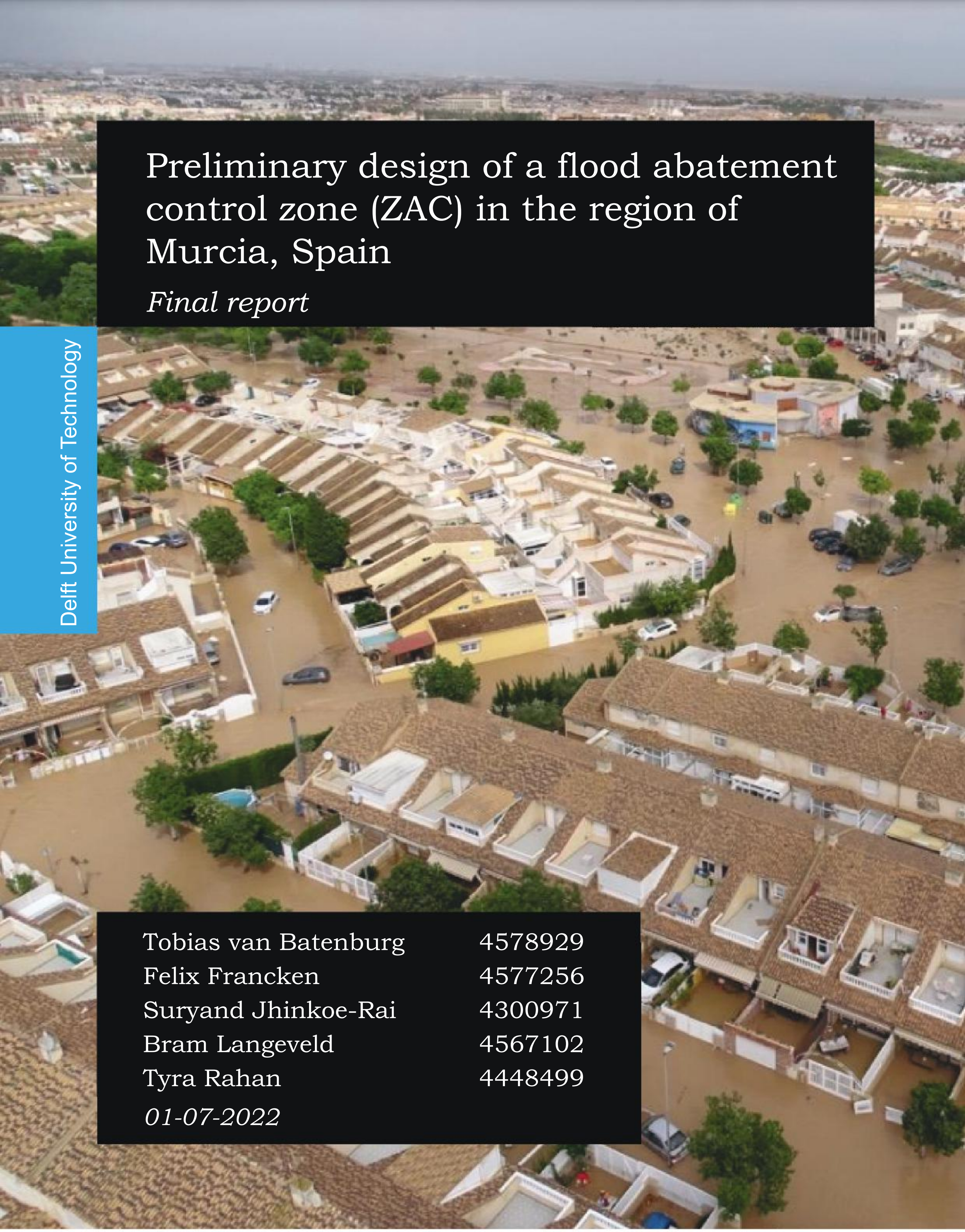


Preliminary design of a flood abatement control zone (ZAC) in the region of Murcia, Spain

Final report

Delft University of Technology

Tobias van Batenburg	4578929
Felix Francken	4577256
Suryand Jhinkoe-Rai	4300971
Bram Langeveld	4567102
Tyra Rahan	4448499

01-07-2022

TUDelft Delft University of Technology

Challenge the future

Multidisciplinary project group MP327

Preliminary design of a flood abatement control zone (ZAC) in the region of Murcia, Spain

Authors and student numbers:

Bram Langeveld, 4567102

Felix Francken, 4577256

Suryand Jhinkoe-Rai, 4300971

Tobias van Batenburg, 4578929

Tyra Rahan, 4448499

Supervisors:

Dr.ing. M.Z. Voorendt, Dr.ir. C.J. Sloff, Dr. J.T. Garcia Bermejo and Dr. J.M. Carrilo Sánchez

Figure S.1: Logos universities.

