulic radius should be determined iteratively with the width of one gate. In order to make the calculation not more complicated than necessary, the hydraulic radius is approximated at 0.75 meter. This can be justified because the friction coefficient has very little influence on the total calculation. The head loss coefficient for friction is equal to 0.035, compared to the head loss coefficient for inlet of 0.5 and the head loss coefficient for outlet of 1.0.

All the variables are determined and the flow speed through the structure can be calculated. The result is displayed in Table 103. The maximum possible flow speed is also included in this table. The maximum indicates for what flow speed the flow becomes super critical, see formula 10 Appendix B.2 - Design Calculations Spillways. To prevent additional energy loss, the actual flow speed should be lower than the maximum flow speed.

Table 103: Flow speed and max flow speed through the control structure

Return period [1/years]	Flow speed through the structure [m/s]	Max flow speed [m/s]
1/5	2.08	2.55
1/10	2.28	2.60
1/15	2.16	2.61

Width Calculation

The flow speed is lower than the super critical boundary and is therefore used to calculate the needed width of the control structure. This is done with equation 29.

$$b = \frac{Q}{d_1 u_2} \tag{29}$$

The calculated widths for each return period are displayed in Table 104. The calculated widths are comparable; therefore, the final total width that will be used for the initial design of the control structure is the same for each return period. In a later stadium of the design, the width can be determined more accurately. The safety factors for each return period are also included in Table 104.

Table 104: Total calculated gate width, final total width and safety factor

Return period [1/years]	Total width [m]	Final total width [m]	Safety factor [-]
1/5	5.1	7.5	1.47
1/10	5.4	7.5	1.39
1/15	6.0	7.5	1.25

Appendix B.6 - Failure Mechanisms Control Structure

By performing unity checks on the possible failure mechanisms, it is possible to determine whether the assumed dimensions are good. This iterative process is repeated until the dimensions are such that the structure is safe for the considered failure mechanisms. Presented in this chapter are all the unity checks of the failure mechanisms of the final dimensions.

Loads

The loads have been calculated per meter of the foundation. The following constants have been used, see Table 105. They are the same as in Appendix B.2 - Design Calculations Spillways. Some loads are present but have not been determined. This is because they are not representative for failure mechanisms of the structure.

Table 105: Constants control structure

Constant	Unit	Value
Steel density	[kN/m^3]	78
Concrete density	[kN/m^3]	25
Soil density	[kN/m^3]	16
Gravitational acceleration	[m/s^2]	9.78
Water density	[kg/m^3]	1,020

For the water levels the following values have been taken. These were determined in chapter 8 Design Discharges and chapter 11 Cluster 2. The water depths can be seen in Table 106.

Table 106: water depths control structure

Water depths [m]	Q (1/5)	Q (1/10)	Q (1/15)
Maximum Kikuletwa South side	3.1	3.5	3.6
Maximum Kikuletwa North side	2.65	2.76	2.79
Maximum for closed gates	2	2	2

Horizontal Loads

The horizontal load consists of water pressure as indicated in Table 107.

Table 107: Horizontal load control structure

	Q (1/5)	Q (1/10)	Q (1/15)
Water pressure [kN/m]	61	76	76

Vertical Loads

For different failure mechanisms different loads are taken as representative. All the possible vertical loads are the following. The values are found in Table 108.

- Concrete total
  - Foundation;
  - Supporting structure;
  - Gate holders.
- Water pressure
  - Under the structure;

- On top of the structure, either when the gates are closed and there is a high water level or when the gates are open and there is a high water level.

- Soil on top of the foundation
- Gates
  - Concrete gates;
  - Equipment to manage gates.

Table 108: Vertical loads control structure

Load [kN/m]	Q (1/5)	Q (1/10)	Q (1/15)
Concrete total	410	450	450
Water gates closed			
Water under structure	314	350	350
Water top of structure	30	30	30
Water gates open			
Water under structure	696	777	789
Water top of structure	534	600	616
Soil	152	152	152
Gates			
Concrete gates	-	-	-
Equipment	13	17	17

Moments

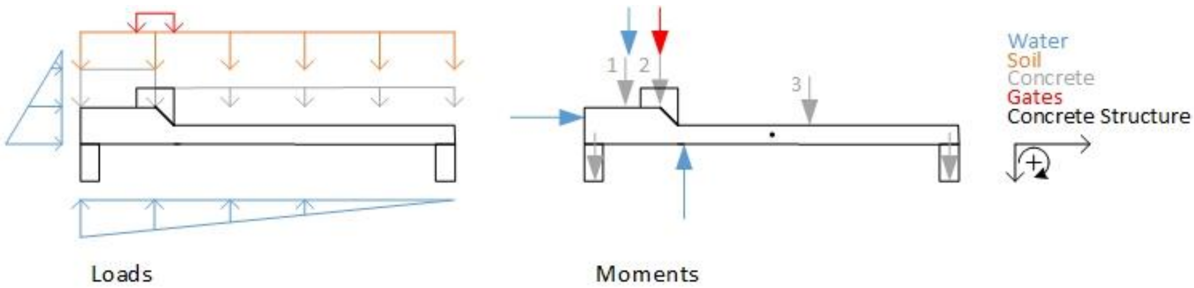


Figure 114: Moments control structure

In Figure 114 the moments are visualised. The moments of the different parts of the structure have been determined and can be read in Table 109.

Table 109: Moments control structure

Moment [kNm/m]	Q (1/5)	Q (1/10)	Q (1/15)
Horizontal water pressure	+71	+99	+99
Concrete total	-769	-1027	-1027
Water gates closed			
Water up	+943	+1050	+1050
Water down	-247	-247	-247
Water gates open			
Water under structure	-	-	-
Water top of structure	-	-	-
Soil	0	0	0
Gates			
Concrete gates	-	-	-
Equipment	-90	-110	-110

Stability Checks

The following stability checks have been done using the Manual Hydraulic Structures (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015).

Horizontal Stability

Determination

The horizontal stability is calculated by comparing the total horizontal load to the total vertical load, see equation 30.

$$\sum H \leq f \sum V \tag{30}$$

Where: - $f = \tan\left(\frac{2}{3}\varphi\right)$

- φ was determined to be 33° in drained conditions, see Appendix D.1 - Soils.

Load Combination

For the horizontal stability, the most unfavourable loading situation is when the least amount of vertical force during operation when there is horizontal water force acting against the gates. The representative loads can be seen below in Table 110.

Table 110: Loads horizontal stability

Horizontal	Vertical
Water pressure	Concrete total Water pressure under the structure and on top of the structure for closed gates Soil on top of the foundation

The weight of the gates and their operation equipment is not included, as this would increase the safety against horizontal stability. The gates are assumed to be closed, as in this situation there is a horizontal force but not a vertical downward force over the entire structure. The water level on the other side is assumed to be zero.

Result

The results can be found in Table 111 below.

Table 111: Horizontal stability control structure

	Q (1/5)	Q (1/10)	Q (1/15)
f [-]	0.4	0.4	0.4
$\sum H$ **[kN/m]**	61	76	76
$f \sum V$ **[kN/m]**	112	114	114
Unity check	**1.83**	**1.49**	**1.49**

The structure is safe against horizontal stability.

Rotational Stability

Determination

The rotational stability is calculated by comparing the vertical loads to the total moments, see equation 31.

$$\frac{\sum M}{\sum V} \leq \frac{L}{6} \tag{31}$$

Where: - $\sum M$ is the total moment around the middle of the foundation [kNm/m]
- $\sum V$ is the total vertical load [kN/m]
- L is the length of the foundation [m]

Loading Combination

The most unfavourable situation in this case is when there are no water pressure forces acting on the structure. This would be, for instance, during construction. This is when there is a lowest possible vertical load acting down. For the moment, the equipment for the gates have been added since this increases the negative moment causing rotation. The loads can be found in Table 112.

Table 112: Loads rotational stability

Moments	Vertical loads
Concrete total	Concrete total
Equipment gates	Soil on top of the foundation

Result

The results of the stability check can be found in Table 113.

Table 113: Results Rotational stability

	Q (1/5)	Q (1/10)	Q (1/15)
$\sum M$ [kNm/m]	-859	-1138	-1138
$\sum V$ [kN/m]	562	601	601
$\sum M/\sum V$ [m]	1.53	1.89	1.89
L/6 [m]	3	3	3
Unity Check	**1.96**	**1.59**	**1.59**

The structure is safe against rotational stability.

Uplift

Determination

Equation 32 can be used to determine if the structure is safe against uplift.

$$\frac{\sum V_{downward}}{\sum V_{upward}} \geq 1 \tag{32}$$

Where: - $\sum V_{downward}$ is the total force downward [kN/m]
- $\sum V_{upward}$ is the total force upward [kN/m]

Loading Combination

This check is done to determine if the structure will lift up; meaning if the structure is heavy enough to counteract the water pressure forces underneath the structure. The most unfavourable situation is when the gates are open. This results in a high upward force along the structure, see Table 114.

Table 114: Loading uplift

Vertical forces down	Vertical forces up
Concrete total	
Soil	
Water on top of structure at extreme water levels	Water pressure acting up below structure at extreme water levels

Result

In Table 115 the unity checks against uplifting for every return period is given.

Table 115: Results uplift

	Q (1/5)	Q (1/10)	Q (1/15)
$\sum V_{downward}$ **[kN/m]**	1097	1201	1218
$\sum V_{upward}$ **[kN/m]**	696	777	789
Unity check	**1.58**	**1.55**	**1.54**

The structure is safe against uplifting.

Vertical Stability

Determination

The vertical stability has been tested by comparing the load acting on the soil with the bearing capacity of the soil, see equation 33. The bearing capacity ($\sigma_{k,max}$) of silt is estimated to be 70 kPa (13).

$$\sigma_{k,max} = \frac{F}{A} + \frac{M}{W} = \frac{\sum V}{bl} + \frac{\sum M}{\frac{1}{6} l b^2} \leq 70 \tag{33}$$

The minimum bearing capacity can also be determined, see equation 34. This minimum value should be larger than zero.

$$\sigma_{k,min} = \frac{F}{A} - \frac{M}{W} = \frac{\sum V}{bl} - \frac{\sum M}{\frac{1}{6} l b^2} \geq 0 \tag{34}$$

Where: - F is the vertical force [kN]

- A is the cross-sectional area [m^2]
- M is the sum of the moments [kNm]
- W is the Section modulus [m^3]
- b is the length of the foundation[m]
- l is the width of the foundation in [m]

Since the equations are already per meter width of the foundation they are only divided by the length. This is equal to the letter b in the equation.

Loading Combination

The most unfavourable situation in this case is also during construction when there are no water forces acting on the structure. This is when the sum of the moments is largest. This is representative in this case. The moments and vertical loads can be found in Table 116.

Table 116: Loading vertical stability

Moments	Vertical
Concrete total	Concrete total
Gates	Gates
	Soil on top of the foundation

Results

In Table 117 the unity checks for vertical stability are given.

Table 117: Results vertical stability

	Q (1/5)	Q (1/10)	Q (1/15)
$\sum V$ [kN/m]	576	618	618
$\sum M$ [kNm/m]	859	1138	1138
$\frac{\sum V}{bl}$ [kN/m²]	32	34	34
$\frac{\sum M}{\frac{1}{6} lb^2}$ [kN/m²]	16	21	21
Total Max	48	55	55
Total Min	16	13	13
Unity Check Max	**1.46**	**1.26**	**1.26**

The unity check for the maximum is safe. The total for the minimum is also larger than zero. Therefore, the structure is safe against vertical stability.

Piping

Bligh

Bligh's method can be used to see if the length of the structure is larger than the seepage length. Bligh's method states that failure occurs if the condition in equation 35 is not met.

$$L > \gamma \times \Delta H \times C_{bligh} \tag{35}$$

Where: - L is the leakage length (the horizontal length and the vertical length) [m]
- γ is a safety factor [-]
- ΔH is the flooding height [m]
- C_{bligh} is the creep factor of Bligh [-]

Since the high water level is not constant, a safety factor of one is considered sufficient. A layer beneath the foundation of gravel is assumed, which gives a creep factor of five. The results can be seen in Table 118.

Table 118: Results Bligh

	Q (1/5)	Q (1/10)	Q (1/15)
Seepage length [m]	15	17	17
Actual length [m]	26	26.4	26.4
Unity check	**1.73**	**1.55**	**1.55**

The actual length is larger than the seepage length of Bligh. Therefore, the structure is safe against piping.

Lane

Lane method states that failure occurs if the condition in equation 36 is not met.

$$L > \gamma \times \Delta H \times C_{Lane} \tag{36}$$

However, L is not an addition anymore of the horizontal and vertical path, but is given with equation 37.

$$L = \sum L_{vert} + \sum \frac{1}{3} L_{hor} \tag{37}$$

- L is the leakage length[m]
- γ is a safety factor [-].
- ΔH is the flooding height [m]
- C_{Lane} is the creep factor of Lane [-].

γ *and* ΔH stay the same as Bligh, C_{Lane} is four for gravel.

The results can be found in Table 119.

Table 119: Results Lane

	Q (1/5)	Q (1/10)	Q (1/15)
Seepage length [m]	12	13.6	13.6
Actual length [m]	14	14.4	14.4
Unity check	**1.16**	**1.05**	**1.05**

The actual length is larger than the seepage length of Lane. Therefore, the structure is safe against piping.

Appendix C – Cost-Benefit Analysis

Appendix C.1 - Determining the Present Value

In order to be able to compare the cost and benefits over the lifespan of the solutions, their present values will need to be determined. In this appendix, the different variables used for this calculation will be discussed.

Time Horizon

The lifespan of 20-25 years for a structure in Tanzania was estimated and agreed upon by the different stakeholders during the first introduction meeting. With proper maintenance, it should be possible to maintain and keep the structures functioning for 25 years. As such, the cost and benefits for this timespan will be calculated.

Discount Rate and Inflation

A discount rate is used in order to calculate the present value of future cash flows. The rate is normally determined based on estimated risks, inflation, interest and the required rate of return. Furthermore, depending on the type of project a commercial or social discount rate is used. The different considerations will be discussed and the discount rate used presented.

Characteristics of the Project

The project needs to improve the situation in the Lower Moshi region with the potential benefits being higher than the cost of the project. The executive director of FT Kilimanjaro, G. Rieks has said that the payback period should be between the 10 and 20 years. Other than this, there is no required rate of return for the project.

Concerning the funding, all funds are expected to come from donors; no loans will need to be taken in order to finance the project. Furthermore, the benefits of the project will not end up with FT Kilimanjaro, but rather the local villagers and farmers who will profit from the solution. As such, no commercial approach can be taken to determine the discount rate for this project, but rather a social discount rate or one used in reference projects.

Reference Projects

In order to gain a good estimation what an appropriate discount rate would be for this project, reference projects were searched for and stakeholders were asked. Unfortunately, reference projects were not found.

Regarding the stakeholders, FT Kilimanjaro has no past experience using the present value or discount rates to evaluate their projects nor had they knowledge if FEMI did.

Another stakeholder contacted an economist at the World Bank, P. Kriss, who recommended the use of 8% for the discount rate and to use the global inflation. The World Bank uses the same discount rate to evaluate similar projects.

This suggestion was supported by J. de Vries of FTK/FEMI and G. Rieks of FTK. As such, this discount rate will be used to calculate the present value of the cost and benefits.

Inflation

The inflation used to calculate the future cost and benefits has been determined on the basis of the historic data for the global inflation of the past 40 years (World Bank, 2015) and the prediction for the coming six years (PWC, 2015).

Based on this data a 95% confidence interval was calculated for the inflation. This results in an inflation within a range of 3.74% and 5.21% with an average of 4.47%. The average inflation will be used for the calculations and the range for the sensitivity study.

Risks

As a given discount rate will be used, it will only be checked if there is a large enough margin to take into account the risks. The discount rate will consist of the inflation and the risk in this situation. With the chosen inflation and discount rate there will be a margin of 3.53% for potential risks. The risks, fluctuations in inflation or the uncertainties related to future cost and benefits can be taken into account in this way.

Conclusion

A discount rate of 8% and an inflation of 4.47% will be used for the calculations of the present value for the cost and benefits.

Appendix C.2 - Monetary Benefits of the Designs

To determine the potential benefits of the project the *"Impact Assessment Flood Control"* by (Rieks- van Hal, 2015) be used as source for the data. Different parameters will be used to translate the identified impact to a long-term estimation for the benefits for the region. In this appendix, only the monetary benefits will be discussed.

Agricultural Benefits

There are two types of agricultural benefits, the prevention of the loss of crops and the increase in farmland utilisation.

The effects of the solution on the agriculture are different for the short and the long rain seasons. During both seasons, the floods damage the crops on the fields. As result of the solution, the flooding during the short rain season will be prevented while the chance of an unwanted long rain flood will decrease.

First, the impacted farming regions will be discussed. Secondly the benefit of preventing the loss of crops and lastly the increase farmland utilisation.

Impacted Farming Regions

In the impact assessment, general regions were given where farming takes place and which are influenced by the floods. The farmland is divided between three regions, Samanga, Ronga and the Kikuletwa South. In these regions, there are roughly 500 acres of land that are regularly cultivated. In Figure 115 an impression of the location of the different regions can be seen. The other areas are located at higher ground and do not experience long periods of floods.

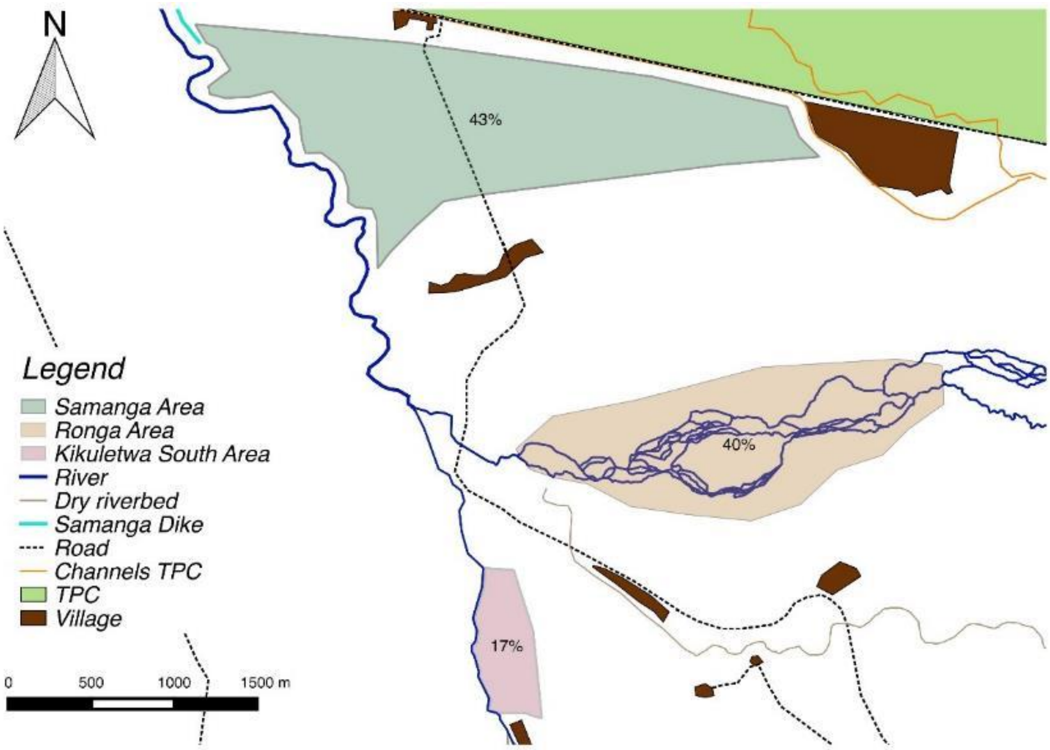


Figure 115: Farming regions influenced by floods

In Table 120 the division of the farmland between the different regions is given and during which rain seasons they typically flood. The Kikuletwa South region is protected by higher banks and experiences no negative effect on the farmlands because of the flooding (Lower Moshi (2015)).

Table 120: Impacted farmland regions

Region	% of total farmland in Lower Moshi	Short rain floods	Long rain floods
Samanga	43	No	Yes
Ronga	40	Yes	Yes
Kikuletwa South	17	No	No

Prevention of Crop Loss

The loss of crops is caused by the unexpected arrival of floods before the farmers can harvest their crops. In Table 121 the loss as identified in the impact assessment is shown. As can be seen in the table the majority of the damage is done during the short rain season.

Table 121: Loss of crops

Rain season	Percentage of total loss [%]	Area effected [acres]	Current Loss	
Short rain	73	147	TZS 637,828,000	$ 292,000
Long rain	27	54.5	TZS 235,909,000	$ 108,000
Total	**100**	**201.5**	**TZS 873,737,000**	**$ 400,000**

The solution will be able to handle all the short rain floods and reduce the number of unexpected long rain floods. This benefit will be taken into account from the moment the construction work is completed.

Increased Farmland Utilisation

As is mentioned before, there are two rain seasons, the long rain and short rain. These rain seasons have an influence on the number of harvests a farmer can do in a year time. The periods for the rain seasons can be seen in Table 122.

Table 122: Rain seasons

Short Rain	Dry	Long Rain			Waiting Period	Dry			Short Rain		
January	February	March	April	May	June	July	August	September	October	November	December

For the short rain season it does not mean that it always rains in these months, but rather that there is a chance on rainfall and floods. In June, the farmers wait for the water levels on their fields to drop before they start working their fields.

Table 123: Time requirement per harvest

Number of harvests	Time required [months]
1	3-4
2	5
3	8+

The number of possible harvests depends on the required time and certainty that the farmlands do not flood. The required time for harvests can be seen in Table 123.

Preventing the short rain floods will allow farmers in the affected areas to do one extra harvest. To make it possible that farmers harvest three times in a year, the period that the long rains affect the region needs to decrease.

For irrigation purposes and decreasing the salinity of the ground, the farmland needs to be flooded for a period of minimal 4 weeks during the long rain season. This is based on the current length of the flooding time. The problem is that after this period in the current situation, there is a still a chance on more flooding and it takes time for the land to dry enough to make sowing possible.

By managing the floods, the possibility will be created for the farmers to do an extra harvest in a years' time. In Table 124 the number of acres can be seen of farmland that is currently not being used during the rain seasons. When the farmers gain trust in the solution, they have said that they are planning to use that land for farming, creating an extra benefit.

Table 124: Potential for extra farmland

	Potential extra farmland			**Sell price of crops per acre**	
	Short rain [acres]	**Long rain [acres]**	**Total [acres]**		
Samanga	0	135	135	TZS 3,196,000	$ 1460
Ronga	69.5	125	194.5	TZS 3,196,000	$ 1460
Total	**69.5**	**260**	**329.5**		

It is not realistic to assume that all the land will be used for farming right after the construction has finished. Furthermore, it needs to be taken into account that extra costs are being made when more land is farmed. Thus, the following assumptions have been made to account for this:

- After construction is completed, an initial 25% extra land during the short rain season will be used. Then it will gradually increase to 100% of the land in 10 years' time.
- For the long rain season, an initial 10% extra land will be used in year 3 after completion. After that, it will take 20 years before all the land is used. The reason for this is that it will take some time for the farmers to gain confidence in the solution, as there is still a chance that it might flood.
- Most of the labour and equipment used for farming comes from within the region; therefore, it is assumed that only 5% of the selling price will be used to buy things outside the region.

Social Benefits

The social benefits are divided into two categories, prevention of damage to assets of the locals and the loss of salary/income.

Prevention of Damage to Assets

The damage to assets is often caused during the long rain season and it consists of damage to schools, clinics, houses and goods of entrepreneurs.

Prevention of Loss of Salary/Income

The second category is influenced by the length of the long rain floods. When the region is flooded, areas become inaccessible. Employees of the schools and clinics are not able to go to their work, while they keep being paid, creating a financial loss for the region. Another element is that the entrepreneurs are not able to travel, causing a loss of income for them.

The benefit of managing the long rain floods based on the yearly social damage can be found in Table 125. The assumption is made that the long rain floods are reduced to two months, as it is hard to predict how well the floods can be managed at this moment.

Table 125: Yearly social benefit

	Damage to assets	Loss salaries social services	Income loss entrepreneurs	Total	
Samanga	TZS 2,000,000	TZS 1,800,000	TZS 3,200,000	TZS 7,000,000	$ 3,200
Ronga	TZS 6,000,000	TZS 833,000	TZS 3,600,000	TZS 10,433,000	$ 47,800
Total	**TZS 8,000,000**	**TZS 2,633,000**	**TZS 6,800,000**	**TZS 17,433,000**	**$ 51,000**

Appendix C.3 - Non-Monetary Benefits

There are benefits associated with the design that cannot be quantified in monetary terms. For these benefits, the required data is missing to make assumptions as to what the monetary value of the benefit could be. However, this does not mean that these benefits do not exist. In this appendix, these identified benefits will be briefly discussed.

Increase of Welfare

During the floods, large parts of the land flood and stay flooded for a long time. This limits the accessibility of large parts of the project area, limiting the access to healthcare facilities. As a result, villagers are unable to travel to the local clinics or the nearby hospital. Furthermore, the clinics cannot be resupplied during the flooding, depleting the stocks of medical supplies in the area.

The floods also carry a lot of waste and flood the toilet pits. In the lower parts of the region, the water will come to a standstill, creating a breeding ground for diseases (Rieks- van Hal, 2015). This causes extra outbreaks of diseases in the region like cholera and malaria. The solution will limit the time and severity of the flooding in the project area, increasing the accessibility while decreasing the time the lands are flooded. This will provide a major increase to the welfare in the region.