Date: 01-07-2022

Faculty of Civil Engineering and Geosciences, Delft University of Technology, and
Escuela Técnica Superior de Ingeniería de Caminos, Canales y Puertos y de Ingeniería de Minas,
Universidad Politécnica de Cartagena

Funded by: FAST university fund, International Internship Fund, X17 Group, Section Hydraulic Engineering of the Civil Engineering Faculty at the Delft University of Technology

Cover photo by J.G. Martínez (29-01-2020).

Figure S.2: Logos grant institutions.

Preface

This is the final report of the multi-disciplinary project of group MP327. It shows a preliminary design of a zone for flood-abatement control (ZAC) and its effect on reducing the flood region in the region of Murcia in Spain. This report was written to fulfill the requirements of the course CIE4061 Multidisciplinary Project - Civil Engineering Consultancy Project at the Delft University of Technology and in cooperation of the Universidad Politécnica de Cartagena.

The project was set up by contacting multiple civil engineering faculties throughout Europe for potential projects. The project of Universidad Politécnica de Cartagena was the most challenging and best suitable to express our skills in structural and hydraulic engineering.

Multidisciplinarity
The design of the ZAC structure is done by Hydraulic and Structural engineering students. The integration of the disciplines is achieved during the translation from the hydraulic HEC-RAS model in Chapter 5 through the functional design in Chapter 6 to the structural design in Chapter 7. The outputs of the flow-model are used as input for the structural model. The outputs of the flow-model are determined iteratively by checking their effects on the structural behaviour of the structural elements. Therefore, Chapter 6 is the most integrated part of hydraulic and structural engineering.

We would like to thank Dr. J.M. Carrillo Sánchez, Dr. J.T. Garcia Bermejo and Dr. A.V. Rodríguez from the Universidad Politécnica de Cartagena for the provided opportunity to study in Spain. Also, we thank our Dutch supervisors Dr.ing. M.Z. Voorendt and Dr.ir. C.J. Sloff for their support and feedback during this process. Furthermore, we would like to thank the FAST program of TU Delft, X17 Group - Engineering and recruiters, and the IIF TU Delft, who were willing to support us financially and partially facilitate the 60 day trip to Spain.

Group MP327

Tobias van Batenburg, Felix Francken, Suryand Jhinkoe-Rai, Bram Langeveld & Tyra Rahan

Summary

The coastal area of the Murcia region in Spain experiences more frequent and more intense flash floods caused by inland heavy rainfall due to climate change. This results in high water levels in towns and cities, leading to risk of loss of life and many financial damages. Currently the region is vulnerable, because there are no hydraulic structures that regulate the flow safely downstream towards the coastline. Instead, at this moment the current channels are inadequate dry rivers. To reduce the risk of flooding, the objective of this design was to design a flood abatement control zone (ZAC). This design followed the design approach for Hydraulic Engineering.

A ZAC consists of a series of dikes that enclose separate reservoirs that are able to temporarily store water. This dampens the peak discharge of the flash flood, which reduces the flood risk of the downstream area. The peak reduction is the main function. The water that is stored is being discharged with a delay, spreading an acceptable discharge over a longer time to discharge the same rainfall event volume.

The first step of the design was to define the system of the ZAC. The ZAC was combined with the creation of an up- and downstream channel with short lengths to fit the structure into the environment. Its design life was set at 50 years, corresponding to a design rainfall event of once per 474 years. To design this structure, it was important to determine the maximum rainfall event discharge and the existing maximum discharge capacity without floods occurring downstream. These 2 factors, together with soil properties and land boundaries, acted as boundary conditions to the system.

In step 2, two locations were considered as potential construction location, both indicated by the client Universidad Polytechnica de Cartagena (UPCT). By applying a multi-criteria analysis (MCA) based on predominantly the water inflow, potential storage area, close company buildings and houses, the more upstream location was chosen.

Step 3: in order to design an adequate ZAC, a model was required to create the before-and-after-construction situation. A hydrological 2D-flow model was created in HEC-RAS. This was closely tied to functional design and design steps were taken iteratively. The 2D-model was able to compute flow in longitudinal and lateral direction, which is strongly needed in a flooded terrain. The most important design parameters to test in the model were the culvert-spillway-structure and the number of reservoirs. The model was validated qualitatively by flood maps from Centro de Descargas del CNIG.