Increased School Attendance

Currently due to the flooding children are unable to go to school due the roads being flooded. The low attendance during mainly the long rain season can have long-term effect on the level of education in the region (Rieks- van Hal, 2015). Controlling the floods will decrease the time the roads are unavailable which in turn will increase the attendance in the schools.

Increased Food Supplies

There are still months where due to the bad harvests people do not have enough food. Increasing the amount of time, the farmlands can safely be used will increase the prosperity and the availability of food in the region.

Increased Employment Opportunities

As stated by a local contractor only skilled labour would be brought in from outside of the project area for the construction works. Most of the unskilled labour would be hired locally, providing extra jobs and training for the local villagers. However, at this moment it is not possible to determine the effects of this due to the stage of the project.

Other opportunities will be provided during the lifetime of the structures, as labour is required to properly maintain and operate them. Moreover, the increased prosperity in the region will also provide for extra jobs, as more money will be available to buy and do things. This will benefit the local entrepreneurs and possibly allow for new services to be offered.

Long-term Commitments/Investments

Controlling the floods in the project area will likely increase the confidence of the local villagers. As a result, the willingness to invest or make longer-term commitments might increase. In the impact assessment, it was also mentioned that due to increased safety that ability for farmers to get a loan could increase. In the long-term, this will increase the potential economic growth of the project area.

Appendix C.4 – Unit Rates

The following assumptions have been made to determine the quantities that are needed for the cost estimation for the designs.

Earthworks

Samanga Dike

The Samanga dike needs to be constructed with soil from behind the dike. This volume of soil needed for the dike has been multiplied by a factor of 1.2 for compaction purposes, as determined in Appendix D.1 - Soils. The length of the dike was determined to be 4.45 kilometres. Therefore, the total volume for the earthworks is known.

Excavation New Kikuletwa South

The volume of soil that needs to be excavated has been determined by multiplying the cross-section of the river by the length of the river. The length was determined to be 3.1 km. The dike east of the New Kikuletwa South will be constructed with the soil from the excavation of the river. The volume of soil needed for the dike has been multiplied by a factor of 1.2 for compaction purposes.

Digging Spillways

The volume of soil that needs to be dug out has been determined by adding up the depth until which the foundation reaches multiplied by the area of the foundation. The coffers have been added also. The volume of soil that needs to be removed for the channels leading to the spillways is the distance from the river to the spillways multiplied by the area of the gate opening.

Building Pit Control structure

A building pit needs to be dug out for the control structure. The total volume of this has been determined by taking the total depth and area that the control structure will occupy and adding 1 meter on each side.

Structural Components

Foundation

A rock layer needs to be placed under the foundations of the spillways and the control structure. To determine the quantity, the area of the foundations have been taken. The rock layer will be 15 cm high.

Gravel

On top of the rock layer, a gravel layer needs to be placed. This layer extends from the bottom of the coffers until the bottom of the foundation. The gravel used is between the 5-20mm in diameter. Furthermore, different types of gravel will be used for scour protection for the control structures. Fine gravel (1.2-6mm), course gravel (10-50mm) and cobbles (25-175mm)

Concrete

The total volume of concrete has been determined by adding all the components of the spillways and the control structure.

Reinforcement

The reinforcement has been taken as 1% of volume of the concrete structures (Walraven & Fennis, 2013).

Mechanisms

The spillways and the control structure need mechanisms to lift the gates. The spillways need one each and the control structure two.

Formwork

The required formwork has been determined by calculating the surfaces of the structures, taking into account what elements will be casted at the same time.

Plastered sheets

The plastered sheets will be placed between the soil and the concrete foundation of the structures to prevent salt intrusion in the concrete. The reason for this is that the soil in the area is very saline.

Demolition

At this moment there are two spillways located in the current Samanga dike. These will need to be removed, as they do not meet the safety requirements. When left in place, they would form weak spots in the dike.

Unit Rates

For each of the quantities that have been described in the preceding section unit rates have been determined. The unit rates were obtained during several meetings with local contractors. The tender prices were reviewed during a meeting with a local engineer to see, based on his experience, if the prices are reasonable. All the unit rates are the tender prices the different contractors use and include transport and other cost. The used unit rates are presented in Table 126.

Table 126: Unit Rates

Clearing Vegetation	Unit	Price/unit	Details
Low density	m^2	TZS 6,000	No vegetation to grassland.
Medium density	m^2	TZS 6,000	Grassland mixed with bushes and banana trees.
High density	m^2	TZS 20,000	Dense vegetation including some trees.
# Trees	-	TZS 400,000	Larger size trees, located on grassland, counted individually.
Earthworks			
Excavation for construction pit	m^3	TZS 20,000	Small scale earthworks by general contractor
Excavation	m^3	TZS 9,000	Large scale earthworks, digging, transport within 200m, compaction.
Structures			
Foundation (15cm rock layer)	m^2	TZS 12,000	To provide an equal surface for the concrete foundation.
Concrete	m^3	TZS 350,000	Casting of concrete, excl. formworks and reinforcements.
Steel Reinforcement (1%)	kg	TZS 4,300	Includes bending, etc.
Formwork	m^2	TZS 25,000	The formwork required for the concrete casting.
Plastered sheets	m^2	TZS 4,500	Sheets placed between the soil and the concrete foundation to prevent saline intrusion.
Gate mechanic spillway	-	TZS 2,000,000	The steel gate including the mechanism to raise it.
Control Structure Mechanic	-	TZS 50,000,000	Cranes for lifting the concrete gates.
Gravel	m^3	TZS 107,000	1.2-175mm gravel
Demolishing current spillways	-	TZS 250,000	Complete removal of current spillways.

Appendix C.5 - Cost Determination Initial Design Phase

The assumptions and values that were used for the determining the cost for the different designs will be discussed in this appendix. The costs determined for the initial design phase are based on the information that was available at that time.

Realisation of the Project

These are the costs associated with the process from bringing the design in this report to full realization.

Finishing the Design

The design that will be presented in this report will be likely of preliminary design detail. This design will need to be further detailed, construction drawings and a bill of quantities need to be made. The percentages used were given in the meeting with a local contractor on the 11th of December. The range for these cost is between the 6-12% of the estimated construction cost. For a project of this size the lower bound of 6% was assumed to be more likely. In Table 127 the design costs are displayed as percentage of the construction cost.

Table 127: Design Cost

	% of construction cost
Bill of Quantity	1-2
Engineer	2-4
Architect	3-6
Total	6-12

Project Management

A project of this size will need to be managed, especially if different contractors will work on it at the same time. A first estimation of the cost has been made on the basis of the meeting with a local contractor on the 11th of December.

It is assumed one project leader (2.5 million TZS/month), two assistants (2.4 million TZS/month each) and two surveyors (1.44 million TZS/month each) are required to supervise the work. Furthermore, an additional 10% of the construction cost should be reserved for overhead expenses. These expenses cover the cost for the office, transportation, etc.

The preparation for the project would start 3 months beforehand and the work itself would take 5 months. The time estimations for the work were given by three contractors.

Construction Costs

Three designs were made in the initial design phase for three different return periods. Based on the determined dimensions and the unit rates (see appendix Unit Rates) the different construction costs were determined.

Contingency

During the construction of the work, there is always a chance that something goes wrong, for example: The bill of quantity is incorrect or there are flaws in the design. In other words, there are a lot of risks and money should be reserved for the costs associated with these risks. Therefore, a contingency budget of 30% of the construction cost is set.

Lifetime Costs

During the lifetime of the structures, there are also associated costs with it. The structures will need to be maintained, operated and repaired. The costs for the operation are not taken into account during the initial design phase, as it is hard to estimate them at this point and they are the same for all options. It will not matter for the decision between the different return periods if these costs are taken into account.

Maintenance

In meetings with contractors and other stakeholders the question was raised if they could make an estimation for the required maintenance work. Most of them could not do this. In the initial design phase maintenance is not considered to be a main topic. Therefore, it is not yet clear how the maintenance of the works will be arranged. This will be included in a later stage of the design, see 21 Implementation.

The assumption is made to use 0.5% of the construction cost for the annual maintenance costs. This percentage is based on the experience of Rijkswaterstaat in the Netherlands. It cannot be said that the situation in Tanzania is the same as in the Netherlands, however, it allows a first estimation to be made of the maintenance costs. More data is not available at the moment.

Repairs due to Exceedance

When the actual discharge exceeds the design discharge for which the structures have been designed, extra damage can occur that is not covered by maintenance. The repair cost is estimated at 10% of the construction costs of the dike, spillway and control structure. To get an annual budget, the estimated cost is multiplied with the expected return period of the design.

Appendix C.6 - Construction Costs of the Integral Design

Clearing

Both cluster 1 and cluster 2 need to be cleared of vegetation. The land has been divided into 3 categories; low density, medium density and high density. This has been done as each of these categories has a different price range for clearing. The division of land can be read in more detail in Appendix C.7 - Cost Estimation for Vegetation Clearing. Additionally, the amount of trees has been estimated in the areas.

Structures

By multiplying the quantities with the unit rates the total construction cost can be determined. The expected start date of the project has not been taken into account for these values, thus they are excluding the inflation. The used unit rates and quantities for cluster 1 are presented in Table 128 and for cluster 2 in Table 129.

Table 128: Construction Cost Cluster 1

Clearing vegetation	*Quantity*	*Unit*	*Price/unit*	*Total*
Low density	99,950	m^2	TZS 6,000	TZS 599,700,000
Medium density	35,100	m^2	TZS 6,000	TZS 210,600,000
High density	11,550	m^2	TZS 20,000	TZS 231,000,000
# Trees	25	-	TZS 400,000	TZS 10,000,000
Total				**TZS 1,051,300,000**
Samanga Dike	*Quantity*	*Unit*	*Price/unit*	*Total*
Earthworks new dike	74,226	m^3	TZS 9,000	TZS 668,034,000
Total				**TZS 668,034,000**
Spillways	*Quantity*	*Unit*	*Price/unit*	*Total*
Excavation Foundation	56	m^3	TZS 20,000	TZS 1,128,000
Excavation Channel	11	m^3	TZS 20,000	TZS 216,000
Foundation (Rock layer)	60	m^2	TZS 12,000	TZS 720,000
Concrete	31	m^3	TZS 350,000	TZS 10,990,000
Steel Reinforcement (1%)	2,516	kg	TZS 4,300	TZS 10,818,800
Formworks	115	m^2	TZS 25,000	TZS 2,862,500
Sand	32	m^3	TZS 45,000	TZS 1,435,500
Gravel	21	m^3	TZS 107,000	TZS 2,273,750
Plastered sheets	60	m^2	TZS 4,500	TZS 270,000
Gate mechanics	1	-	TZS 2,000,000	TZS 2,000,000
Total per spillway				**TZS 32,714,550**
Number of spillways to be built	5	-		TZS 163,572,750
Number of spillways to be demolished	2	-	TZS 250,000	TZS 500,000
Total all spillways				**TZS 164,072,750**
Total of Cluster 1				**TZS 1,883,406,750** **$ 862,600**

Table 129: Construction Cost Cluster 2

Clearing vegetation	*Quantity*	*Unit*	*Price/unit*	*Total*
Low density	49,480	m^2	TZS 6,000	TZS 296,880,000
Medium density	20,320	m^2	TZS 6,000	TZS 121,920,000
High density	35,750	m^2	TZS 20,000	TZS 715,000,000
# Trees	13	-	TZS 400,000	TZS 5,200,000
Total				**TZS 1,133,800,000**
Kikuletwa South	*Quantity*	*Unit*	*Price/unit*	*Total*
Earthworks	56,110	m^3	TZS 9,000	TZS 504,990,000
Total				**TZS 504,990,000**
Control structure	*Quantity*	*Unit*	*Price/unit*	*Total*
Building pit	2,688	m^3	TZS 20,000	TZS 53,760,000
Excavation entry channel	666	m^3	TZS 9,000	TZS 5,994,000
Foundation (Rock layer)	396	m^2	TZS 12,000	TZS 4,752,000
Concrete Structure	328	m^3	TZS 350,000	TZS 114,726,500
Concrete Gates	3	m^3	TZS 350,000	TZS 1,045,800
Steel Reinforcement (1%)	26,488	kg	TZS 4,300	TZS 113,898,400
Formworks	425	m^2	TZS 25,000	TZS 10,615,000
Plastered sheets	396	m^2	TZS 4,500	TZS 1,782,000
Fine Gravel 1.2-6mm	85	m^3	TZS 107,000	TZS 9,095,000
Cobbles 25-175mm	70	m^3	TZS 107,000	TZS 7,490,000
Course Gravel 10-50mm	29	m^3	TZS 107,000	TZS 3,103,000
Gravel under foundation 5-20mm	594	m^3	TZS 107,000	TZS 63,558,000
Gate mechanics	2	-	TZS 50,000,000	TZS 100,000,000
Total				**TZS 489,819,700**
Total Cluster 2				**TZS 2,128,609,700**
				$ 974,900

Appendix C.7 - Cost Estimation for Vegetation Clearing

In this appendix, an estimation is made for the clearing cost of the vegetation at the construction locations. The estimation of the intensity of the vegetation is based on satellite images from Google Earth taken at the 10th of September 2013.

The vegetation was divided into four classes:

- Low density; no vegetation to grass vegetation.
- Medium density; grassland mixed with bushes and banana trees.
- High density; dense vegetation including trees.
- Number of trees; trees located in grassland were counted individually and not as area.

There are two possibilities to cut down the clearing costs. The first possibility would be for the local villagers to do parts of the work, for example the removal of trees and clearing higher density areas. This will need to be discussed with the local communities around the time of construction to see what they are capable of doing. The second possibility would be to make use of TPC equipment for parts of the clearing. However, this equipment is likely not available for long time periods and cannot be counted on.

Thus, it is assumed that contractors will do all the clearing. The other two options will be potential cost savers when the time comes. The estimated unit rates for this work have been based on prices that were received during meetings with local contractors. During the meetings it was also stated that there is no significant different between grassland and banana trees.

Per cluster the total areas for the vegetation and the associated clearing cost will be given.

Cluster 1

The location of the dike and dimensions have been determined in 14 Cluster 1. In total, an area of 146,600 m^2 will need to be cleared of vegetation, which has been specified in Table 130: Clearing cost vegetation of Cluster 1with the associated clearing cost.

Table 130: Clearing cost vegetation of Cluster 1

Vegetation class	Amount	Unit	Unit Rate	Total	
Low density	99,950	m^2	TZS 6,000	TZS 599,700,000	$ 274,700
Medium density	35,100	m^2	TZS 6,000	TZS 210,600,000	$ 96,500
High density	11,550	m^2	TZS 20,000	TZS 231,000,000	$ 105,800
Number of trees	25	piece	TZS 400,000	TZS 10,000,000	$ 4,600
				TZS 1,051,300,000	**$ 481,600**

Cluster 2

The location and dimensions of the control structure and Kikuletwa South have been determined in 15 Cluster 2. In total, an area of 105,550 m^2 will need to be cleared of vegetation, which has been specified in Table 131: Clearing cost vegetation Cluster 2 with the associated clearing cost.

Table 131: Clearing cost vegetation Cluster 2

Vegetation class	Area	Unit	Unit Rate	Total	
Low density	49,480	m^2	TZS 6,000	TZS 296,880,000	$ 136,000
Medium density	20,320	m^2	TZS 6,000	TZS 121,920,000	$ 55,800
High density	35,750	m^2	TZS 20,000	TZS 715,000,000	$ 327,500
Number of trees	13	piece	TZS 400,000	TZS 5,200,000	$ 2,400
				TZS 1,133,800,000	**$ 521,700**

Appendix C.8 - Operation and Maintenance Costs

During the lifetime of the structures, there are various expenses that will be made for the operation and the maintenance of them. The assumptions made for estimating these costs will be explained in this chapter. These are different than the assumptions used during the initial design stage as more knowledge was gained in the meantime.

Regular Maintenance Cost

The maintenance cost consists of both the regular maintenance of the structures as well as the repairs that are required when damage occurs due to exceedance of the design discharge.

The regular maintenance cost is based on the estimation (0.5% of construction cost) Rijkswaterstaat in the Netherlands uses. Although the situation in Tanzania cannot directly be compared with the Netherlands, the estimation gives a good first impression of what the cost will be. Unfortunately, it was not possible to obtain local data for maintenance cost.

A cost and benefit analysis done by VITO (Vlaamse Instelling voor Technologisch Onderzoek) uses the same percentage as Rijkswaterstaat for maintenance of non-moveable structures (Nocker, Broekx, & Liekens, 2004) Thus, the percentage of 0.5% of the construction cost for the regular maintenance is used.

Repairs due to Exceedance

The return period that was taken as the basis for the design is once every fifteen years. This means that there is a chance that the design discharge will be exceeded. If this occurs, the higher water level can cause damage to the structures.

It is estimated that the damage will be 10% of the construction cost of the structures, as they will only fail locally and not entirely. Using the return period of once every fifteen years an annual budget of 0.7% of the construction cost should be reserved for this possibility.

Operation

The solution consists of several manually operable structures, which will need to be operated and overseen. Furthermore, for the maintenance of the structures regular inspections are required by someone who has an understanding of what is required. For this it is required that a team will be set up to take care of the operation and the inspections.

At least one paid supervisor (2,000,000 TZS/month) with sufficient knowledge about maintenance and how the system works should be hired. He will coordinate the operation and oversee the maintenance of the structures. It might not be required that he works fulltime on the project.

If the local villagers/farmers are willing, they could be trained to help with the inspection and operation of the system. When this is not the case, or it is proven unreliable, additional personnel should be hired to assist the supervisor.

Appendix C.9 - Sensitivity Analysis

The calculated costs and benefits are based on many variables. With a sensitivity analysis the relative important of the various variables will be determined and their effect on the project outcome. This will lead to the identification of the variable to which the project is most sensitive.
In order to determine this, first only one variable will be changed at the same moment. Secondly, different combinations of variables will be tested to observe their combined effect. The variables chosen are those that are uncertain or considered to have a large influence on the outcome of the project.

Varied Variables

For the various variables the switching values (the percentage/absolute number a variable need to change for the NPV to become zero) and sensitivity indicators (compares the percentage change in variable with the percentage change in the NPV). This will help identify the potential risks/threats to the project. (Verhaeghe, 2009)

Discount Rate

The discount rate is used to calculate the present day value of future costs and benefits. The discount rate can change depending on the wishes of the backers or other stakeholders.

Table 132: Sensitivity NPV for Discount Rate

Discount Rate	NPV	Compared to original situation
3%	$ 18,632,800	+ 133.82%
5%	$ 13,201,300	+ 65.66%
7%	$ 9,420,900	+ 18.22%
8% (original)	$ 7,968,900	± 0%
9%	$ 6,740,300	- 15.42%
11%	$ 4,803,600	- 39.72%
13%	$ 3,378,400	- 57.61%
22.82%	$ 0	- 100%

In Table 132, the effect of the discount rate on the NPV is presented. It can be observed that the NPV becomes zero only after a significant increase of the discount rate. In normal cases the NPV will stay positive.

Inflation Rate

The inflation rate is used for determining the value of the costs and benefits in the years to. Currently, based on historic date, a probable average for the inflation rate has been determined. However, due to worldwide events it can easily deviate from this average.

Table 133: Sensitivity NPV for Inflation

Inflation Rate	NPV	Compared to original situation
0%	$ 3,179,000	- 60.11%
2%	$ 4,866,200	- 38.94%
3%	$ 5,950,400	- 25,33%
4%	$ 7,237,300	- 9.18%
4.5% (original)	$ 7,968,900	± 0%
5%	$ 8,766,700	+ 10.01%
6%	$ 10,586,600	+ 32.85%
7%	$ 12,754,200	+ 60.05%

In Table 133, the effect of the inflation on the NPV is presented. It can be observed that there is no inflation rate that will cause the NPV to become zero. The only case where this would be possible is when during the lifetime of the project, there is a continuous deflation of minimally 10%, which is deemed unrealistic.

Start Date of the Construction

Due to delays with the design, tender or finding backers for the project, it is possible that the project will start a year later. If that is the case, the NPV will increase by 4.4%. The reason is that due to the inflation, the benefits will increase more than the costs as time progresses. Leading to a more positive outcome the longer is waited with construction.

However, this assumes that the funds will be received and the contractor fees determined at the start date of the construction. When the funds are received at an earlier date, the benefit will become less as the worth of the money decreases. For the contractors, it will have negative consequences when the project starts later while the contracts are already signed. This is caused by the increase in prices while their fee has been already determined. Unless in the contract a reimbursement for inflation is stated then this is not the case.

Construction Time

It is possible that due to delays during construction, the project will be finished a year later. If this is the case, the NPV will decrease with 5.82%, as the expenses increase while the benefits will not be received during that year.

Unit Rates

The unit Rates are obtained from a limited number of contractors, this means that with the actual tender the prices are likely to differ. Furthermore, the estimated costs for design, contingency and project management are based on a percentage of the unit rates. In Table 134, the variations for the unit rates and their effect on the NPV are presented.

Table 134: Sensitivity NPV for Unit Rates

Unit Rates Variation	NPV	Compared to original situation
- 15%	$ 8,454,300	+ 6.9%
- 10%	$ 8,292,500	+ 4.06%
- 5%	$ 8,130,800	+ 2.03%
± 0% (original)	$ 7,968,900	± 0%
+ 5%	$ 7,807,200	- 2.03%
+ 10%	$ 7,645,400	- 4.06%
+ 20%	$ 7,321,900	- 8.12%
+ 30%	$ 6,998,300	- 12.18%
+ 40%	$ 6,674,700	- 16.24%
+ 50%	$ 6,351,170	- 20.30%
+ 246%	$ 0	- 100%

The effect on the NPV for a change in the unit rates is limited. Only for a significant increase of the unit rates the NPV will become zero.

Operation and Maintenance

A first conservative estimation was made for the operation and maintenance cost for the design. In Table 135, the effects of variations in these costs on the NPV are presented. It can be concluded that changes in the operation and maintenance cost have only an insignificant influence on the NPV. This is caused by the relatively low value of them compared to the other cash flows.

Table 135: Sensitivity NPV for Operation and Maintenance

O&M	NPV	Compared to the original situation
- 15%	$ 8,042,000	+ 0.92%
- 10%	$ 8,017,600	+ 0.61%
- 5%	$ 7,993,300	+ 0.31%
± 0% (original)	$ 7,968,900	± 0%
+ 5%	$ 7,944,600	- 0.31%
+ 10%	$ 7,920,300	- 0.61%
+ 15%	$ 7,896,000	- 0.92%

Yield per Acre (Sale Value)

The sell price of the agricultural goods changes over the years. The development of the price is partly taken into account by the inflation. However, the price of agricultural goods is also dependent on the supply and demand. The results of this mechanism on the NPV are presented in Table 136.

Table 136: Sensitivity NPV for Yield per Acre

Yield per acre	NPV	Compared to the original situation
- 71%	$ 0	- 100%
- 10%	$ 6,837,100	- 14.20%
- 5%	$ 7,403,000	- 7.10%
- 2%	$ 7,742,600	- 2.84%
- 1%	$ 7,855,800	- 1.42%
± 0% (original)	$ 7,968,900	± 0%
+ 1%	$ 8,082,200	+ 1.42%
+ 2%	$ 8,195,400	+ 2.84%
+ 5%	$ 8,534,900	+ 7.10%
+ 10%	$ 9,100,900	+ 14.20%

A change in the value of the yield will result in a stronger change in the NPV. When the value of the yield drops by 71% the NPV becomes 0.

Years Until the Farmland is Fully Utilised

In the estimations for the benefits a set period of time was taken after which the farmers would fully use their lands during the short (10 years) and long rain seasons (23 years). These timespans have been varied to determine the effect of the NPV. In Table 137 the effects for the short rain and in Table 138 for the long rain harvest season are presented.

Table 137: Sensitivity NPV for Short Rain Harvest Season

Years till full utilisation of farmland	NPV	Percentage change compared to original
8	$ 8,073,000	+ 1.31%
9	$ 8,022,100	+ 0.67%
10 (original)	$ 7,968,900	± 0%
11	$ 7,936,800	- 0.4%
12	$ 7,902,000	- 0.84%
25	$ 7,473,800	- 6.21%

Table 138: Sensitivity NPV for Long Rain Harvest Season

Years till full utilisation of farmland	NPV	Percentage change compared to original
16	$ 9,480,200	+ 18.96%
20	$ 8,559,300	+ 7.41%
22	$ 8,156,500	+ 2.35%
23 (original)	$ 7,968,900	± 0%
24	$ 7,827,400	- 1.78%
26 (72% of land used in year 25)	$ 7,502,442	- 5.85%
30 (53% of land used in year 25)	$ 7,077,600	- 11.19%

The impact of the extra land use during the short rain is less than that of the long rain. This is due to the fact that land is already being used during the short rain season, while not so much during the long rain season.

Different Scenarios

Two difference combinations of variables for the benefits will be used to see how the project performs if the prediction for the future were too optimistic. There are many other possible combination of variables, but these were considered most relevant.

No Increased Farmland Utilisation

A major part of the potential benefits for the project is the extra land that will be used once the structures have been built. This does not need to be the case. Extra harvests will require more water and nutrients which might not be available enough. Moreover, the farmers might not be willing to use their land if the effort increases to have a reasonable harvest.

It is assumed that there will be no extra farmland used during the short and the long rain season. This will result in a decrease of the NPV by 50,6%, resulting in a NPV of $ 3,971,700. This is still a good positive result.

High Supply of Agricultural Goods

There is a high supply of agricultural goods which results in lower prices for agricultural goods (-20%). Moreover, due to the lower prices the farmers are less inclined to use their land during the short and the long rain season. It will take 15 years before all the land during the short rain season and 25 years before 50% is used during the long rains. Moreover, the inflation is lower than estimated (3.5%).

This will result in a decrease of the NPV by 33.56%, resulting in a NPV of $ 5,292,400. Despite the more pessimistic predictions the project still has a good positive result.

Conclusion of the Sensitivity Analysis

For several variables the sensitivity of the NPV has been tested. It was found that the discount rate and the inflation have a major influence on the NPV. While the prices for the unit rates and operation and maintenance have only a marginal influence.

The time till all the land during the long rain season is used proved to have the largest impact for the benefits. Furthermore, the value of the yield per acre has a strong correlation with the benefits and thus the NPV. The time it takes before all the land is used during the short rain has only a small influence on the result.

Two different scenarios were tested to determine their impact on the result. In case that no extra land will be used during the rain seasons, the project will still have a positive result. In case that the supply of agricultural goods is high, resulting in lower value per acre, less land being used and a lower inflation, the project will also have a positive result.

Appendix D – Integral Design

Appendix D.1 - Soils

This appendix will present the results of the soil testing that was performed by TanRoads. Firstly, the results are shown and hereafter the associated soil properties are presented. These soil properties are used in Part D – Integral Design.

Soil Testing Results

With the results of the soil provided by TanRoads the soil logs from the fieldwork are adapted so that our assumptions can be checked. The Liquid Limit (LL) and the Plasticity Index (PI) are used in order to classify the soil type. The plasticity chart in Figure 116 is used to classify the soil.

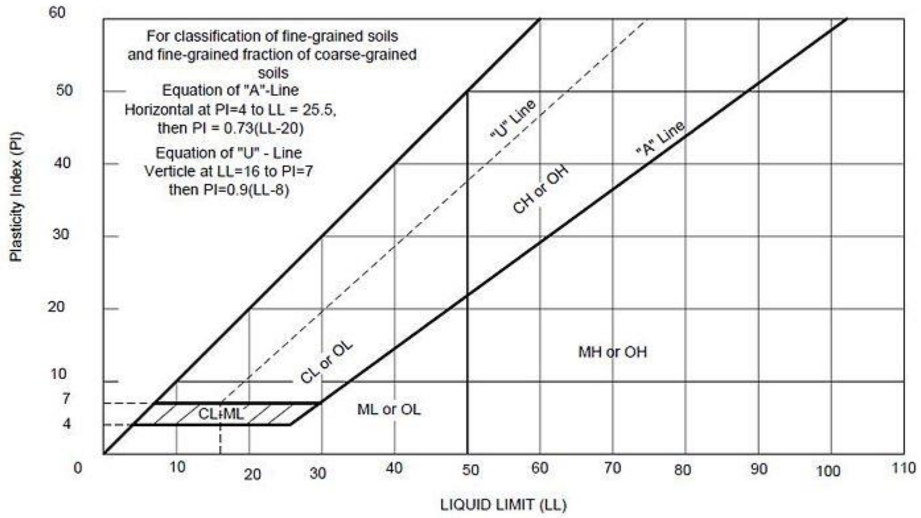


Figure 116: Plasticity chart (Guide, 2012)

Table 139: Soil testing results

Soil Sample	Location	Moisture Content (%)	PI (%)	LL (%)	Soil type	Sand
1BgI	Samanga dike middle (top of dike)	22.0	11	40.7	ML	32% fine sand
1BgII	Samanga dike middle (top of dike)	12.7	8	35.9	ML	6% fine sand
1CgI	Samanga dike middle (bottom of dike)	11.3	11	39.1	M	17 % fine sand
1DgI	Samanga dike end (top of dike)	10.4	15	40.7	ML	15 % fine sand
1DgII	Samanga dike end (bottom of dike)	48.8	16	36.3	CL	9 % fine sand
2BgII	Bifurcation (west side Kikuletwa South)	50.8	8	37.1	ML	39 % fine sand
2Ag 2.4	Bifurcation (east side Kikuletwa South)	48.4	9	38.6	ML	20 % fine sand
3AgI	Control structure Chem Chem	23.0	11	39.9	ML	2 % fine sand
3AgII	Control structure Chem Chem	41.6	15	43.1	ML	7% fine sand 6% course sand 6% fine gravel
1Hg	Samanga dike foundation	33.9	9	31.5	ML	34 % fine sand
SEDIMENT 1	Control structure	28.8	5	31.0	ML	72% fine sand

Soil Properties

The properties of the soil will be treated in this part. They are based on the soil test results treated in the preceding part of this chapter. The soil testing results revealed that the soil is for the most part classified as ML; this is inorganic silt.

Particle Size Sediment Sample

The sediment sample is the only sample from which the particle size has to be determined. This is needed for both the scour protection at the control structure and morphological calculations. The grain size distribution can be determined from the sieve test results. The d_{15} is 0.075 mm and the d_{85} is 0.275. Using the assumption that the d_{50} is the average of the d_{15} and the d_{85}, the d_{50} is determined to be 0.275 mm.

Unit Weight

Dry Unit Weight

The dry unit weight can be determined using the graph in Figure 117.

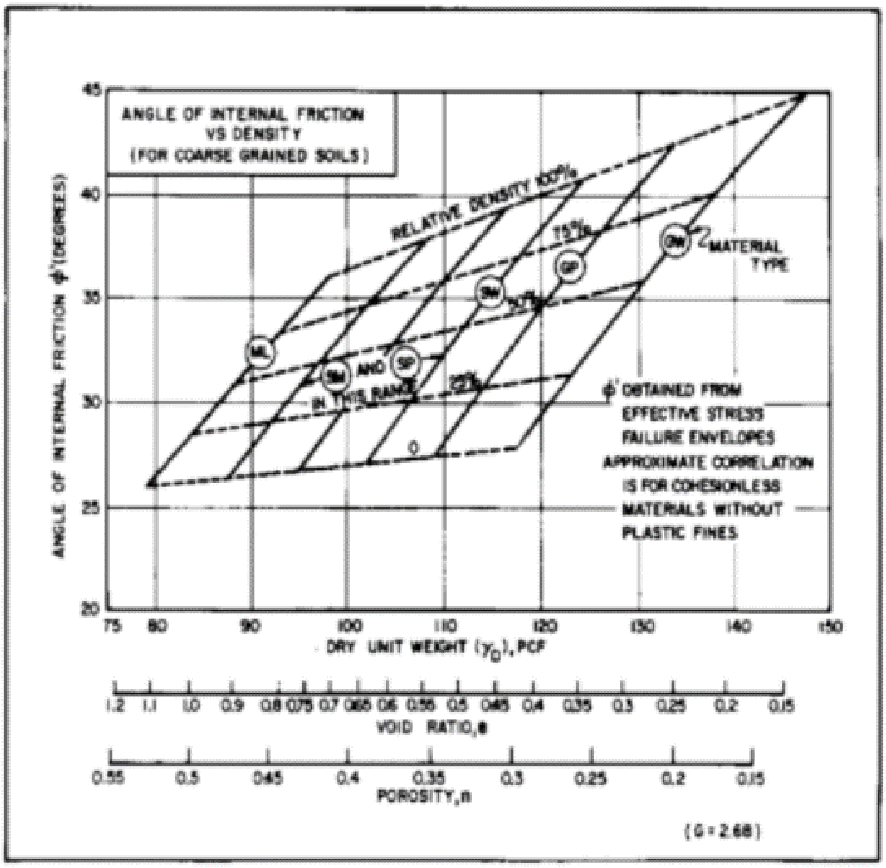


Figure 117: Strength Characteristics (Command, 1986)

The dry unit weight read from the graph for ML that is medium densely packed is around 95 pounds per cubic foot (pcf). Using the conversion of 1pcf = 0.15709 kN/m^3 the dry unit weight is: 14.9 kN/m^3. A rounded up value of 15 kN/m^3 will be used.

Wet Unit Weight

The moisture content for each sample varies. Average moisture content per location has been determined in Table 140: Wet unit weight. By using equation 38 below the wet unit weight can be determined. The resulting wet unit weight should be taken as representative above the phreatic surface.

$$\gamma_{wet} = \gamma_{water}(1-n)(m+G_s) \tag{38}$$

Where - γ_{wet} is the wet unit weight [kN/m^3]

- γ_{water} is the unit weight of water [kN/m^3]
- m is the moisture content [%]
- G_s is the specific gravity [-]
- n is the porosity [-]

γ_{water} is 10 kN/ m^3, m can be found in the soil results, G_s is 2.68 (Compaction) and e , n can be found in Figure 117 and is 0.43.

Table 140: Wet unit weight

Location	Moisture Content [%]	Dry unit weight [kN/m³]	Wet unit weight [kN/m³]
Samanga dike	14.1	15	16.08
Samanga dike foundation	41.35	15	17.63
Bifurcation Kikuletwa South	49.6	15	18.10
Control structure Chem Chem	32.3	15	17.12
Sediment	28.8	15	16.92

Saturated Unit Weight

The saturated unit weight is found using equation 39. This is the unit weight below the phreatic surface. (Compaction)

$$\gamma_{saturated} = \frac{\gamma_{water}(G_s + e)}{(1 + e)} \tag{39}$$

Where: - $\gamma_{saturated}$ is the saturated unit weight [kN/m^3]

- γ_{water} is the unit weight of water [kN/m^3]
- G_s is the specific gravity [-]
- e is the void ratio [-] (Compaction)

γ_{water} is 10 kN/ m^3, G_s is 2.68 (Compaction) and e is determined using the graph in Figure 117 and is equal to 0.75.
The resulting value for saturated soil is: $\gamma_{saturated}$ = 19.6 kN/m^3
An average value of 20 kN/m^3 will be used.

Cohesion

The cohesion has been determined by using the lowest value found with the pocket vane shear tests during field-testing. This value is set at 15 kPa. This can be verified by the near horizontal riverbanks observed in the area, which indicate that there must be cohesion.

Friction angle

The friction angle has also been determined with the graph in Figure 117. The value is 33 degrees.

Loading Conditions

These loading conditions have been determined with the help of geotechnical engineer Keith Ward.

Long-term

Long-term conditions (in which the dike is able to drain), the friction angle should be taken as the dominant factor and the cohesion can be set to zero. The long rain floods last longer and therefore the long-term approach can be taken.

Short-term

For short-term conditions (in which the dike is unable to drain quickly enough), the cohesion is the dominant factor and the friction angle can be set to zero. Since the short rain floods are flashy, the short-term conditions are applied

Post Flood

For post flood conditions the same parameters as for short rains apply; undrained conditions.

Dry Season

During the dry season there will not be any fast loading and therefore the same loading conditions as for the long rain floods apply.

Table 141: Loading scenarios

Loading scenario	Condition	Friction [°]	Cohesion [kPa]
Short rain	Undrained	0	15
Long rain	Drained	33	0
Post flood	Undrained	0	15
Dry season	Drained	33	0

Compaction

The existing dike has moisture content around 15% and its consistency was relatively hard, meaning it had been compacted. The new dike will be compacted in a similar way. The soil tested from around the area has a moisture content of around 40%. Since the porosity is 0.43, this means that the percentage of soil particles is 57%. Moisture content is expressed as a percentage of the dry soil weight, as can be seen in equation 40 (Compaction).

$$m = \frac{W_w}{W_s} \tag{40}$$

Where: - W_w is the weight of the water [kg]
- W_s is the dry weight of the soil [kg]

Therefore, if the moisture content is 15% this translates into 9% of the total weight in this case. Similarly, 40% moisture content translates to a 23% moisture percentage as a whole. The ratios can be found in Figure 118 and Figure 119.

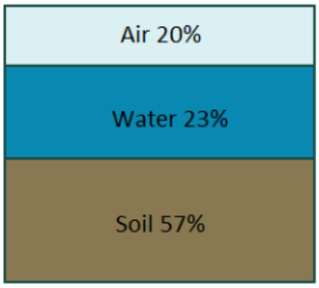


Figure 118: Soil not compacted, moisture content 40%

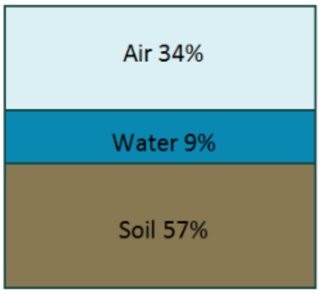


Figure 119: Soil Compacted, moisture content 15%

Therefore, is the soil is compacted from 40% moisture content to 15% moisture content the ratio with which the volume of soil and water will decrease is 0.83. This is assumed to be equal to the volume decrease. A compaction factor of 1.2 is taken as representative. This means that the soil that is necessary for the dike should be multiplied by 1.2 for arrive at the compacted volume.

Appendix D.2 - Elaboration on the Guiding Dikes

On each side of the spillway 3 meters is kept free to prevent heavy erosion on the toe of the guiding dikes. The reasoning behind this is that the flow will have more time to slow down after leaving the spillway, therefore less erosion is expected. The dikes will be 1 meter high and a crest width of 1 meter is used. Assuming a slope of 45 degrees is possible the total width of a guiding dike will be 3 meters, see Figure 120 for a cross-section of the dike. The length of the dikes will be 20 meters, covering the entire gap behind the Samanga dike. Failure mechanisms for this dike will not be included in this report, because the dikes do not influence the flood safety directly. Furthermore, the dike's dimensions are assumed on the safe side and can be adjusted relatively easy. Determining the guiding dike dimensions more accurately can be done in a later stage of the project.

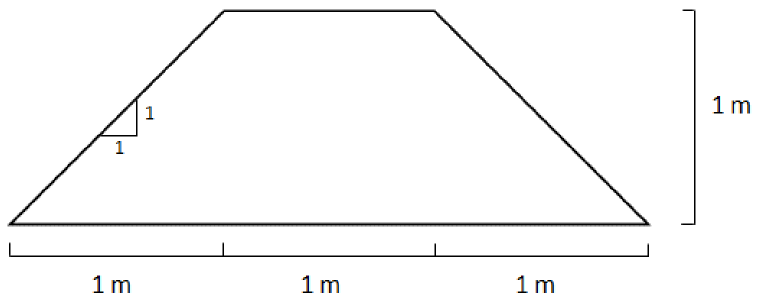


Figure 120: Cross-section guiding dike

To quickly calculate the volume of soil needed for the guiding dikes the length of the dike is multiplied by the cross-sectional area. This results in 40 m^3 soil per dike, so 80 m^3 soil per spillway. Assuming a compaction coefficient of 1.2 this results in 1.2 x 80 = 96 m^3. The soil needed for the guiding dikes can be taken from the location indicated in the plan view, see Figure 121. Taking the soil from this location will favour the flow in the right direction. The area needed for this excavation is 96 divided by 0.75 meter (depth of the channel), resulting in 128 m^2. The length of the excavation is 128 divided by 12.5 meter (the total width between the guiding dikes), resulting in approximately 10.5 meter.

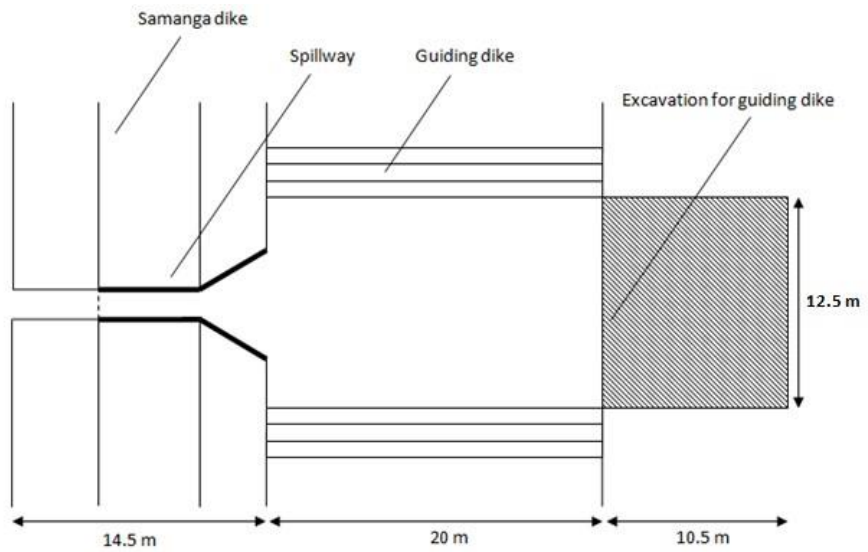


Figure 121: Plan view guiding dikes

Appendix D.3 - Scour Protection

This appendix will mainly focus on the bed protection needed at the spillways and the control structure. Besides the bed scour, some information about protection against other scour types will be provided. However, these will not be worked out in detail, for the final design it is advised to look into this. First, the concept of bed scour will be explained and the necessity of protection against scour emphasized. Then a short summary of different types of bed protection that are possible to make with local materials will be provided. Finally, the scour protection dimensions for both the spillways and the control structure will be calculated. Assumptions made will be elaborated and the formulas used will be provided.

Necessity of Bed Protection

Bed scour can be described as the removal of native soil by the flow of the water, also referred to as erosion. This can occur before and after hydraulic structures. The removal of the soil will form a scour hole, which can decrease the stability of the structure and this can even lead to failure. A number of different mechanisms can lead to bed scour. The sudden change from hard material (concrete) to the native soil can cause additional turbulence in the flow. Additionally, the flow speed can be locally higher at the entrance or exit of a structure. Furthermore, there is less sediment passing the structures causing a sediment imbalance, this occurs only at the exit of the structure.

As mentioned before the scour mechanisms create a scour hole. If the bed is not protected the scour hole can have large negative effects on the functionality and stability of the structure. In Figure 122 an example is given of scour holes that can occur next to a hydraulic structure.

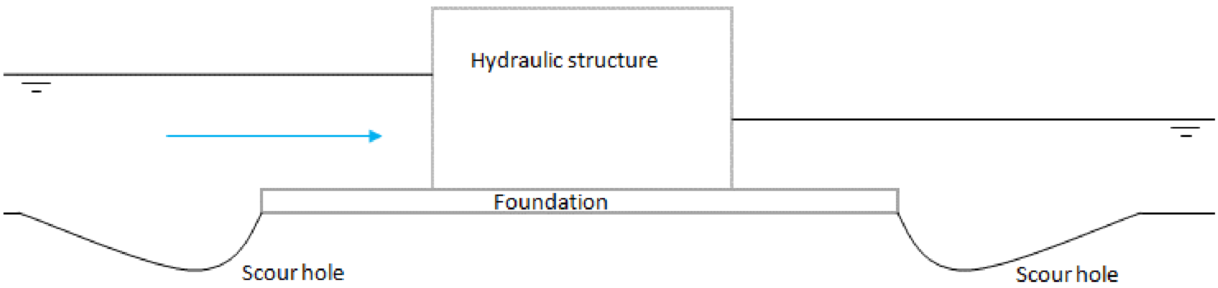


Figure 122: Scour holes at a hydraulic structure

It is not always necessary to design against the erosion caused by the bed scour. In some cases, the scour that occurs will not have large negative effects. Therefore, there should be good argumentation for the designed protective measures.

Types of Bed Protection

There are a large number of different protections possible, not all will be listed here. The most important distinction that can be made is between soft protection and hard protection.

Soft protection implies that there will be no hard protective layer to prevent the erosion. The erosion problem will be solved using a different approach. This is done by supplying new soil to compensate for the removed soil. Soft protection treats the erosion rather than preventing it. This can be achieved in different ways, but for this project only replenishment of removed soil will be considered. For the construction works presented in this project, this is the most practical form of soft protection and least complicated.

Hard protection implies that there will be a protective layer that prevents erosion from occurring at the protected location. Hard protection usually moves the erosion problem to a location where it is no longer threatening for the structure that needs to be protected. There are many hard protections possible, using a large number of different materials. For this project, only protections of loose rock/gravel will be considered, because this is the easiest and cheapest material to acquire locally. Furthermore, the equipment and expertise to construct the other kinds of protections might not be available. Rock/gravel protections often consist of multiple layers with different stone sizes and gradations.

Scour Protection at the Spillways

The scour protection at the spillways will be elaborated here. The scour protection at the front of a spillway (entrance) and the scour protection at the back (exit) will be treated separately. At each side, the necessity of protection will be discussed and the different options for protection are considered. First, the scour protection at the front will be looked at. The choice of protection type for the front will be a soft protection. Then the scour protection at the back will be explained, here the choice is made for a hard bed protection.

Scour Protection at the Front

First, the necessity of protective measures is considered and then the type of protection best suited will be presented. The scour protection at the front of the spillway will be based on observations done during fieldwork. This is possible because there are two spillways present that are similar to the spillways designed for this project.

Necessity of Protective Measures

During the field trips, observations were made concerning the erosion around the existing spillways. It was clearly visible that after a flood the bed and the dike around the spillway had eroded a few centimetres. However, the erosion did not cause direct problems for the functionality and stability of the structure. Considering a larger amount of time, protection against the erosion is necessary because after a number of floods the erosion can cause problems for the flood safety. The flood safety will decrease mainly because of the erosion at the dike. The erosion observed during the fieldwork can be seen in Figure 123.

It is clearly visible that the front of the dike eroded; there was no soil replenishment since the construction of the spillway. This was a year before the picture was taken. The dike should be protected with soft or hard protection.

Figure 123: Erosion at the excising spillway, observed during fieldwork 1

Type of Protection

In this case, soft protection is the best solution to the erosion. Entailing that the removed soil will be replenished manually after a certain time. This can be done relatively easily by the local population without the use of heavy machinery. The following arguments can be given for the choice of soft rather than hard protection:

- During a flood, it is very hard to dimension for both the flow into the spillway and the flow from the river along the dike.
- With hard protection, the erosion due to scour will be moved to a place where the protection stops. This will only move the problem and in this specific case, there is no good location to move the problem to.
- There is a high chance that protection material will wash away with the river current during a flood. It is not likely that protection material that is washed away can be recovered, resulting in very high reparation costs.

Soft protection is possible in this case because after a few floods, there is no direct effect on the flood safety or stability of the flood defence works. Only after a longer period, problems can occur, but there is enough time to execute the soft protection works, by replenishing eroded soil. The only disadvantage for soft protection is that it is labour intensive and the spillways should be regularly monitored after floods.

Based on the fieldwork observations the soil that needs to be replenished after a flood can be estimated. The estimation for an average flood is 2 m^3 per spillway. This is only an indication because every flood is different in intensity. Further research must be done should the volume of soil be determined exactly.

Scour Protection at the Back

Like with the protection at the front, first the necessity of protective measures is considered and then the type of protection best suited will be presented. In this case, the choice is made for a hard protection. The scour protection at the back of the spillway will be dimensioned using reasonable estimations and field measurements. First, the length of the protection is determined, then the grain size, followed by the filter calculations and finally the layer thickness is given.

Necessity of Protective Measures

On this side of the spillway, protective measures are advised because an erosion hole just after the spillway can cause a number of problems. Firstly, the leakage length decreases resulting in a higher chance of piping. Secondly, the erosion can continue under the structure risking instability of the structure. Finally, a big hole after the spillway will hinder the flow towards the Samanga area. The flooding will be less smooth and will be delayed.

Type of Protection

A soft protection will not solve all the problems mentioned above. Because most of the problems occur during the flooding and it is not possible to supply soil manually during the flooding. Therefore, a hard protection is preferable. The scour hole that will develop after the hard protection (moving of the problem) can be filled up after the flooding. This is the case assuming that, this scour hole occurs at all because the flow speed will be much lower further from the spillway. Exact calculations for this scour hole can be done in a later stage of the project, if it is necessary.

Length Protection

The length of the protection will be calculated using equation 41 (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015). This equation is used for first estimations of the protection length and is for this stage of the project sufficient. A schematisation of the bed protection and the maximum scour depth is given in Figure 124.