Then, the functional design of the ZAC was done in step 4. The ZAC was placed partially dug into the soil upstream, and partly sticking out of the soil downstream. Upon iteration, it was decided to create 5 reservoirs, because of costs and a smaller marginal peak reduction effect of extra reservoirs. The ZAC is created in combination with a downstream funnel, downstream outflow channel and upstream channel. This means that the reservoirs are enclosed by 2 side dikes and 6 lateral dikes.

Then, detailed design in step 5 followed. The culvert-spillway structure was designed. The aim of this structure is to let water through the reservoirs without overflowing and therefore damaging the dikes. The structure consists of a culvert, spillway and retaining walls. For each element an MCA was set-up to determine the optimal shape, followed by choosing the design alternative. The design is a trapezoidal spillway, an arched culvert and a retaining wall.

With the optimal design, a conceptual design is constructed. In this conceptual design, the reinforced concrete dimensions and governing load combinations are determined. From this, the strength of the ZAC structure is evaluated in the finite element method program DIANA.

Finally, the final design was created, as shown in Figure S.3. The conclusion is that the combination of structural elements that have been modelled in the system satisfies the aim to reduce the design rainfall event flood wave enough to avoid flood risk to the downstream areas of Murcia. The main uncertainties are the scaling of the data of the design rainfall event and the used soil characteristics. Further research could look into quantitative validation of the 2D-flow model, the implementation of sediment transport in the model and into optimizing bed protection downstream of the ZAC.

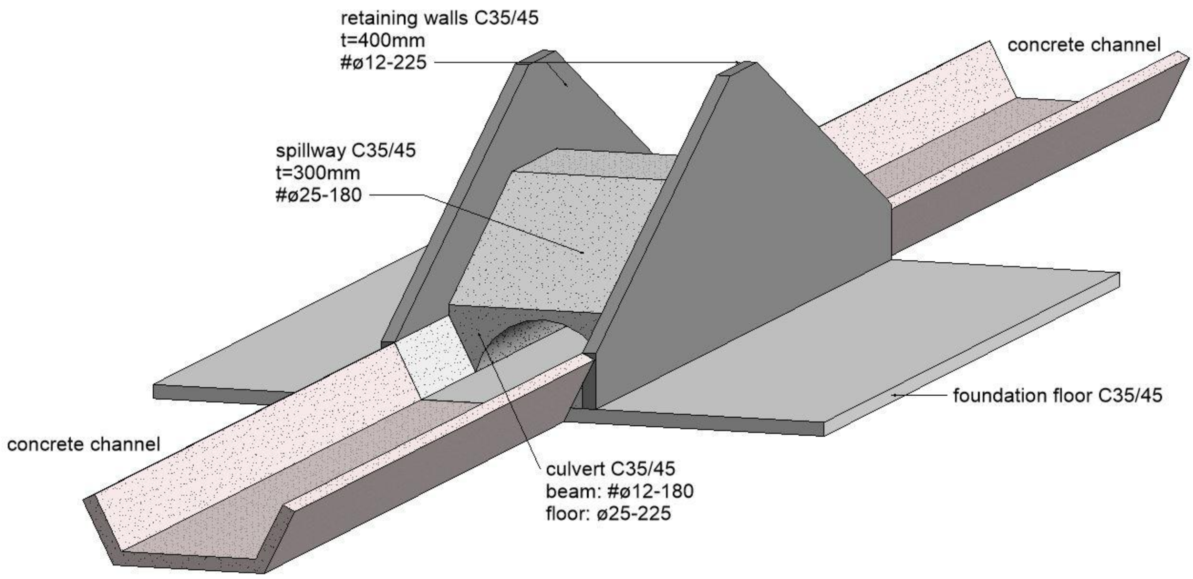


Figure S.3: 3D summary of structural requirements of the elements and the necessary reinforcement steel.

Contents

1 | Introduction

1.1 Project initiation

The Spanish national administration (Confederación Hidrográfica del Segura) responsible for watershed management asked the Universidad Politechnia de Cartagena (UPCT) to develop a master plan to propose and prioritise a series of actions on waterways and dry rivers to reduce the risk and vulnerability of the territory to flooding. The UPCT formed a group of hydraulic engineers that developed a list of approximately 200 solutions to solve the flood risk problem. The network of ZAC structures has been chosen as the main solution in this region. The UPCT has given this proposal to the national administration for water management, which is outsourcing different ZAC projects to engineering companies. This report aims to work out the solution of a ZAC at a location, and is used to improve understanding for the supervisors who worked on the large network project. The project is not being used for real world engineering application. Therefore, if 'the client' is mentioned, there is referred to supervisor J. T. G. Bermejo and supervisor J.M. Carrillo Sanchez.

1.2 Motivation for the project

The region of Murcia in Spain is prone to floods, where the last large flooding was in 2019 (Wade, 2019). Towns like Los Alcazares and San Javier experienced water damages to vehicles, floods in houses and excessive sediment deposition in the streets due to the sudden rise of the water level (Reardon, 2020). These heavy floods are caused by extreme weather events, originating from heavy rainfall in the region. Rainfall events have increased in frequency from once in 5 years to once in 3 years, and the event is given the name Gota Friá. It occurs in the months of September and October along the Mediterranean coast (Wade, 2019).

The Murcia region in Spain experiences floods during high discharge events, because the area is low and flat, see Figure 1.1. Through the past decades, the frequency of days with more than 40 mm of rainfall has increased from once every five years to once every three years (Avigueras Rodriguez, 2022). The area is prone to floods, because there is no regular water flow in many river banks: there are many dry rivers which only convey water during rainfall events.

The accumulation of rain water due to precipitation in the region must be discharged by dry rivers and creeks. These waterways are seasonally dry, and during the wet season they experience high water levels and discharge events. Subsequently, this causes substantial societal hindrance and damage to the affected areas. At the downstream end, the water flows to the lagoon Golfo Del Mar Menor, which is located next to the Mediterranean Sea.

The current river protections are not capable of protecting the hinterland against the high water events caused by these extreme discharges. Furthermore, the phenomena is amplified by the fact that climate

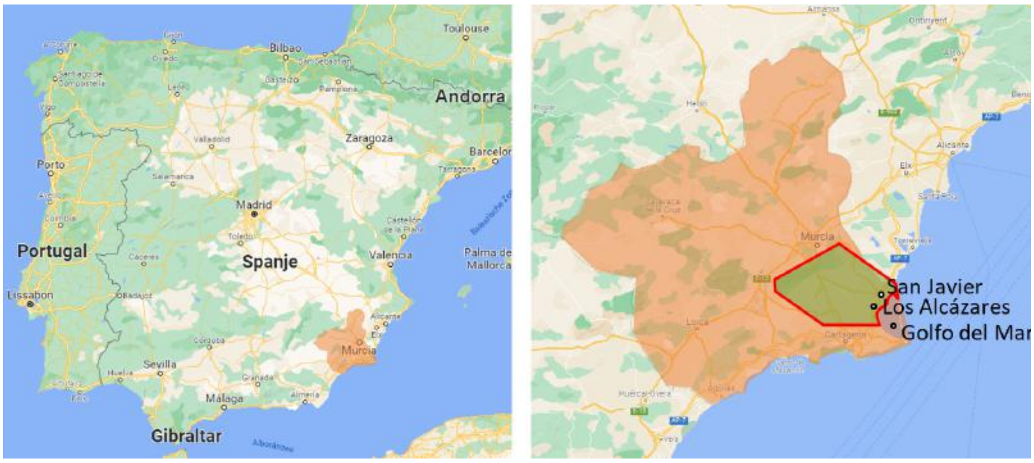


Figure 1.1: Location of Murcia in Spain (orange) and the area of interest (red line).

change will increase the number of extreme weather events in the future (Avigueras Rodríguez, 2022). This design aims to reduce the impact of floods in the region of Murcia.

To reduce the risk of future floods in cities, Bermejo (2022) proposed a network of so-called flood abatement control zone (ZAC) structures. The marked green area in Figure 1.1 is shown in detail at the left side of Figure 1.2. This report focuses on the design of a ZAC structure in the region encircled by the red dashed line. This flow area is a dry river reach that only conveys water during rainfall events. During a rainfall event there is a risk of flooding for the town Balsicas in the downstream area. This is shown at the right side of Figure 1.2. The figure also shows the current flood area for a once in a 100 year rainfall event in the current situation, so without any ZAC structure. From the downstream area, the water flows in direction of the coast towards San Javier and Los Alcazares.