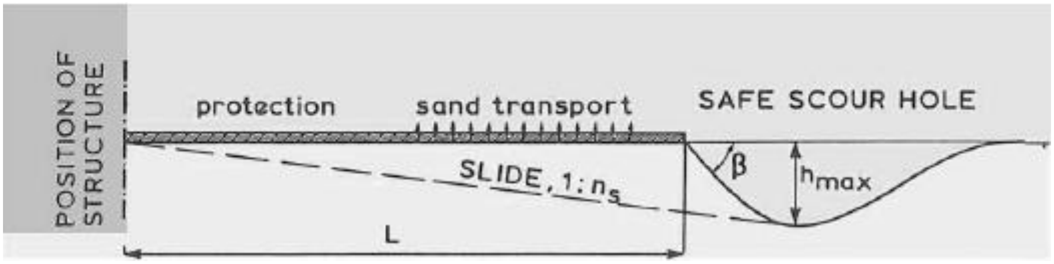


Figure 124: Bed protection against scour (Vrijling, Bezuyen, Kuijper, & Molenaar, 2015)

$$L \geq \gamma * n_s * h_{max} \tag{41}$$

Where: - L is the protection length [m]
- γ is a safety factor [-]
- $1{:}\,n_s$ is the average slope of the slide [-]
- h_{max} is the maximum scour depth [m]

The safety factor is set at 1.4, lower is not advised because there are still a lot of uncertainties and higher is not necessary because the project is in a very early stage. The slope n_s is set at 8 corresponding to a densely packed cohesive material. The maximum scour depth can be calculated using equation 42, assuming that there is no sediment coming from upstream.

$$h_{max} = h_0 * \frac{(0.5 * \alpha * u) - u_c}{u_c} \tag{42}$$

Where: - h_0 is the initial water depth [m]
- α is a coefficient to include turbulent effects [-]
- u is the depth averaged flow velocity at the end of the bed protection [m/s]
- u_c is the critical velocity of the soil particles [m/s]

The initial water depth is estimated at 0.75 meter. For the coefficient to include turbulent effects, a value of 3 can be taken. The depth averaged velocity is estimated on 0.35 m/s. This is 4 times lower than the highest possible velocity in the spillway channel, see Appendix B.2 - Design Calculations Spillways. The flow velocity here is taken much lower because the flow width increases from 1.8 meter to 12.5 meter. The critical velocity of soil particles can be found using the Shields equation, see equation 43.

$$u_c = C\sqrt{\Psi_c * \Delta * D_{n50}} \tag{43}$$

Where: - C is the Chézy coefficient [$m^{1/2}/s$]
- Ψ_c is the Shields stability parameter [-]
- Δ is the relative density [-]
- $D_{n50}$is the median nominal diameter of the soil particles [m]

The Chézy coefficient can be calculated using a depth of 0.75 meter, a width of 12.5 meter and a roughness equal to 2*D_{n50}.The Shield stability parameter is estimated on 0.1, this is because there are very fine silt and sand particles present in the soil. The relative density is 0.8, assuming silt ground with a unit weight of 1800 kg/m^3 and water of 1000 kg/m^3. The D_{n50} is determined from the soil tests and is determined to be 0.05 mm.

The results of the calculations done are listed in Table 142.

Table 142: Results length calculation

Equation	To be calculated	Result	Unit
43	u_c	0.1766	m/s
42	h_{max}	1.48	m
41	L	17	m

Therefore, the length of the bed protection should be at least 17 meters long. Now that the length of the protection is known a plan view of the scour protection can be made, see Figure 125.

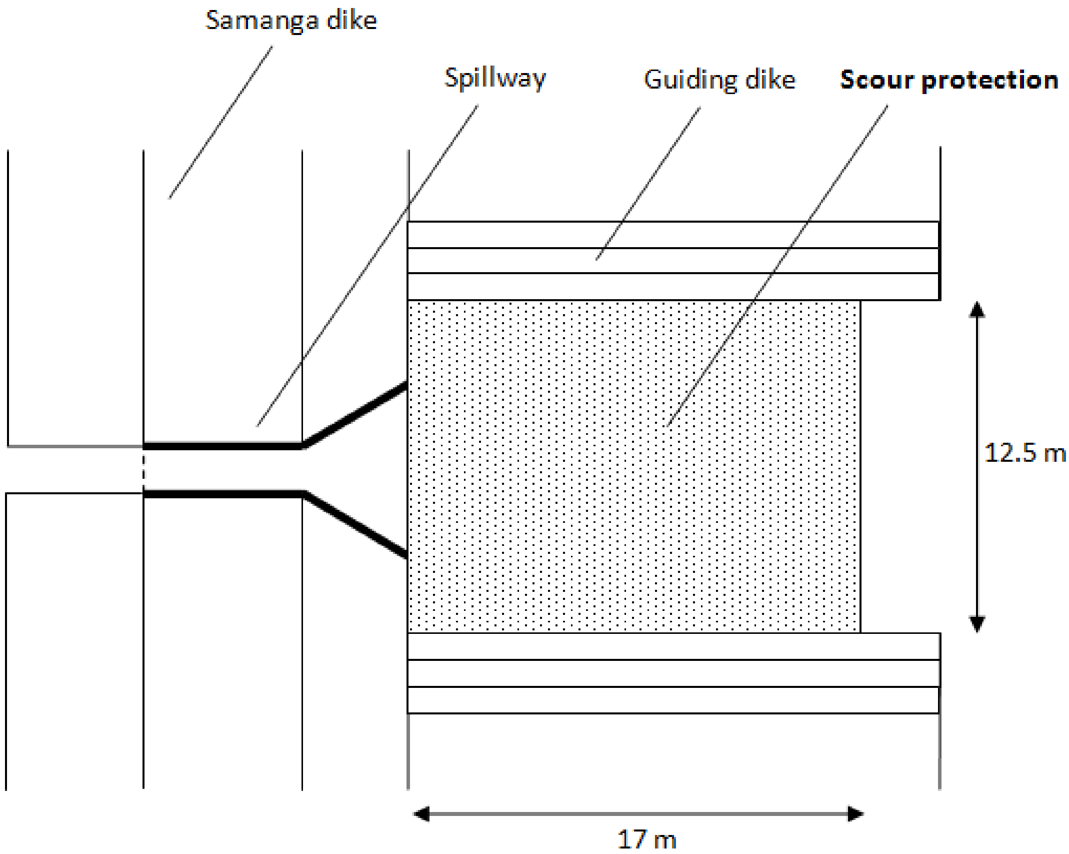


Figure 125: Plan view scour protection

Grain Size

The grain size for the scour protection can be calculated using Shields equation (Schiereck & Verhagen, 2012). Assumed is a high flow velocity of 1.4 m/s, equal to the flow velocity in the small channel in the spillway. This is a conservative approach because the exact flow velocity at the scour protection is unknown. The depth is assumed to be 0.75 meter, for the Shields parameter a standard value of 0.03 is chosen. For the relative density 1.65 is assumed, this depends on the type of stone used for the construction, which is not yet known. No factor for turbulence is included because the spillway ends in a streamlined shape. The Chézy coefficient can be determined iteratively using equation 44. This is an iterative calculation because the Chézy coefficient depends on the grain size.

$$C = 18 * \log(12 * \frac{R}{k_r}) \tag{44}$$

Where: - R is the hydraulic radius [m]

- k_r is the equivalent sand roughness [m]

The hydraulic radius can be determined using the depth and the width; the depth is assumed at 0.75 meters and the width is 12.5 meters. The equivalent sand roughness can be approximated by $2{*}D_{n50}$. After four iterations, the median nominal diameter of the soil particles needed is determined on 25 mm.

Filter Design

The native soil particles are much smaller than the median grain size of the scour protection. Therefore, multiple layers of different gradations are necessary; this is called a granular filter. The granular filter will prevent erosion of the lower layers. There are two types of granular filters, the geometrically open and the geometrically closed filter. Each filter type has one main advantage and one main disadvantage, see Table 143.

Table 143: Advantage and disadvantage granular filters

Filter type	Main advantage	Main disadvantage
Geometrically open	Hydraulic loads are taken into account leading to a more economical design.	Detailed information on the loading gradients needed (not available).
Geometrically closed	Conservative design ensuring that no erosion occurs.	Conservative design leading to over dimensioning and therefore higher costs.

It is not possible to determine the exact hydraulic loads in this stage of the project. Therefore, a more conservative design is the better option. Especially taking the risk of an under dimensioned geometrically closed filter into account. Repairs to a damaged filter will cost more than the over dimensioning of the geometrically closed filter. Taking the advantages and disadvantages into account the geometrically closed filter is selected. The design of the filter will be treated here.

Geometrically Closed Filter

There are three relations for a geometrically closed filter: stability between the filter layer and base layer, permeability and internal stability (Schiereck & Verhagen, 2012). See equation 45, 46 and 47.

$$\text{Stability: } \frac{d_{15F}}{d_{85B}} < 5 \tag{45}$$

$$\text{Internal stability: } \frac{d_{60F}}{d_{10F}} < 10 \tag{46}$$

$$\text{Permeability: } \frac{d_{15F}}{d_{15B}} > 5 \tag{47}$$

Where: - d_{15} is the sieve diameter which is passed by 15% of the mass of the grains
- d_{85} is the sieve diameter which is passed by 85% of the mass of the grains

This is the same for d_{60} and d_{10} with 60% and respective 10%. Subscript F indicates that this value concerns the filter layer, while subscript B indicates the base layer. The soil data of the first base layer, the native soil, can be found in Table 144. The d_{85} was determined in the soil tests, the d_{15} is assumed to be $\frac{1}{2}d_{85}$. The d_{50} is assumed to be equal to $\frac{1}{2}(d_{85} + d_{15})$.

Table 144: Sieve diameters of the first base layer (native soil)

Sieve diameter	Value [mm]
d_{15}	0.0375
d_{50}	0.05
d_{85}	0.075

Using the stability and permeability relations for geometrically closed filters, this results in a maximum d_{15} for the filter layer of 0.375 mm and a minimum d_{15} of 0.1875 mm. By setting the d_{15} at 0.2 mm the other sieve diameters for the filter layer can be determined, see Table 145. For internal stability, a $\frac{d_{60}}{d_{10}}$ ratio of 10 is equal to a $\frac{d_{85}}{d_{15}}$ ratio of 12. The ratio can be taken lower but not higher.

Table 145: Sieve diameters of the first filter layer (second base layer)

Sieve diameter	Value [mm]
d_{15}	0.2
d_{50}	1.3
d_{85}	2.4

The d_{50} is lower than the needed diameter determined in the grain size calculation, which was 25 mm. Therefore, another layer is necessary, now the first filter layer is regarded as the base layer. Repeating the process, the properties of second filter layer can be determined, see Table 146. The maximum d_{15} for the second filter layer is 12 mm and the minimum d_{15} is 1mm. The $\frac{d_{85}}{d_{15}}$ ratio is set at 5, this gives a lower variation in grain size and therefore it might me easier to acquire.

Table 146: Sieve diameters of the second filter layer (top layer)

Sieve diameter	Value [mm]
d_{15}	10
d_{50}	30
d_{85}	50

The d_{50} of this layer is bigger than the diameter determined in the grain size calculation. Therefore, there is no need for a third layer.

Layer Thickness

The top layer consists of medium to coarse gravel (Verruijt, 2007). A rule of thumb is a layer thickness of $2d_{50}$. This results in a thickness of 2 x 30 = 60 mm. This is a minimum thickness, for more safety a layer of 100 mm is advised, this is equivalent to 0.1 meters.

The first filter layer consists of medium to very coarse sand (Verruijt, 2007). Using the same rule of thumb as before the layer thickness will be, 2 x 1.3 = 2.6 mm, which is almost impossible to realise. Therefore, a layer thickness of a few decimetres should be sufficient. In this specific case, 1.5 dm should be sufficient; this is equivalent to 0.15 meter. The cross-section of the total scour protection can be seen in Figure 126.

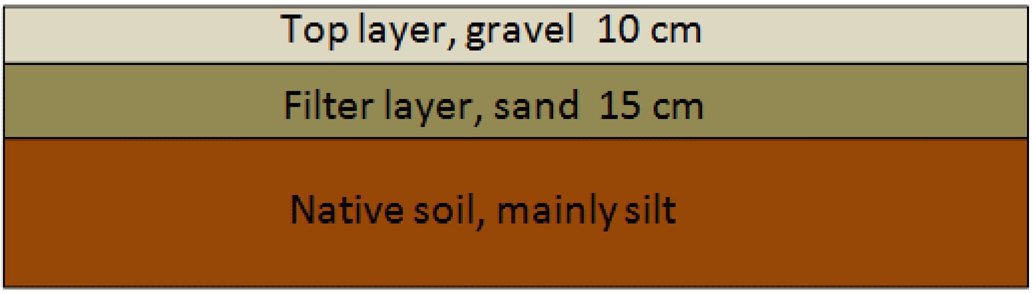


Figure 126: Cross-section bed protection back side of the spillway

Scour Protection at the Control Structure

The scour protection at the control structure will be elaborated here. The bed protection will be the main point of focus here. The riverbank protection will be mentioned but no calculations will be done. For the final design the bank protection should be included. The scour protection at the front of the control structure (entrance) and the scour protection at the back (exit) will be treated separately. At each side, the necessity of protection will be discussed and the different options for protection are considered. First, the scour protection at the front will be looked at. The choice of protection type for the front will be a hard protection, consisting of a loose rock with a geometrically closed granular filter. Then the scour protection at the back will be explained, here the choice is made for a hard bed protection. Which is similar to the front protection, only smaller gravel is used because the normative flow speed on this side is expected to be lower.

Scour Protection at the Front

Like with the protection for the spillways, first the necessity of protective measures is considered and then the type of protection best suited will be presented. In this case, the choice is made for a hard protection. The hard scour protection at the front of the control structure will be dimensioned using reasonable estimations and field measurements. First, the length of the protection is determined, then the grain size, followed by the filter calculations and finally the layer thickness is given.

Necessity of Protective Measures

Bed protection at the front of the control structure is essential to keep the scour hole that will develop on a safe distance from the structure. If the scour hole gets too close to the structure a number of problems can occur. Firstly, the leakage length decreases resulting in a higher chance of piping. Secondly, the erosion can continue under the structure risking instability of the structure. Finally, a big hole for the control structure can influence the flow towards the structure. Decreasing the discharge that can flow towards the Kikuletwa South, especially during low water levels.

It is very likely that protective measures at the riverbanks are needed at the interface of the river and the control structure. These are not further elaborated in this project and should be included in a later stage. The necessity and dimensions of the riverbank protections depend on the exact location of the control structure, the flow speeds that will occur and available material.

Type of Protection

A soft protection by replenishing eroded soil is ruled out in this case, because the soil would be placed in an active river and this is very hard to realise. Furthermore, the occurrence of a scour hole close to the control structure, even for a short time can give stability problems. Therefore, a hard protection is preferable.

Length Protection

The length of the protection is calculated using the same approach as for the back side of the spillways. This formula assumes that there is no sediment coming from upstream, which is not the case for this design. However, this will only result in a conservative protection length. A conservative length for the bed protection is in this stage of the project not a problem. Later more detailed models can be made to optimise the design. In Table 147 the values used for the calculation are displayed, together with the resulting protection length. For the maximum scour depth 2.8 meter is assumed, equal to the maximum occurring water level, see chapter 8 Design Discharges. For n_s a value of 6 is chosen, corresponding to a cohesive soil. A safety factor of 1.1 is taken. A low safety factor is chosen because the calculation is already conservative.

Table 147: Bed protection length for the front of the control structure

Quantity	Symbol	Value
Safety factor [-]	γ	1.1
Average slope of the slide [-]	$1{:}\,n_s$	6
Maximum scour depth [m]	h_{max}	2.8
Length bed protection [m]	L	**18.5**

Grain Size

The grain size needed for the protection is calculated using the Shields equation. This calculation is very similar to the grain size calculation for the spillways. This time a factor for turbulence is included resulting in equation 48. For the iteration of the Chézy coefficient, the turbulence factor should not be taken into account.

$$D_{n50} = \frac{u_c^2 * K_v^2}{\Delta * \Psi_c * C^2} \tag{48}$$

Where: - K_v is the velocity turbulence factor [-]

In Table 148 the values used for the calculation are given, together with the resulting median nominal diameter of the soil particles needed for the top layer of the bed protection. The design flow speed is assumed to be 2 m/s, equal to the maximum flow speed through the control structure. The turbulence factor is set on 1.2, this is just an estimation because the exact turbulence cannot yet be determined. For the depth a value of 1 meter is taken. This is not the depth that occurs at the given flow speed, but this gives a more conservative result. It is a combination between two normative scenarios, the highest flow speed and lowest water level. If more time was available, the scenarios could be worked out in more detail. For now, this will give a good indication.

Table 148: Determination of the grain size

Quantity	Symbol	Value
Design velocity [m/s]	u_c	2.0
Turbulence factor [-]	K_v	1.2
Relative density [-]	Δ	1.65
Shields stability parameter	Ψ_c	0.03
Chézy roughness after three iterations [$m^{1/2}$/s]	C	35.1
Diameter without turbulence [mm]	D_{n50}	65.59
Median nominal diameter [mm]	D_{n50}	**94.44**

Geometrically Closed Filter

The layer calculations are very similar to the spillway calculations. The first base layer (native soil) is assumed to consist of deposited sediment or similar material. In Table 149 the results of the soil tests on the sediment sample are displayed.

Table 149: Sieve diameters of the native soil

Sieve diameter	Value [mm]
d_{15}	0.075
d_{50}	0.275
d_{85}	0.4

In the Table 150 the results for the filter layer are displayed and in Table 151 the results for the top layer. For the filter layer a $\frac{d_{85}}{d_{15}}$ ratio of 5 is taken and for the top layer a ratio of 7.

Table 150: Sieve diameters of the filter layer

Sieve diameter	Value [mm]
d_{15}	1.2
d_{50}	3.6
d_{85}	6

Table 151: Sieve diameters of the top layer

Sieve diameter	Value [mm]
d_{15}	25
d_{50}	100
d_{85}	175

Only two layers are enough for this bed protection. The median nominal diameter of the top layer is bigger than the needed median nominal diameter.

Layer Thickness

The top layer consists of coarse gravel mixed with stones. A rule of thumb is a layer thickness of $2d_{50}$. This results in a thickness of 2 x 100 = 200 mm. This is a minimum thickness, for more safety a layer of 250 mm is advised, equivalent to 0.25 meters.

The filter layer consists of fine gravel. Using the same rule of thumb as before the layer thickness will be, 2 x 3.6 = 7.2 mm, which is almost impossible to realise. Therefore, a layer thickness of a few decimetres should be sufficient. In this specific case, 1.5 dm should be sufficient; equivalent to 0.15 meter. The cross-section of the total scour protection can be seen in Figure 127.

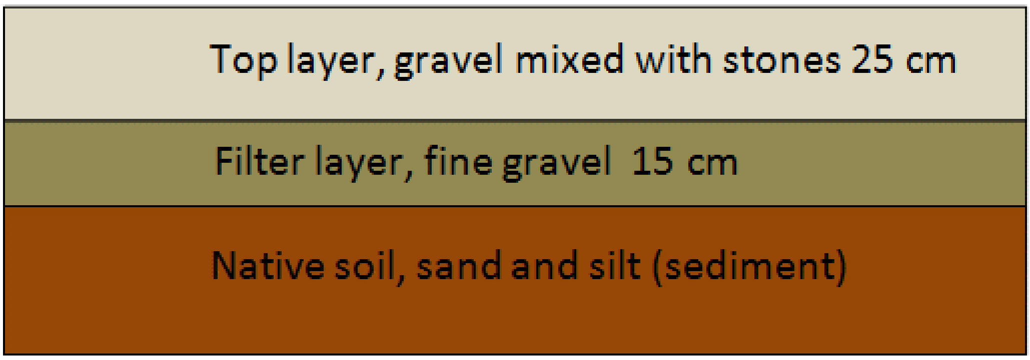


Figure 127: Cross-section scour protection at the front of the control structure

Scour Protection at the Back

Like with the protection at the front, first the necessity of protective measures is considered and then the type of protection best suited will be presented. The calculations and assumptions are very similar to the previous calculation. Therefore, this bed protection will be described very briefly. For more detail the reader is referred to the bed protection calculations for the spillways and the bed protection calculations for the front of the control structure.

Necessity of Protective Measures

The necessity of the bed protection at the back of the control structure has the same reasons as the protection at the front. Bed protection at the back of the control structure is essential to keep the scour hole that will develop on a safe distance from the structure. If the scour hole gets too close to the structure a number of problems can occur. Firstly, the leakage length decreases resulting in a higher chance of piping. Secondly, the erosion can continue under the structure risking instability of the structure. Finally, a big scour hole after the control structure can influence the flow towards the Kikuletwa South.

Type of Protection

A soft protection by replenishing eroded soil is possible, if the eroded soil can be replaced during the dry season, when the water levels in the Kikuletwa South are low. It is uncertain if this can be realised. Moreover, some of the problems mentioned before occur during high water discharges and soft protection cannot prevent these problems. Therefore, a hard protection is preferable.

Length Protection

The length of the protection is calculated in the same way as for the front of the control structure. In Table 152 the values used for the calculation are displayed, together with the resulting protection length. For the maximum scour depth, a depth of 2.9 meter is assumed and this is equal to the maximum occurring water level, see chapter 15 Cluster 2.

Table 152: Bed protection length for the back of the control structure

Quantity	Symbol	Value
Safety factor [-]	γ	1.1
Average slope of the slide [-]	$1{:}n_s$	6
Maximum scour depth [m]	h_{max}	2.9
Length bed protection [m]	L	**19**

Grain Size

The grain size needed for the protection is calculated using the Shields equation, including turbulence. In Table 153 the values used for the calculation are given, together with the resulting median nominal diameter of the soil particles needed for the top layer of the bed protection. The design flow speed is set on 1.38 m/s. This flow speed is determined using the expected flow speed during the long rains in the Kikuletwa South, equal to 1.1 m/s (see Appendix D.4 - Design Calculations Kikuletwa South) times a safety factor of 1.25. The turbulence factor is set on 1.2, this is just an estimation because the exact turbulence cannot be determined yet. For the depth a value of 1 meter is taken.

Table 153: Determination of the grain size

Quantity	Symbol	Value
Design velocity [m/s]	u_c	1.38
Turbulence factor [-]	K_v	1.2
Relative density [-]	Δ	1.65
Shields stability parameter	Ψ_c	0.03
Chézy roughness after three iterations [$m^{1/2}/s$]	C	44
Diameter without turbulence [mm]	D_{n50}	19.73
Median nominal diameter [mm]	D_{n50}	**28.4**

Geometrically Closed Filter

The native soil is assumed to be the same as the native soil at the front of the control structure. Therefore, the filter layer is also the same. In the Table 154 the results for the top layer are displayed. The top layer is different because a lower median nominal diameter is required. For the top layer a $\frac{d_{85}}{d_{15}}$ ratio of 5 is taken.

Table 154: Sieve diameters of the top layer

Sieve diameter	Value [mm]
d_{15}	10
d_{50}	30
d_{85}	50

Only two layers are enough for this bed protection. The median nominal diameter of the top layer is bigger than the needed median nominal diameter.

Layer Thickness

The top layer consists of coarse gravel. With a layer thickness of 0.1 meter.

The filter layer consists of fine gravel. With a layer thickness of 0.15 meter. The cross-section of the total scour protection can be seen in Figure 128.

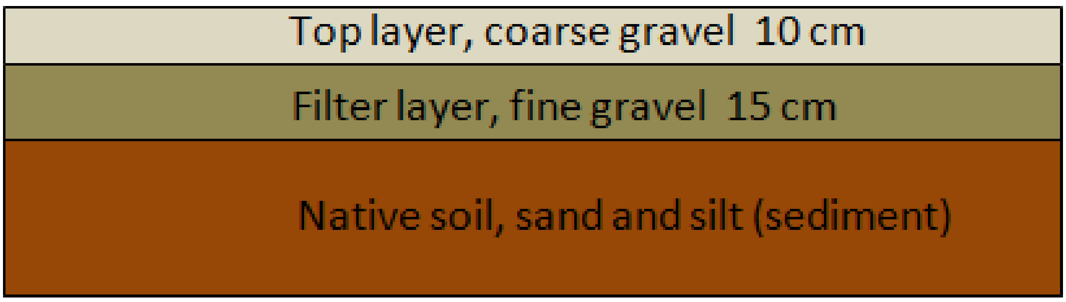


Figure 128: Cross-section of the scour protection at the back of the control structure

Appendix D.4 - Design Calculations Kikuletwa South

In this appendix the calculations for the integral design of the New Kikuletwa South are explained.

Cross-section

The integral design of the New Kikuletwa South consists of three sections, the deep section, main section and flood section. The cross-section of the New Kikuletwa South is given in Figure 129.

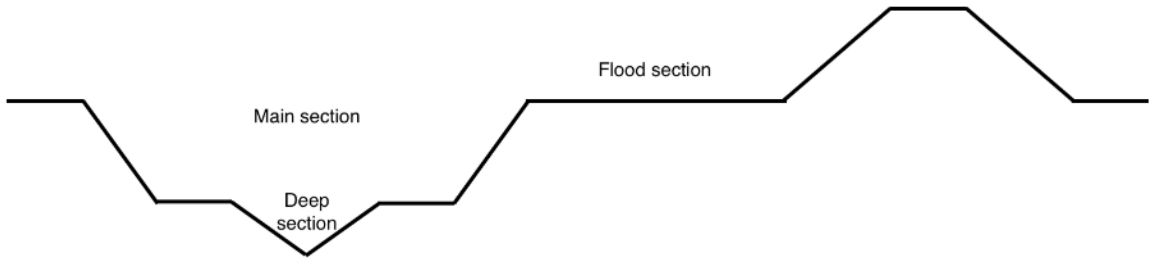


Figure 129: Sections New Kikuletwa South

Manning's Roughness Coefficient

The roughness coefficient for the flood section is determined using Figure 130. It is assumed that the floodplain has scattered brush and heavy weeds. According to this description the value for the roughness coefficient is set on 0.05 for the flood section. The roughness coefficient for the main section and the deep section together was also set on 0.05. Therefore, the total roughness coefficient of the river is 0.05.

3. Floodplains	Minimum	Normal	Maximum
a. Pasture, no brush			
1.short grass	0.025	0.030	0.035
2. high grass	0.030	0.035	0.050
b. Cultivated areas			
1. no crop	0.020	0.030	0.040
2. mature row crops	0.025	0.035	0.045
3. mature field crops	0.030	0.040	0.050
c. Brush			
1. scattered brush, heavy weeds	0.035	0.050	0.070
2. light brush and trees, in winter	0.035	0.050	0.060
3. light brush and trees, in summer	0.040	0.060	0.080
4. medium to dense brush, in winter	0.045	0.070	0.110
5. medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. dense willows, summer, straight	0.110	0.150	0.200
2. cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. same as 4. with flood stage reaching branches	0.100	0.120	0.160

Figure 130: Manning's n for floodplains (Chow, 1959)

Calculations

The dimensions of the New Kikuletwa South are iteratively determined using Manning's equation. For the Manning equation see Appendix B.3 - Design Calculations Kikuletwa South. The hydraulic radius and the cross-sectional area depend both on the width and height of the river. The bed slope only depends on the height of the river.