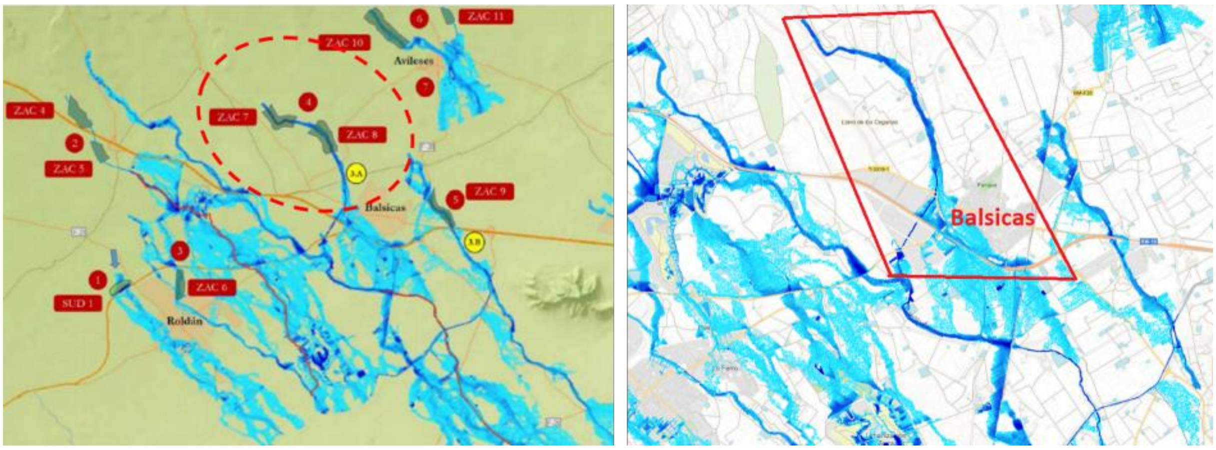


Figure 1.2: Left: Network of future ZAC structures in the Murcia region to reduce flood risk. Right: Dry river reach near Balsicas that is considered this design. Both marked areas indicate the same area (Centro de Descargas n.d.).

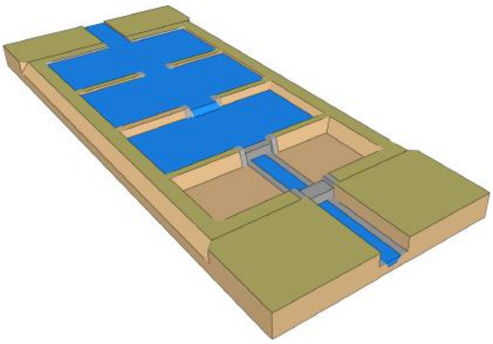
1.3 Problem statement

As stated in Section 1.2, the Balsicas region is prone to flooding and the current rivers are not capable of handling these extreme discharges. From this, the following specific problem statement can be formulated as the core of the project:

Currently, there is no hydraulic structure that reduces the peak discharges in the river during heavy rainfall in the Murcia region. This imposes a risk of floods in the downstream populated regions.

1.4 Project objective and focus

The objective of the project is to deliver a solution at conceptual level to reduce the impact of flooding in case of a high discharge event. For this project, the client formulated a solution to the problem. To reduce the peak flow, a ZAC has to be designed. A ZAC is a relatively short and artificially wider part of a river with a series of consecutive dikes, where each dike creates a separate reservoir, as shown in Figure 1.3 (Avigueras Rodriguez, 2022). In case of a peak discharge from a rainfall event, these reservoirs fill up in order to reduce the discharge downstream and are emptied over time when the incoming discharge reduces. It acts as a buffer zone upstream to reduce the burden downstream. This design aims to determine sufficient reduction of the peak flow discharge in case of an extreme discharge event and to design the dimensions of the dikes and spillways for the ZAC. Furthermore, the response of the river due to the implementation of this ZAC is modelled using HEC-RAS software. For this design sediment transport will not be taken into account due to time constraints.



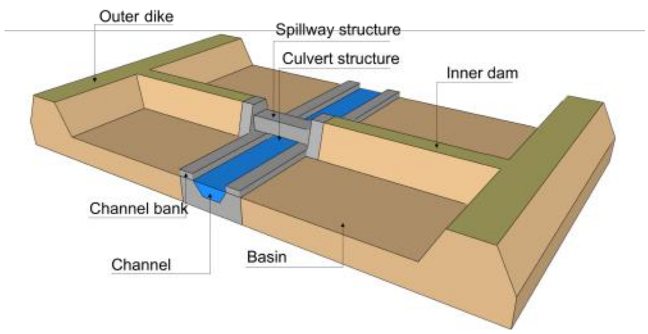


Figure 1.3: Left: Sketch of ZAC structure when three of four reservoirs are filled. Right: Structural elements of ZAC.