Just like in the initial design the deep section is designed to discharge 1 m^3/s. Only this time the height is set at 0.8 meter with a corresponding width of 4 meter. This was done to flatten the slope of the deep section, which increases the stability. The main section is designed for the short rains instead of the long rains. This change is made to decrease the excavation. The slope of the riversides is again set on 1:2 (height:width). In the integral design no extra safety height has been added to the maximum water height. Therefore, during the short rains the maximum water level reaches the top of the main section. In Table 155 the dimensions of the deep and main sections are shown.

Table 155: Dimensions deep and main section

	Deep section	Main section
h [m]	0.8	1.5
b (bottom) [m]	0	8
b (top) [m]	4	14
A [m^2]	1.6	16.5

In addition to the initial design a dike is built on the east side of the river. This is done in order to prevent the area of flooding during the long rains. The dike is built 5 meters from the river to prevent the dike from failing due to erosion of the river. The slope of the dike is 1:3.5 (height:width), just like the Samanga dike. The top width is 2.5 meter, which is the minimum width such that a compactor can drive over it. In order to determine the water level during the long rains it is assumed that the design is symmetrical; the same dike is assumed to be on the west side of the river. Using this assumption, the maximum water level during the long rains is 0.6 meter above surface level and therefore the height of the dike is chosen to be 1 meter. The dimensions of the dike and the flood section are given in Table 156.

Table 156: Dimensions flood section and dike

	Flood section	Dike
h [m]	0.6	1.0
b (bottom) [m]	24	9.5
b (top) [m]	28	2.5
A [m^2]	14.8	6.0

In Table 157 the bank full discharge for each section is determined using Manning's equation.

Table 157: Bank full discharge of each section

	Deep section	Main section	Flood section
i_b [-]	0.0026	0.0026	0.0026
n [$s/m^{1/3}$]	0.05	0.05	0.05
P [m]	4.3	15.0	29.2
A [m^2]	1.6	18.1	32.9
R [m]	0.37	1.2	1.13
U [m/s]	0.65	1.2	1.1
Q [m^3/s]	1	21	36

An impression of the maximum water levels during the dry, short rain and long rain seasons can be seen in Figure 131, Figure 132 and Figure 133.

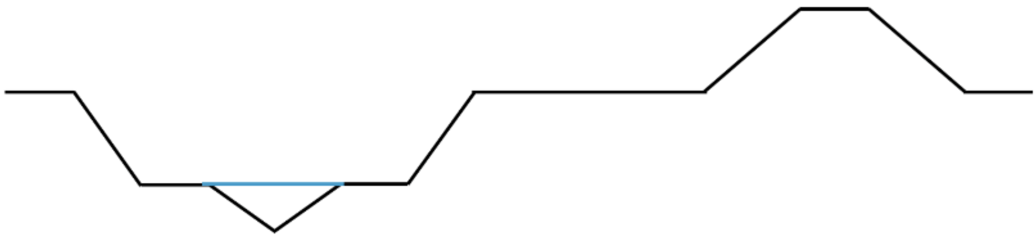


Figure 131: Maximum water level dry season

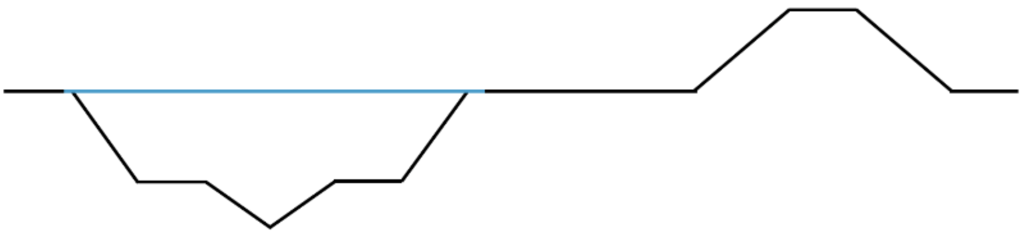


Figure 132: Maximum water level short rain season

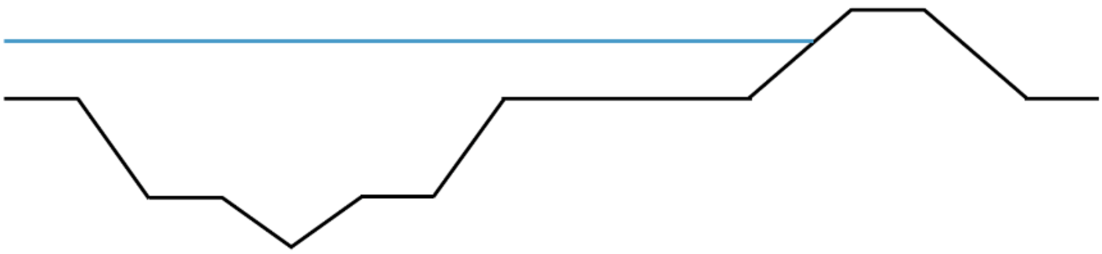


Figure 133: Maximum water level long rain season

Stability

In Figure 134, Figure 135, Figure 136 and Figure 137 the flooding scenarios of the New Kikuletwa South are shown.

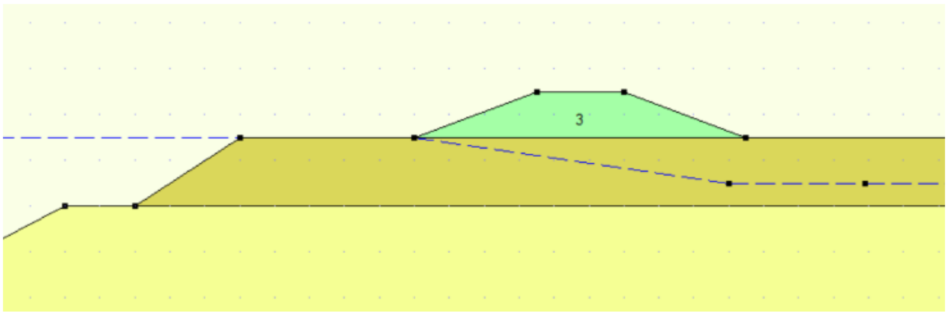


Figure 134: Short rain flood

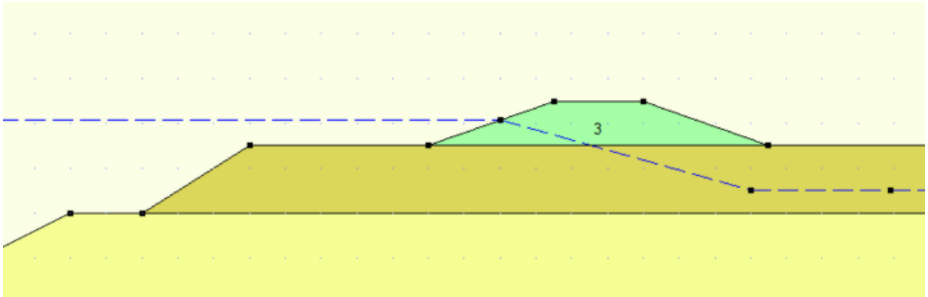


Figure 135: Long rain flood

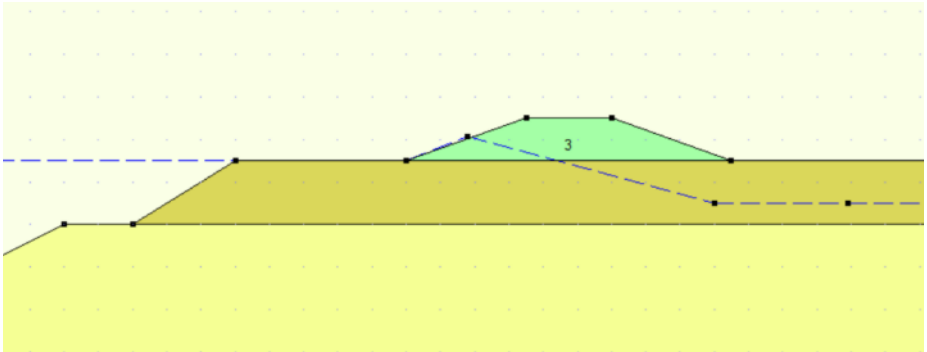


Figure 136: Post flood

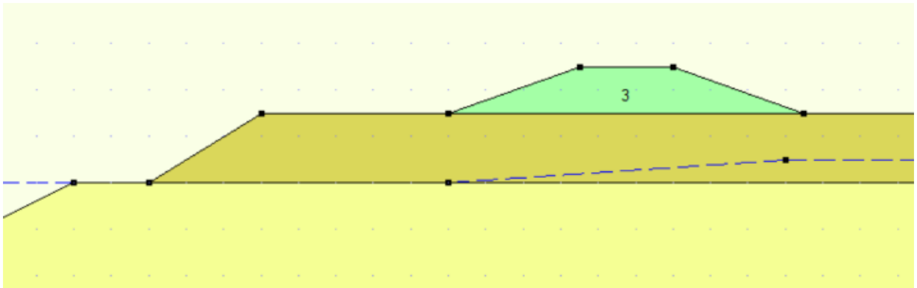


Figure 137: Dry season

Appendix D.5 - Reference Level

The reference level of the project is determined using the cross-sections of the rivers from the prefeasibility study. It is used to determine the connection between the control structure and the New Kikuletwa South.

Current Situation

In Figure 138 and Figure 139 the cross-sections of the Ronga and the Kikuletwa South Small at the bifurcation are shown. The black horizontal line represents the water level. It is assumed that the cross-section of the Ronga at the bifurcation is the same as the cross-section of the Kikuletwa North at the bifurcation. Using this assumption, the connection of the Kikuletwa North and Kikuletwa South Small is shown in Figure 140. The bottom of the Kikuletwa North at the bifurcation is chosen to be the reference level of the project.

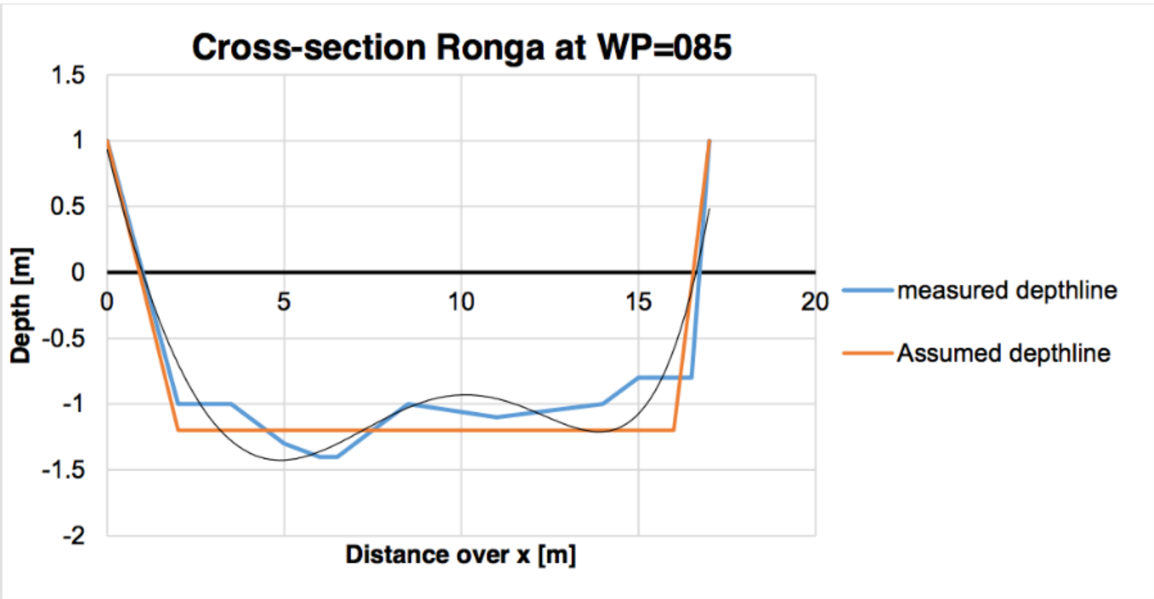


Figure 138: Cross-section Ronga at bifurcation (Lower Moshi (2015))

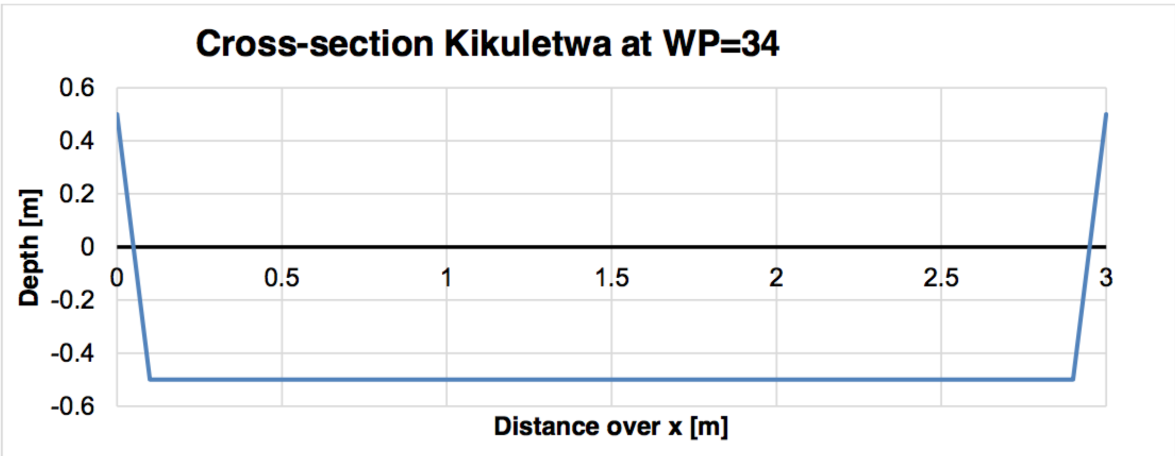


Figure 139: Cross-section Kikuletwa South Small at bifurcation (Lower Moshi (2015))

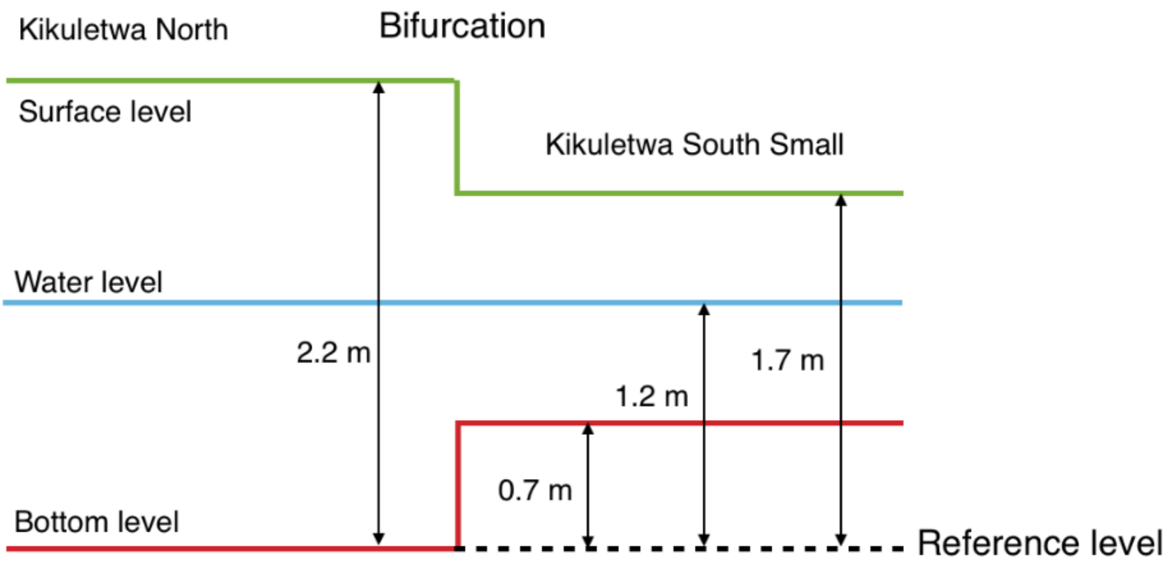


Figure 140: Connection bifurcation current situation

Excavation New Kikuletwa South

The dimensions of the New Kikuletwa South are taken from chapter 15 Cluster 2 and are shown in Figure 141.

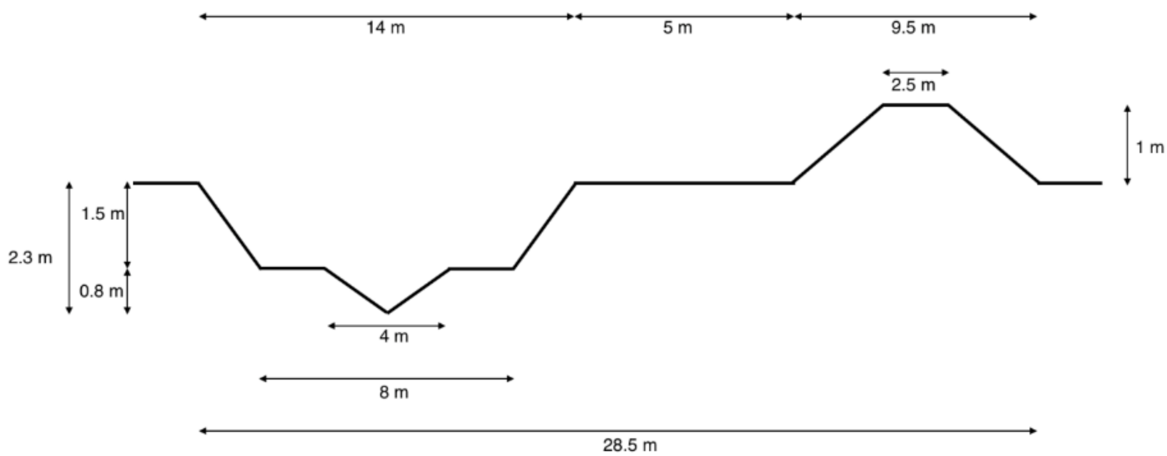


Figure 141: Dimensions New Kikuletwa South

The bottom level of the New Kikuletwa South is 2.3 meter below surface level and the top of the dike is 1 meter above surface level. Taking this into account the bottom of the New Kikuletwa South is 0.6 meter below reference level, see Figure 142. The green dotted lines are the surface levels of the dikes.

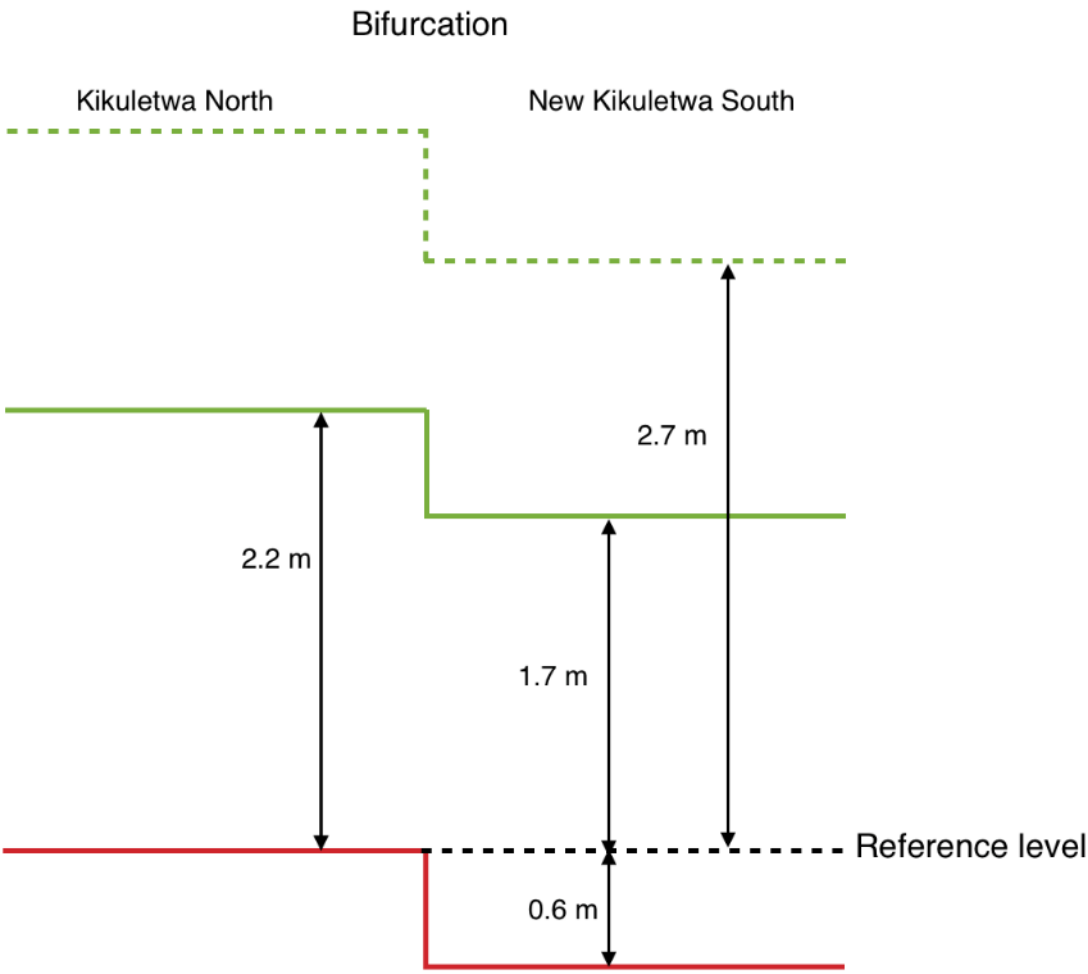


Figure 142: Connection at bifurcation after changes

Control Structure

In order to be able to determine the dimensions of the control structure and to ensure super-critical flow is not happening the maximum water levels at both sides have to be known. The maximum depths for both rivers are given in Table 158 and are taken from chapter 8 Design Discharges and chapter 15 Cluster 2 Integral Design.

Table 158: Maximum water depth on both sides of control structure

	Kikuletwa North	New Kikuletwa South
Depth [m]	2.8	2.9

The difference in water level is shown in Figure 143.

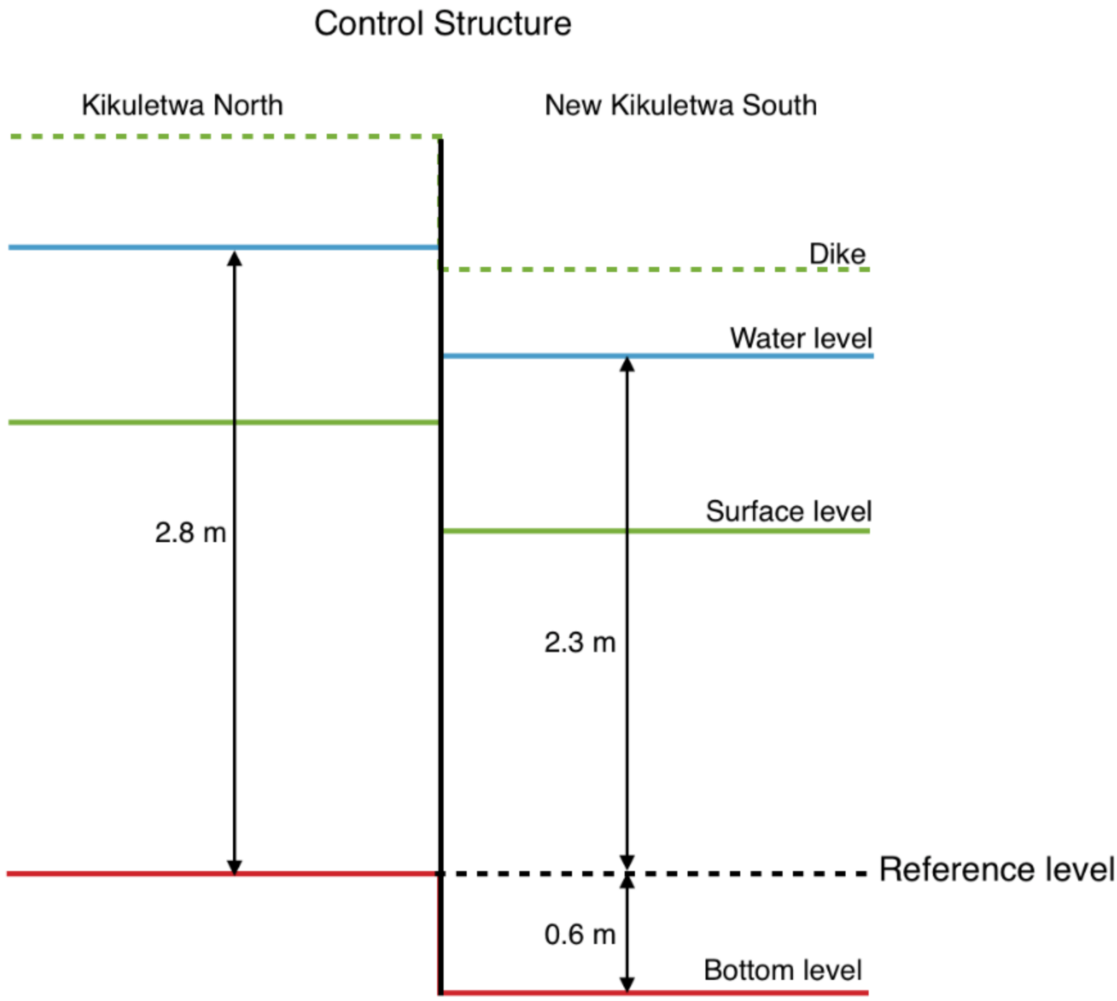


Figure 143: Maximum water level control structure

Appendix D.6 - Location Control Structure

The location of the control structure has been assumed to be located just after the bifurcation in the Kikuletwa South. However, this location needs to be more precisely determined in order to properly calculate the design and the associated costs. The different requirements for the location shall be given first, after which the chosen location shall be explained.

Requirements

First of all, there needs to be sufficient space available for the control structure itself and the additional space required during the construction phase. The total length of the structure, including scour protection is 55.5m with a width of the foundation is 22 meters, as determined in section 11.3.3 Design. Furthermore, for the construction pit minimally 5 meters on all sides need to be available. In the meetings with the contractors a lower requirement was given for this space, however, it is a good safety margin as the maps used to determine the location are not accurate.

During the fieldwork is was observed that logs and other debris are transported by the river. Ideally, the control structure would be located in such a way, that the majority of the debris would follow the Ronga, rather than enter or hit/damage the control structure. This would be the case when the control structure be placed perpendicular to the Ronga.

Moreover, when determining the location of the control structure the planned route of the Kikuletwa South should also be taken into account. When the structure is orientated in such a way that the connection channel between the control structure and Kikuletwa South has to make large turns, it would be suboptimal.

Additionally, all construction should take place within 60 meters from the sides of the river as this is the area Pangani Basin Water Board (PBWB) controls. If the solution is placed outside this area, it would make things more difficult.

Choice of the Location

Four alternative locations have been roughly draw on the map in order to determine the location that best meets the requirements. In Figure 144 these locations have been indicated, along with the border of the PBWB area and the current route of the Kikuletwa South Small. The different options for the location are argued below.

Option 1; placing the control structure perpendicular on the Kikuletwa North is not possible as it would be impossible to connect the structure to the Kikuletwa South Small. The connection channel would leave the PBWB area.

Option 2; placing the structure on the west side of the bifurcation, as was suggested as possible location during the fieldwork, would be possible as it would stay in the PBWB area. However, it is expected that large amounts of debris would obstruct/hit the structure due its location. Furthermore, the Ronga would become a branch of the Kikuletwa River, while its intended to be the other way around.

Option 3; this option would be located at the current location of the Kikuletwa South Small. The same reasoning as option 2 applies here.

Option 4; this option would decrease the required length of the Kikuletwa South Small slightly and it will be easy to connect to the river. Furthermore, the angle with the Ronga river is larger than with option 2 and 3, reducing the chance on debris hitting the structure. Moreover, sufficient space is available for construction. Additionally, it is not built in an outer bend of the river, reducing the possible sedimentation that can take place.

Figure 144: Alternative locations control structure

Therefore, the decision is made to use the orientation and estimated location of option 4 for the control structure. It would be possible to build inside the PBWB area, reduces the chance on debris hitting the structure and sedimentation taking place and finally it would be easy to connect to the Kikuletwa South.

Appendix D.7 - Failure Mechanisms Control Structure

The unity checks that were performed on the control structure in Appendix B.6 - Failure Mechanisms Control Structure have been repeated for the new dimensions. In this appendix the results of these unity checks are presented. For the methodology of the checks, see Appendix B.6 - Failure Mechanisms Control Structure.

Loads

For the water levels the following values have been taken. These are different than the levels used in the initial design for the Kikuletwa South side, see Table 159.

Table 159: water depths control structure

Water depths [m]	Q (1/15)
Maximum Kikuletwa South side	2.9
Maximum Kikuletwa North side	2.79
Maximum for closed gates	2

Stability Checks

The following stability checks have been done using the Manual Hydraulic Structures (17).

Horizontal Stability

The results can be found in Table 160 below.

Table 160: Horizontal stability control structure

	Q (1/15)
f [-]	0.4
$\sum H$ **[kN/m]**	76
$f \sum V$ **[kN/m]**	114
Unity check	**1.49**

The structure is safe against horizontal stability.

Rotational Stability

The results of the stability check can be found in Table 161.

Table 161: Results Rotational stability

	Q (1/15)
$\sum M$ **[kNm/m]**	-1138
$\sum V$ **[kN/m]**	601
$\sum M / \sum V$ **[m]**	1.89
L/6 [m]	3
Unity Check	**1.59**

The structure is safe against rotational stability.

Uplift

The results of the uplift check can be found in Table 162.

Table 162: Results uplift

	Q (1/15)
$\sum V_{downward}$ **[kN/m]**	1218
$\sum V_{upward}$ **[kN/m]**	789
Unity check	**1.54**

The structure is safe against uplifting.

Vertical Stability

Table 163: Results vertical stability

	Q (1/15)
$\sum V$**[kN/m]**	618
$\sum M$**[kNm/m]**	1138
$\frac{\sum V}{bl}$**[kN/m²]**	34
$\frac{\sum M}{\frac{1}{6}lb^2}$**[kN/m²]**	21
Total Max	55
Total Min	13
Unity Check Max	**1.26**

From Table 163 it follows that the unity check for vertical stability is positive. Therefore, the structure is safe against vertical stability.

Piping

Bligh

Table 164: Results Bligh

	Q (1/15)
Seepage length [m]	17
Actual length [m]	26.4
Unity check	**1.55**

From Table 164 it follows that the actual length is larger than the seepage length of Bligh. Therefore, the structure is safe against piping.

Lane

Table 165: Results Lane

	Q (1/15)
Seepage length [m]	13.6
Actual length [m]	14.4
Unity check	**1.05**

From Table 165 it follows that the actual length is larger than the seepage length of Lane. Therefore, the structure is safe against piping.

Appendix D.8 - Lift Installation for the Control Structure

In this appendix the lift installation that can be used to operate the control structure will be elaborated. A step by step description of the chosen lift installation will be provided. The design of the total installation that is needed to operate the control structure can be split up into three parts, in each part more detail is included. First the total system will be looked at. Than the control platform will be elaborated. Followed by a more information about the gates and the crane.

Total Lift System

The lift system should make it possible to remove and place the gates without using electricity. Unnecessary complexity must be avoided to ensure that the local population can use the equipment and that the construction materials are available. The solution as presented here will meet these two main requirements.

For the lift installation a simple swivel crane with a hand chain hoist can be used, see Figure 145 for an example. A swivel crane is a steel construction that can lift heavy loads. The loads are lifted using the hand chain hoist, a device that makes it possible to lift these heavy loads using nothing more than muscle power.

Figure 145: Example of a simple swivel crane (Huches Pillar Jib Crane, 2016) and (Workstation Lifting Products: Manual Products, 2016)

Besides the crane itself there are two more things to consider: There should be enough room to store the gates when they are not placed in the gate holders and it must be possible to connect the hoist to the gates. A plan view of the total lift system can be seen in Figure 146. The plan view includes the location of the cranes, bridges, ladders, gates in closed condition and gate storage places (when the gates are open).

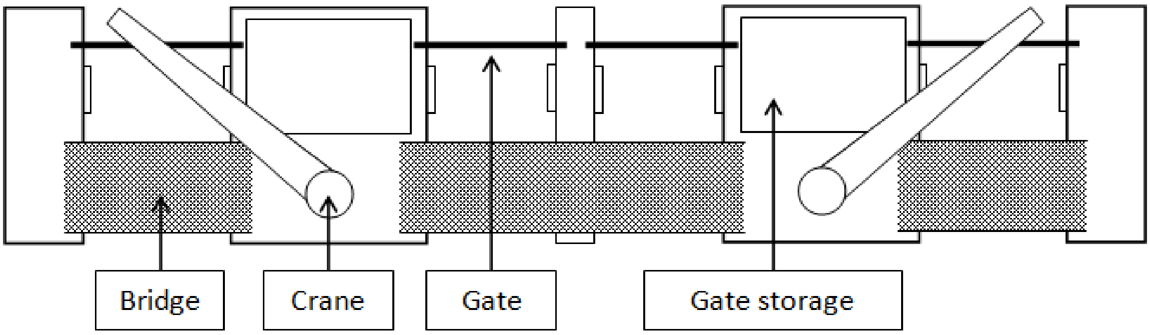


Figure 146: Plan view of the total lift system

The bridges ensure that people can reach each platform; this is essential in order to operate the control structure. It is even possible to cross the river using this bridge, which can be seen as an additional benefit. If the bridge will be used often, it is advised to make it wider than indicated in this design. The ladders ensure that the hoist can be attached to the gates; an operator can climb down and attach the hoist.

The cranes are situated on the two bigger platforms referred to as the control platforms. These control platforms are located in such a way that that each crane can operate two gate holders.

Control Platforms

The control platforms are the most important platforms in the control structure. Here the cranes can be controlled and the gates can be stored. Figure 147 gives an indication of the plan view of one of the control platforms.

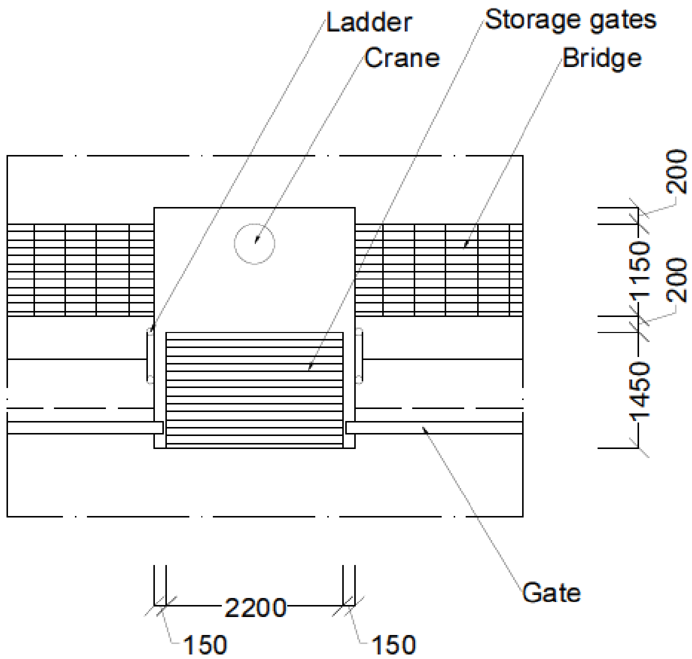


Figure 147: Plan view of the control platform

The total width of the platform is 2.5 meter and the total length is 3 meter. The storage of the gates is designed on a maximum of six gates with a width of 0.15 meter, resulting in 0.15 x 6 = 0.9 meter. An extra 0.55 meter is added, to make sure there is enough room to manoeuvre the gates to the storage place.

This results in a total storage width of 1.45 meter. On both sides of the platform 0.2 meter is reserved for a safety wall leaving 1.15 meter for the bridge.

Design of the Gates

To make the connection between the hoist and the gates possible, the gates will be equipped with two steel hooks on top of the gate, as indicated in Figure 148. Each gate will have small holes on the bottom of the gate matching the hooks. The holes ensure that it is still possible to put the gates on top of each other. The hooks and the holes may be a maximum of 5 centimetres thick to ensure that there is enough coverage on the sides of the hole, 5 centimetres on each side.

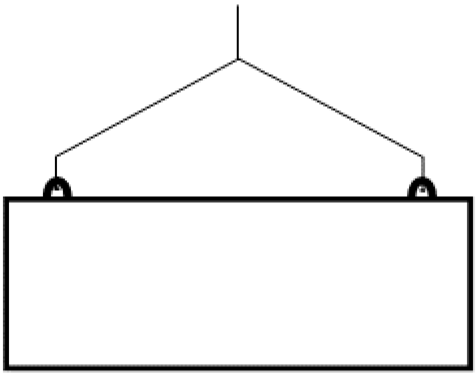


Figure 148: Gate with steel hooks

Design of the Crane

It should be possible to lift the gates by hand, because power is not available at the planned location. This can be done using a hand chain hoist, which can lift up to 20,000 kg (Workstation Lifting Products: Manual Products, 2016). The weight of the normative gate can be calculated using the volume of the gate and the specific weight of the material. The specific weight of the concrete used is estimated on 2,500 kg/m^3. The dimensions of the gate can be found in section 11.3 Control Structure, resulting in 2.2 x 0.15 x 0.8 x 2500 = 660 kg. A hand chain hoist that can lift 1,000 kg (1 ton) should therefore be sufficient.

For the vertical pole of the crane a diameter of 0.5 meter is assumed. This is just an approximation because it depends on the exact crane that is used in the final design. For the length of the crane 3.75 meters should be sufficient. This is based on the maximum reach the crane should have, from the point of its foundation to the centre of the bigger gate. The height of the crane should be at least 2.5 meters. This is based on the height of the gate and the additional room that is needed to lift the gate. The additional room consists of the triangle of chain to the centre, the hooks on the gate, the hook of the crane and the hand chain hoist.

Appendix D.9 - Calculations Morphology

In this appendix, the calculations that support the chapter 16 Morphological Effects will be elaborated. First, the flow speed in each river stretch is determined. Then the sediment load for every river stretch is calculated. With the sediment load, the equilibrium bed slope for each river stretch will be provided. Finally, a conclusion about the locations where erosion or sedimentation is likely to occur will be elaborated. All the calculations will be for the prevalent discharge during the dry season. For a complete morphology study multiple situations should be taken into account, this is however not essential for this project and a detailed study on this subject can be done in a later stage. Furthermore, the most southern part of the Kikuletwa is not included because for prevalent discharge very little water is flowing through the Kikuletwa South.

Flow Velocity

In order to determine whether sedimentation will take place, the flow velocity of each river is determined. This is done for the average water depth in the dry season. The data from IDD1 is used to determine the prevalent water depth and from Figure 149, it can be read that this is 0.8 m. By using the dimensions of the river at IDD1, which are taken from the prefeasibility study, and the average water depth at the IDD1, the flow velocity and the discharge can be determined with Manning's equation.

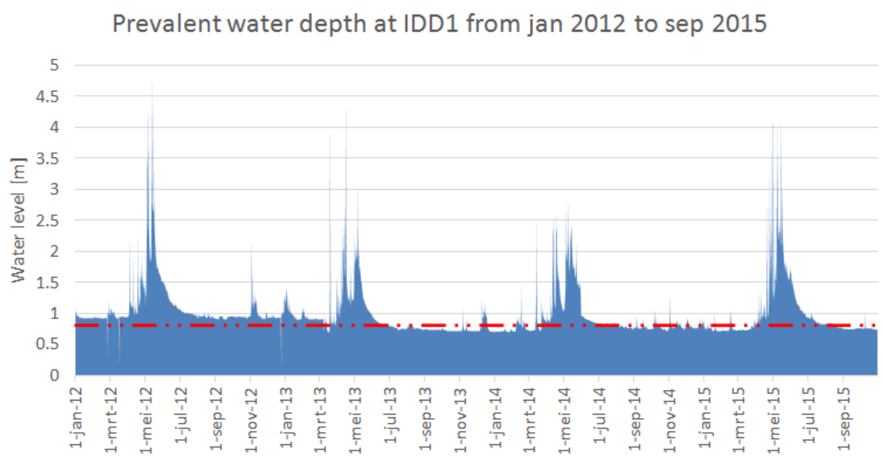


Figure 149: Prevalent water depth at the IDD1 measurement station

After IDD1, the following assumptions are made:

- All the discharge at IDD1 continues to the Kikuletwa North.
- The discharge is distributed over the New Kikuletwa South and Ronga as follows: 1 m^3/s flows into the New Kikuletwa South, the rest into the Ronga.
- At the Ronga Braided, the discharge is distributed proportionally to the capacity of both channels.
- All the dimensions of the rivers are taken from the prefeasibility study and can also be found in Appendix A.3 - Current River Dimensions.
- Except for the New Kikuletwa South, the dimensions for the Kikuletwa South can be found in chapter 15 Cluster 2.
- The manning-roughness coefficients originate from the prefeasibility study.
- The width of each river is the average width.

In Table 166 the flow velocity of each river stretch is given. Provided that the flow velocity does not decrease downstream there will be no sedimentation. Sedimentation does not necessarily mean that preventive measures must be taken.

Table 166: Flow velocity

	IDD1	Kikuletwa North	New Kikuletwa South	Ronga	Ronga Braided		Ronga South
					1	2	
Q [m³/s]	12	12	1	11	4	7	11
i_b [-]	0.0011	0.0011	0.0026	0.0014	0.0014	0.0014	0.0011
n [s/m$^{1/3}$]	0.05	0.05	0.04	0.05	0.05	0.05	0.05
h [m]	0.8	0.85	0.8	1.9	0.85	0.95	1.5
B [m]	28	24	4	7.5	8.5	11	10
A [m²]	22.4	21.3	1.6	14.1	7.2	10.5	15
P [m]	29.6	26.7	4.3	12.8	10.2	12.9	13
R [m]	0.8	0.8	0.4	1.1	0.7	0.8	1.2
u [m/s]	**0.55**	**0.57**	**0.66**	**0.80**	**0.59**	**0.65**	**0.73**

Sediment Load

The sediment load indicates the amount of sediment the river is able to transport. The sediment transport equation of Engelund-Hansen is used to determine the sediment transport and is shown in equations 49 and 50.

$$s = a\,u^b = \frac{0.05}{\sqrt{g} * C^3 * \Delta^2 * D_{50}} u^b \tag{49}$$

Where: - s is the sediment transport per unit width [m²/s]'
- a is a Engelund-Hansen coefficient [m]
- b is a Engelund-Hansen coefficient [-]
- g is the gravitational constant [m/s²]
- C is the Chézy coefficient [m$^{1/2}$/s]
- Δ is the relative density [-]
- D_{50} is the median grain diameter [m]
- u is the flow velocity [m/s]

The Engelund-Hansen coefficient b is assumed to be 5.

$$S = s * B \tag{50}$$

Where: - S is the sediment transport [m³/s]
- B is the width of the river [m]

The Chézy coefficient can be written as (equation 51):

$$C = \frac{1}{n} * R^{1/6} \tag{51}$$

Where: - n is Manning's roughness coefficient [s/m$^{1/3}$]
- R is the hydraulic radius [m]

In equation 52 the relative density is given.

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{52}$$

Where: - ρ_s is the soil density [kg/m^3];
- ρ_w is the water density [kg/m^3].

From the soil test results, see Appendix D.1 - Soils, can be concluded that the soil in the sediment mainly consists silt. The density for silt is 2793 kg/m^3 (Soil Science, 2015). From the soil test, results can also be concluded that the median grain diameter is 0.275 mm.

The sediment transport is determined for each river and can be seen in Table 167. An increase in sediment load usually indicates erosion, while a decrease indicates sedimentation.

Table 167: Sediment transport

	IDD1	**Kikuletwa North**	**New Kikuletwa South**	**Ronga**	**Ronga Braided**		**Ronga South**
					1	2	
g [m/s^2]	9.78	9.78	9.78	9.78	9.78	9.78	9.78
n [s/m$^{1/3}$]	0.05	0.05	0.04	0.05	0.05	0.05	0.05
R [m]	0.8	0.8	0.4	1.1	0.7	0.8	1.2
C [m$^{1/2}$/s]	19.1	19.3	21.2	20.3	18.9	19.3	20.5
ρ_s [kg/m^3]	2800	2800	2800	2800	2800	2800	2800
ρ_w[kg/m^3]	1000	1000	1000	1000	1000	1000	1000
Δ [-]	1.8	1.8	1.8	1.8	1.8	1.8	1.8
D_{50} [mm]	0.275	0.275	0.275	0.275	0.275	0.275	0.275
u [m/s]	0.55	0.57	0.66	0.80	0.59	0.65	0.73
B [m]	28	25	2	7.5	8.5	11	10
s [m^2/s]	**0.00013**	**0.00015**	**0.00012**	**0.00069**	**0.00020**	**0.00029**	**0.00043**
S [m^3/s]	**0.00366**	**0.00377**	**0.00047**	**0.00519**	**0.00168**	**0.00319**	**0.00432**

Equilibrium Bed Slope

The equilibrium bed slope indicates the slope the bed of the river will have after a long time. Even when at a certain location erosion or sedimentation occurs, the river will reach a natural equilibrium. The equilibrium bed slope can be calculated using equation 53.

$$i_{eq} = \left(\frac{S}{aB}\right)^{\frac{3}{b}} * (\frac{B}{C^2 Q}) \tag{53}$$

Where: - i_{eq} is the equilibrium bed slope [-]
- Q is the discharge in the river [m^3/s]

The equilibrium bed slope is determined for each river and can be seen in Table 168. The initial slope i_b is also included to compare to the equilibrium slope. A higher value for the equilibrium slope indicates a steeper slope and a lower value indicates a less steep slope.

Table 168: Equilibrium bed slope

	IDD1	Kikuletwa North	New Kikuletwa South	Ronga	Ronga Braided 1	Ronga Braided 2	Ronga South
Q [m^3/s]	12	12	1	11	4	7	11
B [m]	28	24	4	7.5	8.5	11	10
C [m$^{1/2}$/s]	19.1	19.3	21.2	20.3	18.9	19.3	20.5
S [m^3/s]	0.00366	0.00377	0.00047	0.00519	0.00168	0.00319	0.00432
i_b [-]	0.0011	0.0011	0.0026	0.0014	0.0014	0.0014	0.0011
i_{eq} [-]	**0.00104**	**0.00103**	**0.0012**	**0.00082**	**0.00117**	**0.00119**	**0.00085**

Erosion or Sedimentation

In Table 169 a list of the river stretches is given with the conclusions about the morphological effects that are likely to occur after the implementation of the solution proposed in this report. With the words little, medium and severe an indication is given about the extent of the erosion or sedimentation. At some locations no conclusion about the sediment load is given, this is because no conclusion can be drawn from the resulting data. This is especially the case at a bifurcation point, with the method used for this study it is not possible to determine the exact distribution of sediment at the bifurcation point.

Table 169: Conclusions about the morphological effects

	Conclusion using the flow speed	Conclusion using the sediment load	Equilibrium bed slope	Final conclusion
Kikuletwa North	Erosion	Erosion	0.00103	Little erosion
New Kikuletwa South	Erosion	-	0.0012	Medium erosion
Ronga	Erosion	Erosion	0.00082	Severe erosion
Ronga Braided	Sedimentation	Sedimentation	0.00117/0.00119	Medium sedimentation
Ronga South	Erosion	-	0.00085	Medium erosion

Appendix D.10 - Validation of the Design

The design was validated by the criteria set during the analysis which can be found in chapter 6 Validation. For the different criteria the reasoning will be given if the design meets them. The results are also presented in chapter 18 Validation.

Risks; The uncertainties in the data and assumptions were kept to a minimum. Unfortunately, due to the available time there are still uncertainties left but they have been identified. During the next stages of the design these can be resolved.

Cost Effectiveness; This criterion is met as the benefits outweigh the costs for the design and thus a positive result will be achieved.

Water Management of the Agricultural Land; The design takes into account that it should be possible to irrigate, drainage and manage the discharges. Controllable spillways have been included in the Samanga Dike to allow for irrigation. The control structure is capable of handling the different discharges throughout the year and no dikes are located at locations that prevent the drainage of the land.

Locations of the designs; The planned construction locations are available for construction with no objects located there that cannot be moved. Furthermore, the total area where construction will take place has been significantly reduced compared to the feasibility study, reducing the impact on the surrounding area.

Resources and Construction Methods; The design has been discussed with local contractors to check if the materials area available and the structures can be built. This is the case for the design, although the crane for the control structure has to be locally made or imported from another country.

Morphological Effect; A first study has been conducted for the morphological effects and recommendations have been made. However, the data available is insufficient to draw a definitive conclusion about the morphological effects and this will need to be done at a later stage.

Operation; The same type of structures is used in the design which are all manually operable. Furthermore, they can be operated by muscle. The control structure requires more effort than was first designed, however, this was the only way to handle the different discharges. Furthermore, the number of times an intervention is required is limited.

Maintenance; The designed structures are relatively simple and should be easy to maintain. Some knowledge will be required for the maintenance, which will always be the case.

Longevity; With proper maintenance the structures should be able to last for the set timespan. The damage in case of exceedance of the design discharge has been taken into account in the cost estimation.

Appendix E – Risks and Implementation

Appendix E.1 - Risk Register

The risk register is a document, which should be updated regularly during the entire process of the project. The following risk register shows the detected potential risks up until this point in the project. Additional risks that are identified later on in the project can be added to the risk register.

Components

The risk register includes the following components. This section should serve as a reading guide.

Categories: The risks are divided up into the following categories:

- Preparation; risks during the preparation phase of the project till the moment the construction commences.
- Construction; risks during the construction phase of the project.
- Operation and Maintenance; risks during the lifetime of the structures.
- External; risks that are outside the direct control of the project.

Causes: The cause(s) that lead to risk is explained. This is formulated as a fact.

Risk Events: The risk event is identified next. This is formulated as a chance.

Consequences: The consequences are identified as:

- Project success; the solution is not working as intended.
- Costs; the costs of the project increase.
- Time; the project is delayed.
- Safety; the local inhabitants are at risk.

Probability: The probability that the risk will happen will be determined in the following range:

- Very Low;
- Low;
- Medium;
- High;
- Very High.

Impact: The impact is determined in the same way as the probability:

- Very Low;
- Low;
- Medium;
- High;
- Very High.

Response: The responses to the risk that were used are formulated as:

- Reduce; decrease the chance/impact of the risk.
- Avoid; completely prevent the risk from happening.
- Enhance; the positive impact of the risk will be enhanced.
- Accept; the chance/impact of the risk are accepted and no action will be taken.

Actions: The action that should be done for the response, before and after the potential risk occurs.

General

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
1	Preparation	Incorrect data gathered, assumptions made and errors in calculations during the prefeasibility study and current study.	The design is wrongly dimensioned, allowing either flooding to still take place during the short rains or no flooding during long rains.	Project success, safety, costs.	Medium	High	Reduce	Validate the prefeasibility design. Review work of colleagues during current study. Check the gathered data. Have a professional engineering company validate both studies and make a detailed design. Have entire area surveyed.
2	Preparation	Tanroad Laboratory makes a mistake while examining the soil samples, leading to the soil test result not representing the reality.	The wrong soil characteristics are used for the design causing failure.	Project success, safety, costs, time.	Low	Very High	Reduce	Send a dummy soil sample to check if the results come back the same.
3	Preparation	Sponsors not found for the project.	Insufficient funding for the project.	Project success, time.	Low	Very High	Reduce	Make an appealing benefit picture to convince sponsors. Keep costs as low as possible.
4	Preparation	The measurements gathered from IDD1 are incorrect causing the design to be wrongly dimensioned.	The floods cannot be controlled.	Project success, safety, costs.	Low	Very High	Reduce	Check with locals if data corresponds with experience. Install measuring stations along river.
5	Preparation	The Pangani Basin Water Board disagrees with the solution.	No permission is given for the project, meaning the project cannot take place.	Project success, time.	Very Low	Very High	Avoid	Include PBWB in the process from the start; involve them in the decision-making.
6	Preparation	The cost estimation is based on many assumptions causing the actual costs to deviate from the estimation.	The required funds are higher than expected	Time, Costs	Medium	High	Reduce	Determine a range of costs to take this variation into account. Meet with other contractors and engineers to gain more data about the possible costs.

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
7	Preparation	Loss of information between stages of the project causing construction, operation and maintenance to go wrongly.	Damage/failure of design.	Project success	Medium	High	Reduce	Make a project team for the duration of the project. This team should supervise the work done by the engineering company and contractors to check if things go, as they should. Assist in making maintenance and operation plans.
8	Construction	Contractor has insufficient knowledge, cuts corners, thinks own way is better, miscommunication with designer, designer not present to answer questions, causing the contractor to deviate from the design.	Structures do not meet the standards.	Project success, safety, time, costs.	Medium	High	Reduce	Make a communication network between designer and contractor. Reviews and checks during construction process by the project team. Select contractors based on reputations to ensure they have sufficient quality. Choose right kind of contract.
9	Operation and Maintenance	Floating objects in the river (trees, bushes, etc) cause blockages in river.	Damage/failure of design.	Safety, costs.	Medium	High	Reduce	Educate local community that blockages need to be removed as soon as possible. Remove the blockages before they can damage structures.
10	Operation and maintenance	Due to insufficient, visible results in the start, the local farmers stop supporting the solution and do what they think is best.	The structures are not maintained and are damaged.	Project success, costs.	Low	Mediu m	Reduce	Education and communication with the local community (FTK). Maintenance and Operation plans. Have periodic checks to see if maintenance and operation is following to plan.
11	External	Extreme weather conditions causing excessive flooding.	Morphological change of the river, therefore	Project success, safety.	Medium	Very High	Reduce	Try to determine long-term morphological response of

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
		River responds in an extreme way to structures.	changing its course.					the river.
12	External	River discharge exceeds design discharge.	Damage of design.	Safety, costs.	Low	Medium	Accept	Design robust structures. Repair damages.
13	External	Msitu wa Tembo district decides to build a dike on their side of the river causing the water level to exceed the design level.	Failure of design.	Safety, costs.	Very Low	Very High	Reduce	Maintain contact with Msitu wa Tembo and update them on plans.
14	External	Afforestation/ Deforestation in the Kilimanjaro catchment and global warming causing different river discharges.	Design is wrongly dimensioned; farmland does not flood anymore/too much.	Project success.	Medium	Low	Reduce	Design flexible structure that can deal with a variety of water levels.
13	External	Due to insufficient visible results in the start, the local farmers do not trust the solution.	The farmers do not plant as much as they could.	Project success.	Medium	High	Reduce	Educate farmers on the solutions and how they work.

Samanga Dike

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
1	Preparation	Assumptions made for floodplains, roughness and slope of bed, river profile are not correct causing the design to be wrongly dimensioned.	Flooding still takes place during the short rains and too much or not enough during the long rains.	Project success, safety, costs.	Medium	High	Reduce	Have entire area surveyed, adjust design accordingly.
2	Preparation	Dike does not reach far enough along the Ronga.	Unwanted flooding.	Project success, safety.	Low	High	Reduce	Include in design, obtain an accurate height map.
3	Preparation	The design is wrongly dimensioned causing mass instability.	Dike failure.	Project success, safety, costs.	Low	High	Reduce	Do stability checks during design. Regular checks to see if the dike shows signs of failure. Repair and strengthen if occurs.
4	Preparation	The design is wrongly dimensioned causing seepage.	Dike failure.	Project success, safety, costs.	Medium	High	Reduce	Include preventative measures in the design such as impervious layer, stabilizing berm, cut-off walls, toe drains or relief wells. Repair and strengthen if occurs.
5	Preparation	The design is wrongly dimensioned causing overtopping or scouring which causes internal erosion.	Dike failure.	Project success, safety, costs.	Medium	High	Reduce	Include surface protection in the design, place dike a certain distance from the river. Repair and strengthen if occurs.
6	Preparation	The design is wrongly dimensioned causing piping.	Dike failure.	Project success, safety, costs.	Low	High	Reduce	Make piping length sufficient in the design. Repair and strengthen if occurs.
7	Preparation	The design is wrongly dimensioned causing settlement.	Dike failure.	Project success, safety, costs.	Very Low	Medium	Reduce	Monitor settlement. Reconstruct dike to design height.

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
8	Preparation	Local deviation in soil properties not accounted for causing weak points in the dike.	Dike breach.	Project success, safety, costs.	Low	High	Reduce	Do more extensive soil testing. Repair and strengthen if occurs.
9	Construction	Dike not built to correct height. Causing overtopping during high water levels, which causes erosion.	Dike breach and unwanted flooding.	Project success, safety, costs.	Low	High	Reduce	Do checks during construction. Repair and heighten if occurs.
10	Operation and Maintenance	Farmer's dig their own irrigation channels through the dike.	Dike breach.	Project success, safety, cost.	Medium	High	Reduce	Educate farmers on consequences. Repair breach if occurs.

Spillways

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
1	Preparation	The design is wrongly dimensioned causing horizontal sliding, rotational instability or insufficient vertical bearing capacity.	Structure fails.	Project success, safety, costs.	Low	High	Reduce	Design safe structure. Run stability checks. Repair and strengthen if occurs.
2	Preparation	The design is wrongly dimensioned causing piping.	Structure fails.	Project success, safety, costs.	Medium	High	Reduce	Make piping length sufficient in the design. Run stability checks. Repair and strengthen if occurs.
3	Preparation	Assumptions made for floodplains, roughness and slope of bed, river profile are not correct.	Structure fails or does not operate as it should.	Project success.	Medium	High	Reduce	Have area surveyed.
4	Preparation	The design is wrongly dimensioned which causes high flow velocities to cause scour before and after the structure.	Structure fails.	Project success, safety, costs.	High	High	Reduce	Apply scour protection in the design. If scouring is observed, repair as soon as possible. Temporary measures (e.g. sandbags).
5	Preparation	The design is wrongly dimensioned causing settlement causing cracking of concrete.	Structure fails.	Project success, safety, costs.	Very Low	Medium	Reduce	Monitor settlement. Fill cracks to protect reinforcement from corrosion.
6	Preparation	Spillways located at a weak spot in the dike.	Structure fails.	Project success, safety, costs.	Low	High	Reduce	Choose definitive locations only after proper surveying. Repair and strengthen if occurs.
7	Preparation	The river width varies over the length of the river along with the depth causing the water level being too low for the spillways	Not enough discharge through the spillways for sufficient flooding or irrigation	Project success.	High	High	Reduce	Survey the river; design the spillways on local river dimensions.

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
8	Operation	Controllable structure not being used in a fair way because farmers don't know how to operate the spillways and/or no clear division of responsibility causing the structure not to operate as it should.	Agricultural land not flooded sufficiently during long rain season, flooding through spillways during short rains.	Project success.	High	Medium	Reduce	Appoint person(s) to be in charge of spillways. Make operation plans and educate community on these. Communication (FTK)
9	Operation	Flood wave arrives at structure before gates have been configured properly.	Structure does not operate as it should.	Project success.	Medium	High	Reduce	Devise warning system.

Control Structure

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
1	Preparation	The design is wrongly dimensioned causing horizontal sliding, rotational instability or insufficient vertical bearing capacity.	Structure fails.	Project success, safety, costs.	Low	High	Reduce	Design safe structure. Run stability checks. Repair and strengthen if occurs
2	Preparation	The design is wrongly dimensioned causing piping.	Structure fails.	Project success, safety, costs.	Medium	High	Reduce	Make piping length sufficient in the design. Run stability checks. Repair and strengthen if occurs
3	Preparation	Assumptions made for floodplains, roughness and slope of bed, river profile are not correct.	Structure fails or does not operate as it should.	Project success.	Medium	High	Reduce	Have area surveyed, design a flexible structure.
4	Preparation	The design is wrongly dimensioned which causes high flow velocities to cause scour before and after the structure.	Structure fails.	Project success, costs, safety.	High	High	Reduce	Apply scour protection in the design. If scouring is observed repair as soon as possible. Temporary measures (e.g. sandbags).
5	Preparation	The design is wrongly dimensioned causing salt intrusion.	Structure fails.	Project success, safety, costs.	Low	High	Reduce	Include measures to counteract this in the design. Repair and strengthen if occurs.
6	Preparation	The design is wrongly dimensioned causing settlement causing cracking of concrete.	Structure fails.	Project success, safety, costs.	Very Low	Medium	Reduce	Monitor settlement. Fill cracks to protect reinforcement from corrosion.
7	Preparation	Sedimentation causes blockage of the control structure and water level rise.	Structure does not operate as it should.	Project success.	Medium	High	Reduce	Investigate the impact of the structure on sedimentation. Remove sediment if it occurs.

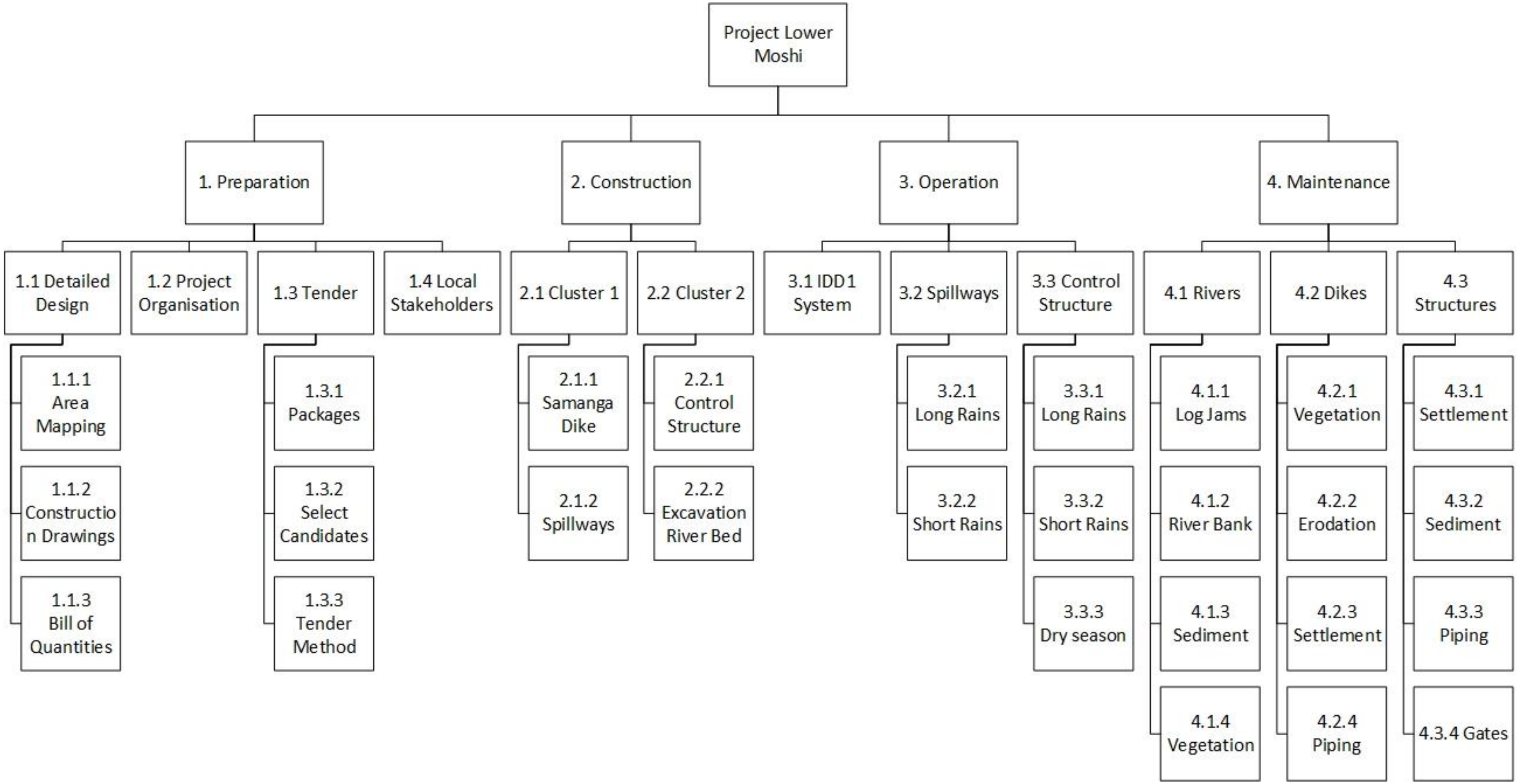
#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
8	Preparation	Backwater curves arise due to structures causing an increase in water level upstream.	The Samanga dike overtops.	Project success, safety.	Low	Medium	Reduce	Make a calculation to see if backwater curves arise. Increase dike height if this occurs.
9	Construction	The rain season starts early causing the building pit to flood, or the walls cannot handle the water pressure from the river.	Construction delayed.	Costs, time.	Low	High	Reduce	Place building pit at a distance from the river. Pump out water if it occurs.
10	Operation	Farmer's do not know how to operate the control structure. No clear division of responsibility for the operation.	Structure does not operate as it should	Project success.	Medium	High	Reduce	Appoint person(s) to be in charge of control structure. Make operation plans and educate community on these. Communication (FTK)
11	Operation	Flood wave arrives at structure before gates have been configured properly.	Structure does not operate as it should.	Project success.	Medium	High	Reduce	Devise warning system.

New Kikuletwa South

#	Category	Cause	Risk event	Consequence	Probability	Impact	Response	Actions
1	Preparation	Not enough discharge going through Kikuletwa South during dry season.	Silting up of the Kikuletwa South.	Project success.	Medium	Medium	Reduce	Include in design. Remove the silt.
2	Preparation	Assumption for natural slope is not correct causing erosion or sedimentation of river profile.	River changes shape, does no longer meet design standards.	Project success, costs.	Medium	Medium	Reduce	Research the parameter.
3	Preparation	Assumptions made for roughness and slope of bed are not correct.	New river cannot handle expected discharge.	Project success.	Medium	High	Reduce	Have area surveyed to gain better assumptions.
4	Preparation	Water levels too high at river crossing.	Cars and people cannot cross the river.	Safety.	High	High	Accept	Build a bridge.
5	Preparation	The discharge arriving at the confluence downstream from the bifurcation is very high.	Extreme flooding downstream.	Project success, safety.	High	High	Reduce	Install measuring station at confluence.
6	Preparation	Capacity of the Kikuletwa South downstream is lower than expected causing extreme flooding.	Houses get damaged, crops/livestock lost.	Project success, costs.	Low	Very High	Reduce	Survey the Kikuletwa South to determine exact capacity.
7	Preparation	The excavation causes water for irrigation purposes to become available in previously dry areas.	New farmland is created.	Project success.	Medium	Medium	Enhance	Distribute land fairly to compensate those who lost land due to construction. FTK should be involved in this procedure.
8	Construction	People living in houses located at excavation location do not want to move somewhere else.	Not possible to excavate.	Time, costs	Low	High	Reduce	Identify potential houses. Contact the inhabitants at an early stage, compensate them for their loss.
9	Construction	Rock is present in the soil at the location of the planned excavation.	Excavation not possible with planned equipment.	Time, cost	Medium	Medium	Reduce	Do soil testing and surveying. Find an alternative route, use different equipment

Appendix E.2 - Work Breakdown Structure

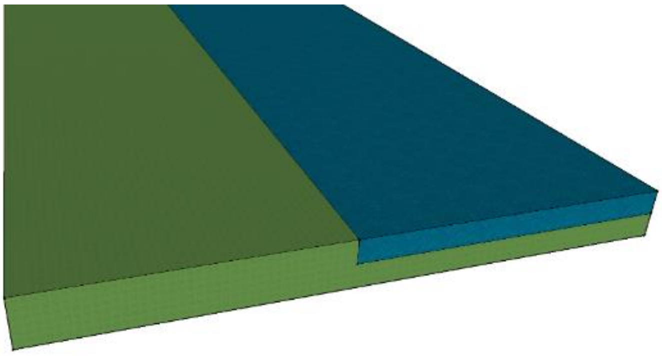
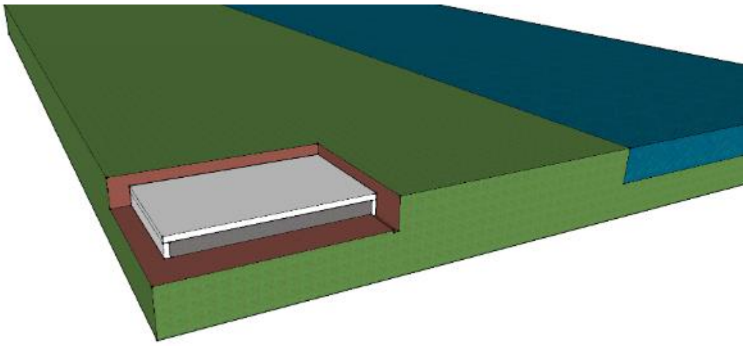
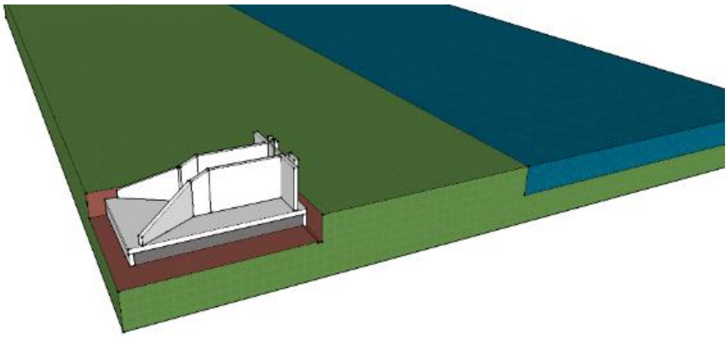
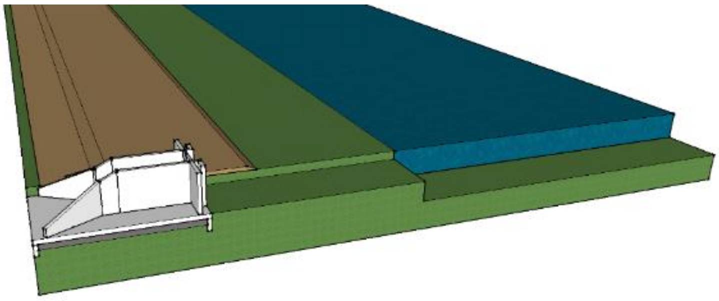
This appendix shows the complete work breakdown structure that is explained in 21 Implementation.

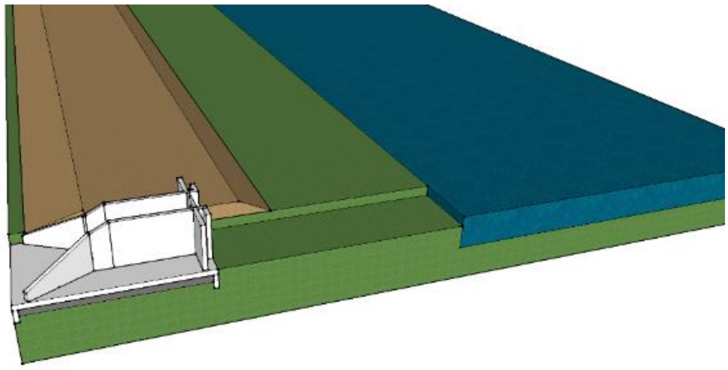
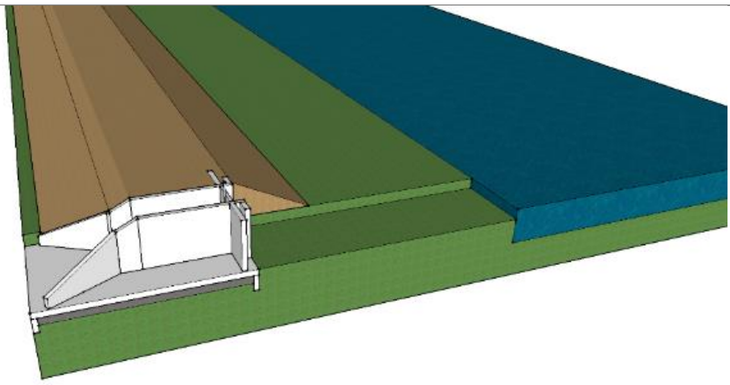
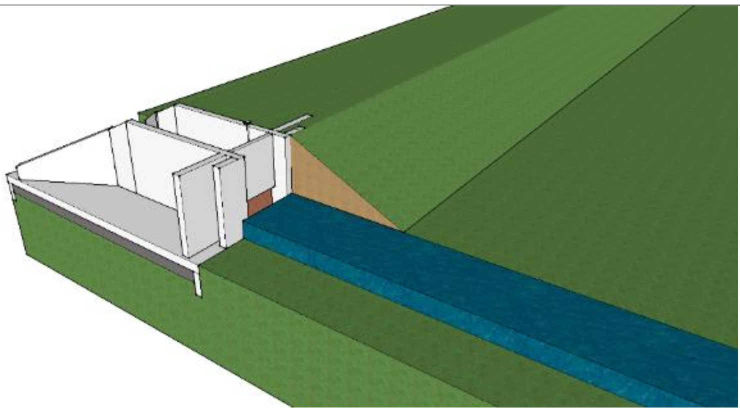
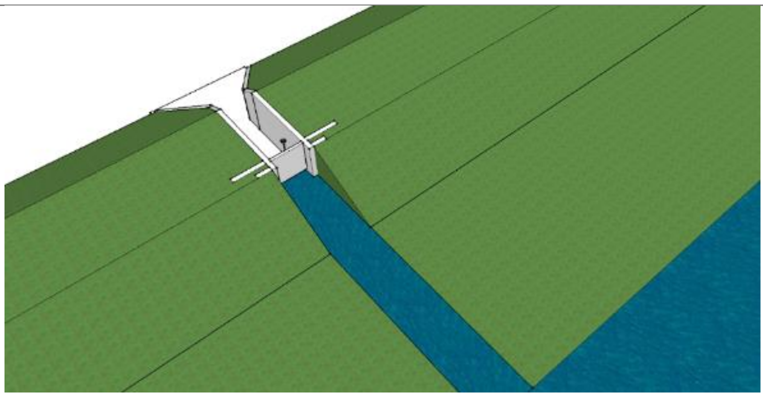


Appendix E.3 - Construction Visualisation

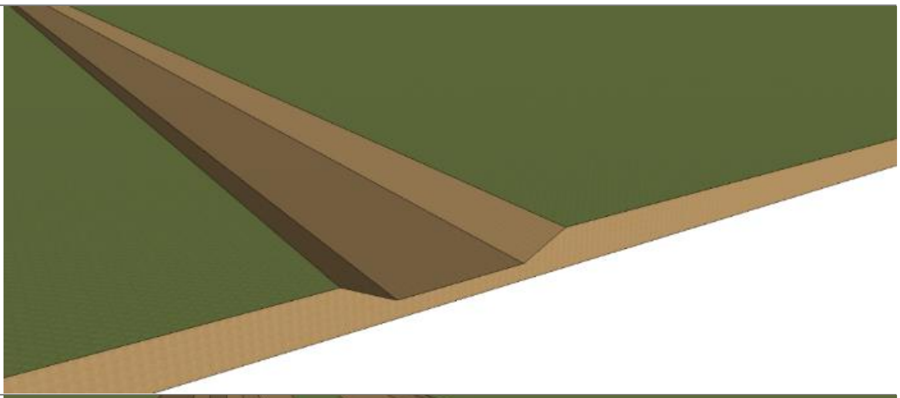
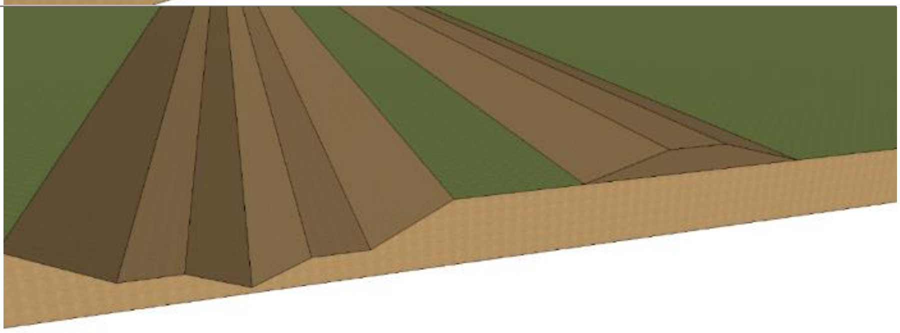
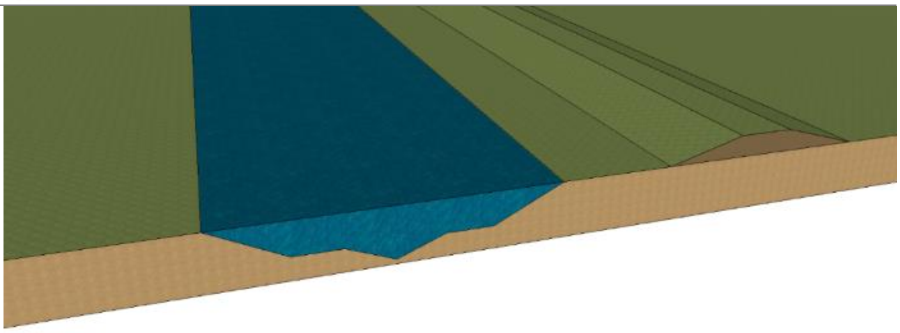
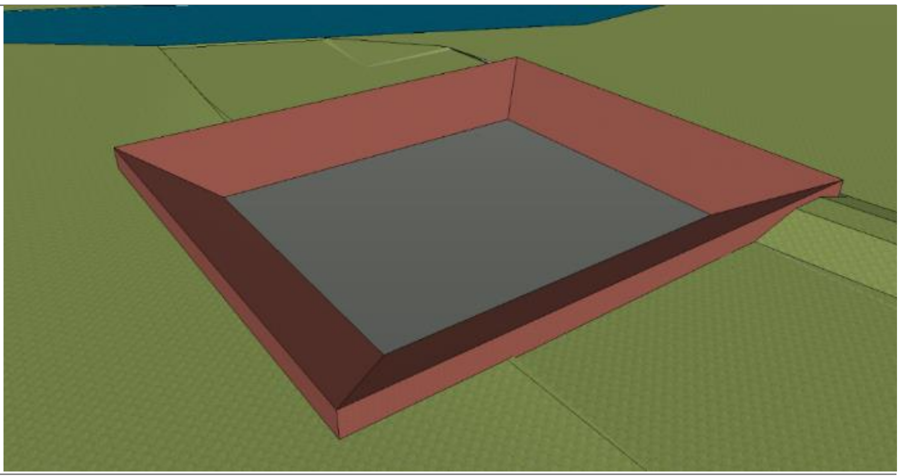
In this appendix the phasing of the construction is visualised. The construction phasing is elaborated in section 21.3 Construction.

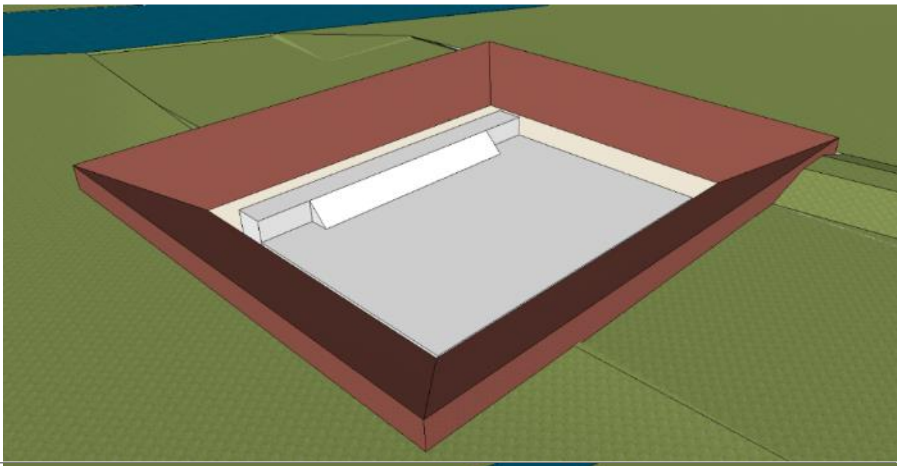
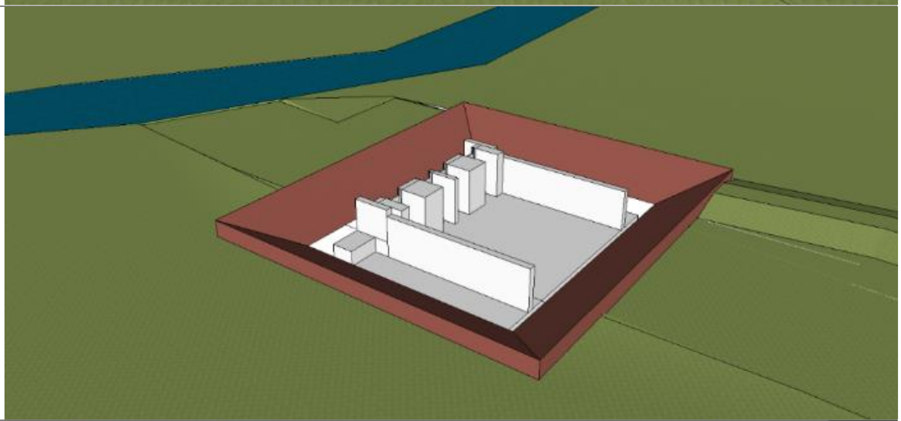
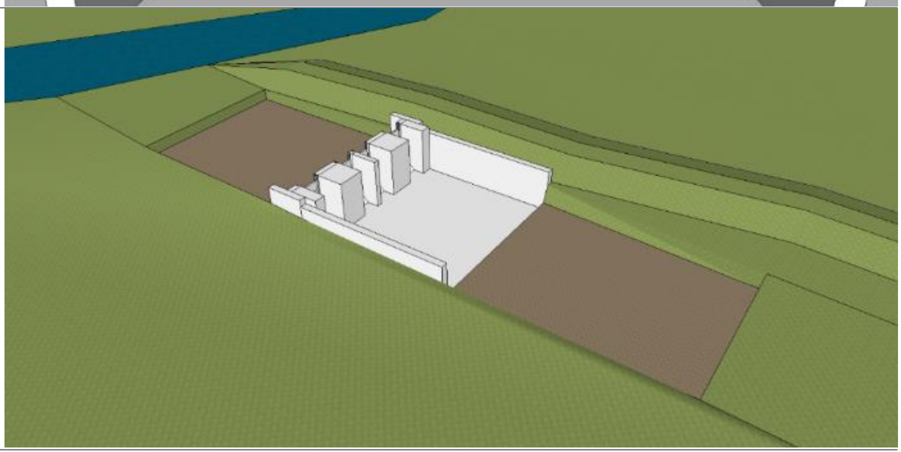
Cluster 1

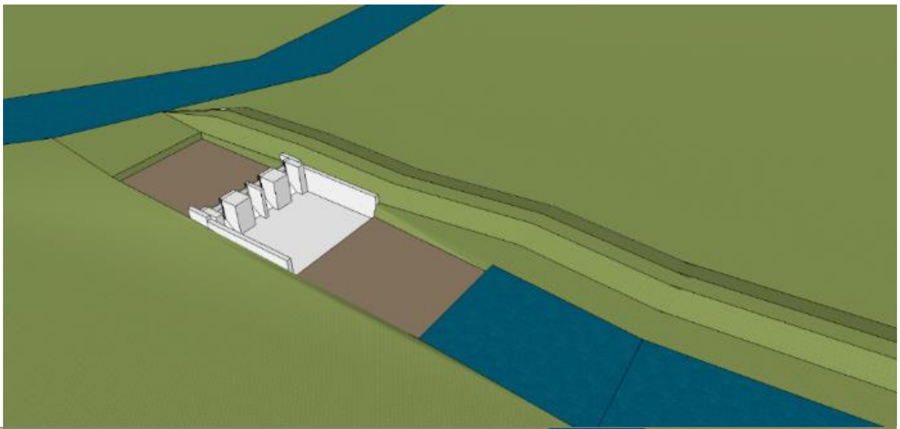
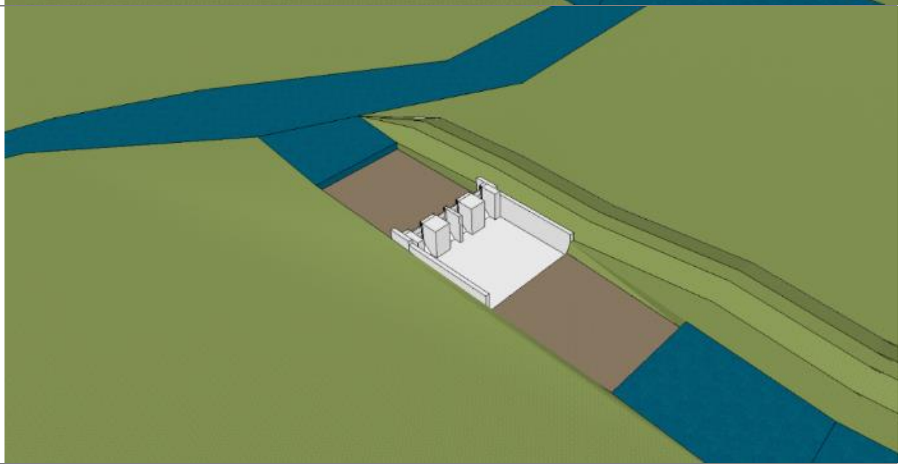
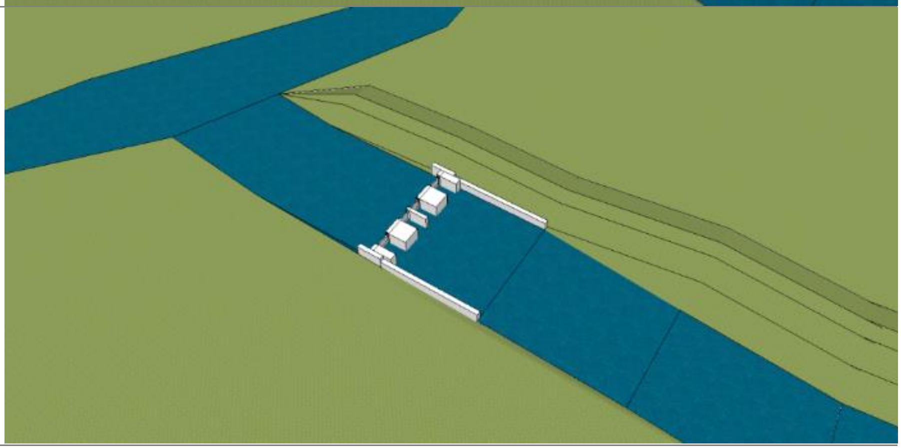
Removing vegetation and organic topsoil layer	
Excavate foundation spillway, add rock layer and pour concrete foundation	
Make rest concrete construction and add steel door mechanism	
Construct embankment and entrance path spillway	

Construction and compacting dike in layers of 30 cm	
Finish construction dike	
Add layer of organic soils and plant Vetiver grass	
Final	

Cluster 2

Excavation Kikuletwa South downstream	
Further excavation and making dike	
Final look New Kikuletwa South and dike	
Make building pit control structure and add rock layer	

Pour concrete foundation	
Make rest concrete structure	
Place gates	
Remove building pit and make scour protection	

Connect Kikuletwa South to the control structure	
Connect control structure to Ronga	
Final	

Appendix E.4 - Visualisation Operation Phase

This appendix will treat the visualisation of the different operation procedures for the spillways and the control structure. The reasoning behind the different configurations is treated in chapter 21 Implementation. First the procedure during the long and short rain season for the spillway will be presented. Then the procedure during the short and long rain and dry season will be presented for the control structure.

Spillways

Long Rains

In the long rain season the flooding of the Samanga area behind the dike is wanted, therefore the spillways should be opened.

Minimum Discharge

The bank full discharge is reached when a minimum discharge is in the Kikuletwa North, see Figure 150. The spillways should be opened for four days in order to flood the Samanga area.

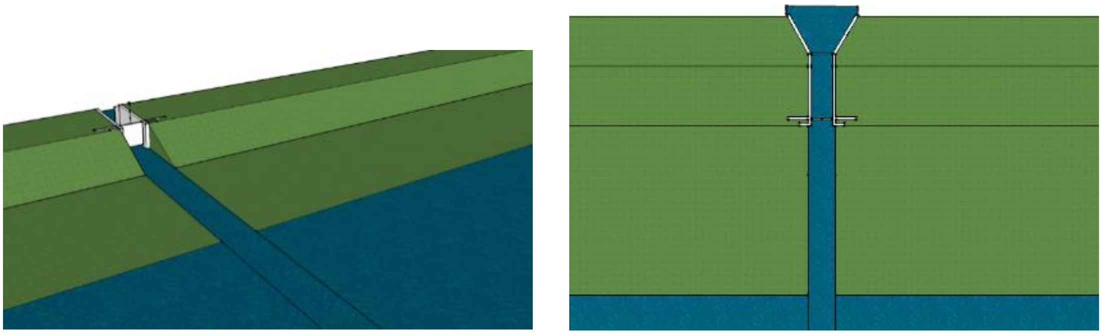


Figure 150: Spillway Long Rains Minimum Discharge

Maximum Discharge

The bank full discharge is exceeded when there is a maximum discharge in the Kikuletwa North, see Figure 151. The spillways should be opened for one day in order to flood the Samanga area.

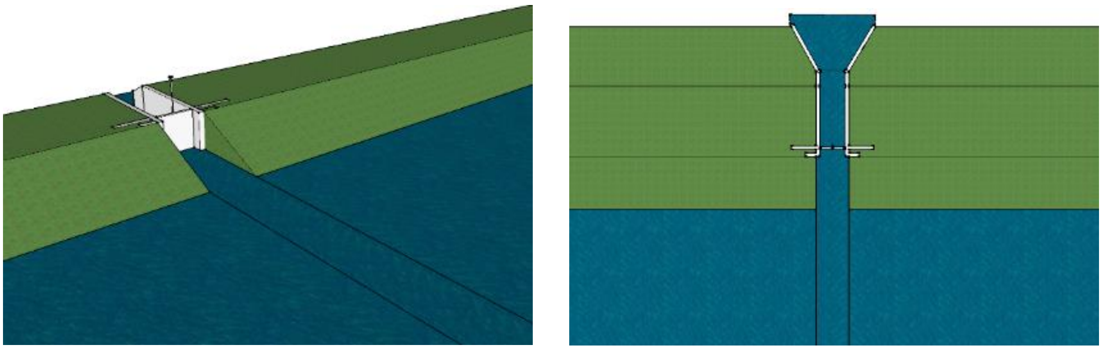


Figure 151: Spillway Long Rains Maximum Discharge

Short Rains

In the short rain season the flooding of the Samanga area is unwanted, thus the spillways should be closed as shown in Figure 152.

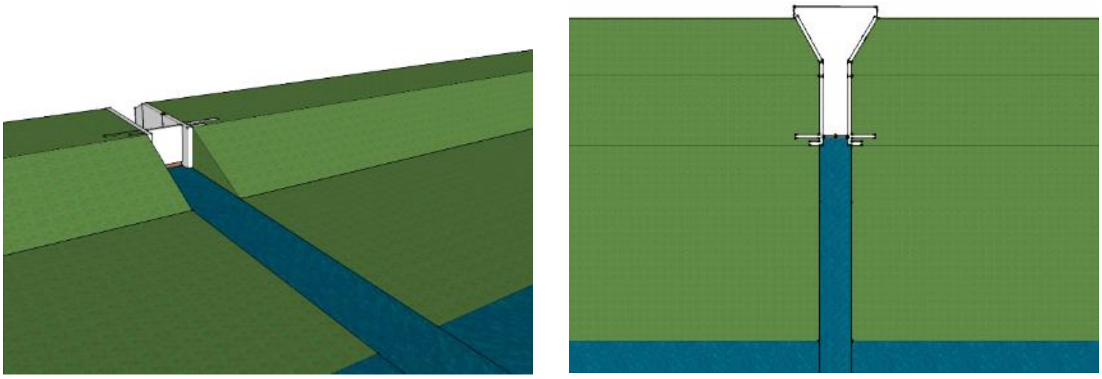


Figure 152:Spillway Short Rains

Control Structure

Long Rains

In the long rain season the flooding of the Ronga area is wanted, so the control structure has to ensure that enough water is flowing through the Ronga to flood the area.

Minimum Discharge

One gate should be opened to ensure that a sufficient amount of water is flowing through the Ronga while enough water enters the Kikuletwa South to prevent drying up, see Figure 153.

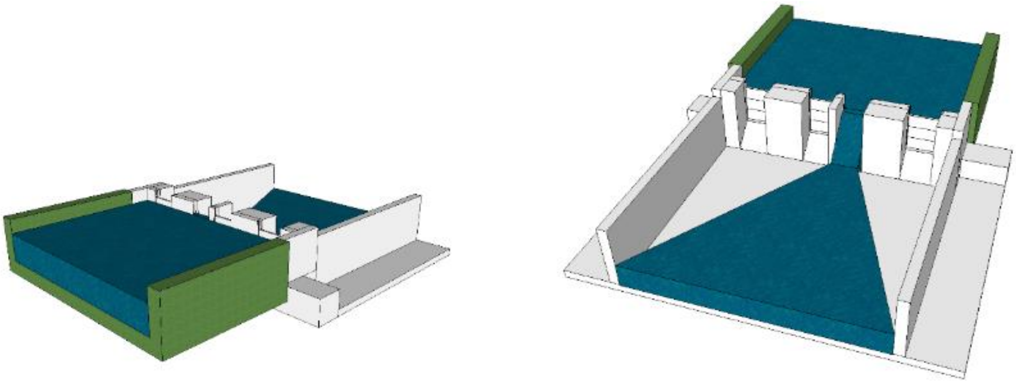


Figure 153: Control Structure Long Rains Minimum Discharge

Maximum Discharge

All the gates should be opened to ensure that the excessive amount of water is flowing into the Kikuletwa South while the Ronga will still flood, see Figure 154.

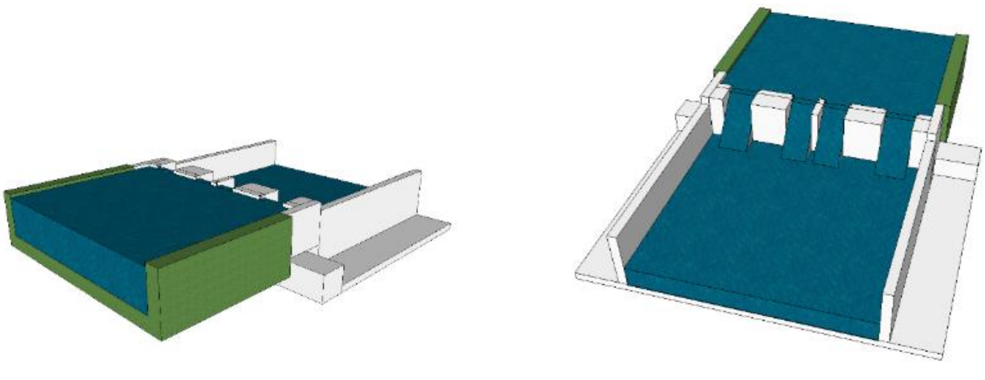


Figure 154: Control Structure Long Rains Maximum Discharge

Short Rains

In the short rain season the flooding of the Ronga area is unwanted, so the control structure has to ensure that the excessive amount of water will flow through the Kikuletwa South while enough water will be available in the Ronga for irrigation.

Minimum Discharge

One gate should be opened to ensure enough water in the Ronga for irrigation, while enough water is entering the Kikuletwa South to avoid drying up of the river, see Figure 155.
This situation is also valid for the dry season when water should flow through both rivers to avoid drying up of the rivers.

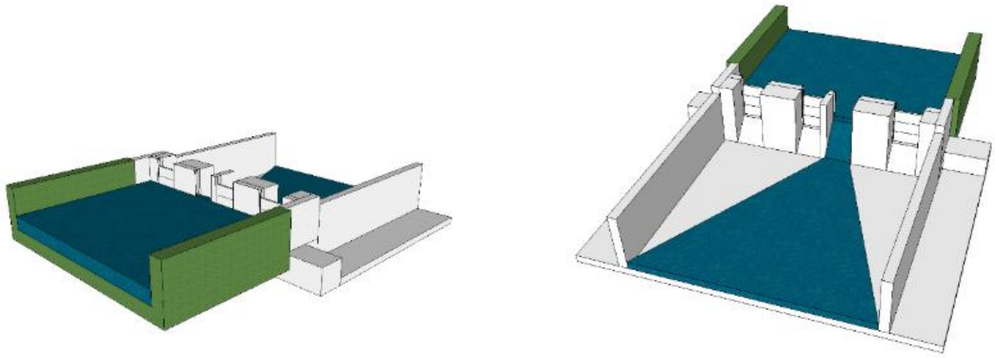


Figure 155: Control Structure Short Rains Minimum Discharge

Maximum Discharge

Two gates should be opened so that the excess water can flow into the Kikuletwa South in order to prevent flooding of the Ronga, but still ensure enough water in the Ronga for irrigation, see Figure 156.

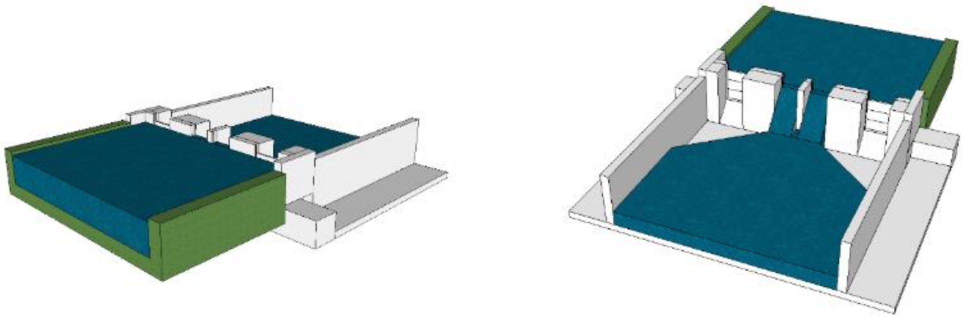


Figure 156: Control Structure Short Rains Maximum Discharge

Appendix F – External Files

Appendix F.1 - Fieldwork

Field Reports

A number of field reports have been written regarding the field visits that took place. The names of the files are stated below and can be supplied upon request.

- Field Report 1 12-11-2015
- Field Report 2 17-11-2015
- Field Report 3 18-11-2015
- Field Report 4 24-11-2015
- Field Report 5 25-11-2015
- Field Report 6 03-12-2015

Soil Logs

During the fieldwork, soil samples were taken and these were at a later point in time tested by Tan Roads. The log sheets were devised during the fieldwork and minor alterations were done after the results of the tests were known. The soil logs can be supplied upon request. The location of the soil logs can be found in the field reports. To understand the soil logs firstly it is advised to look at these two documents:

- Labelling System
- Soil Logs Definitions

The soil logs can be found in these documents:

- Logsheet Location 1B
- Logsheet Location 1C
- Logsheet Location 1D
- Logsheet Location 1G
- Logsheet Location 1H
- Logsheet Location 1I
- Logsheet Location 2A
- Logsheet Location 2B
- Logsheet Location 3A

Appendix F.2 - Soil Results

The soil results that were provided by Tanzania National Roads Agency (TanRoads) can be found in the following files. They performed tests on soil gradation, Atterberg limits and moisture content.

- Lab No. 1Bg I
- Lab No. 1Bg II
- Lab No. 1Cg I
- Lab No. 1Dg I
- Lab No. 1Dg II
- Lab No. 1gH
- Lab No. 2Ag 2.4
- Lab No. 2Bg II
- Lab No. 3Ag I
- Lab No. 3Ag II
- Lab No. Sediment 1

Appendix F.3 - Excel Files

During the course of the project various excel files were used to make calculations. These excel files can be provided upon request.

Analysis

The following documents were used to determine the river capacities and the design discharges through the clusters in the analysis.

- MaxManning Discharge per Section
- Long Rain Return Period
- Short Rain Return Period
- Design Discharge Cluster 1
- Design Discharge Floodplains
- Design Discharge Cluster 2

Initial Design

These documents were used to dimension and determine the failure mechanisms of the initial design

- Samanga Dike Initial Design
- Spillways Design and Failure Mechanisms
- Excavation Initial Design
- Control Structure Calculations Initial Design
- Discharge Through Control Structure
- Unit Prices and Quantities Initial Design
- CBA Initial Design

Integral Design

- Excavation Integral Design
- Reference Level
- Control Structure Calculations Integral Design
- Morphology Calculations
- Scour
- Vegetation
- CBA Integral Design
- Sensitivity Analysis