Despite the many possible solutions for the problem, the client asked to investigate the possibilities of a ZAC structure in the area of interest at the two given locations A & B. An overview of the location of the canals, cities and overall delta region are shown in Figure 1.1, 1.2 and 1.4. A photo of the dry river in location B is shown in Figure 1.5 The aim is to choose the optimal location (A or B) of the ZAC to lower the impact of the flooding on the amount of flood area in the downstream region. At the start of the project, the most optimal of these two locations, A or B, is selected to locate the ZAC.

From data provided by Bermejo (2022), it is observed that the frequency of extreme discharge events is slowly increasing. This is attributed to climate change. The design rainfall event with a return period of 474 years in 2022 will be smaller than the 474-year rainfall event at the end of the lifetime of the structure in 2072. Since the storage capacity of the ZAC stays constant during its lifetime, this means that the return period of the design rainfall event slowly decreases over the lifetime of the ZAC. Consequently, the probability of failure value in 2022 for the entire lifetime of the structure will increase by an unknown amount in 2072. Climate change and therefore this effect have not been included in the design.

Figure 1.4: Left: Suggested location A Right: suggested location B which is located downstream of Location A. The yellow arrow is the location and direction of the photo as displayed in Figure 1.5

Figure 1.5: Dry river location B.

1.5 Design approach and report outline

In order to deliver a design of the ZAC, a form of system engineering is used. A general version of the method can be found in the general lecture notes of Hydraulic Engineering 1 (Molenaar & Voorendt, 2020).

Exploration of the project
Chapter 2 focuses on the problem analysis of the project. This contains a hydraulic analysis of the ZAC

structure, a concise environmental analysis, a stakeholder analysis and a function analysis.

Project requirements and boundary conditions

Before the design of the structure, the project requirements are determined. One of the most important requirements is the amount of water the structure is expected to intercept. These parameters are determined by using the design rainfall event flow hydrograph, and they are used for the rest of the project duration. Also, the boundary conditions which the structure has to satisfy are indicated in Chapter 3.

Selection of the project location

Two locations are already marked as a potential location for the structure. These two potential locations can be found in Figure 1.4. The choice for the location depends on whether the location can effectively accommodate the structure, the environment, and where the structure has the most positive impact on the river system and its surroundings. This is the focus of Chapter 4.

Set-up and application of a 1D & 2D hydraulic flow model in HEC-RAS

A 1D flow model is used to determine the maximum discharge capacity of the dry river in the current situation without a ZAC. The 2D flow model is used to generate results of the impact of the ZAC structure, which is required for the functional design. This is used as input for structural design. This is described in Chapter 5.

Functional hydraulic design

This part of the design focuses on implementing the project requirements in a preliminary way and is discussed in Chapter 6. This makes it possible to quickly iterate with different parameters as to find the optimal solution for the structure. One can think about for example the material used to construct the dikes and spillways, the number of dikes used to make the ZAC as effective as possible and lastly to check how the ZAC affects the river and its surroundings. In the functional design, the steps are taken in order: conceptualisation of different alternatives, evaluation via multi-criteria analyses, selection of the alternatives and validation by the HEC-RAS model.

Structural design

The structural design in Chapter 7 focuses on making a more detailed design of the different design elements of the optimal solution provided in the functional design, including the critical load combinations. This focuses on the strength and stability of the different elements, but also the constructability has to be taken into account. The same steps are performed as in the functional design.

Presentation of the final design

Chapter 8 summarizes the functional and structural design. It shows the difference between the current situation and the situation with a ZAC. Furthermore, it discusses the size and source of uncertainties in the project and it gives recommendations on better execution of this design.

2 | Exploration of the project

As stated in Chapter 1, the main problem in the project area is the flood risk in Balsicas during a high water event due to heavy rainfall. In this chapter a stakeholder analysis, and process and function analyses are made in case a ZAC structure is implemented at the given locations A or B (see Figure 1.4).

2.1 Stakeholder analysis

Many stakeholders are affected by in the construction of a hydraulic structure, either directly or indirectly. In order to properly manage the various stakeholders, it is important to get a clear overview of all the stakeholders with their involvement and influence. Below, a list is given with all the stakeholders involved in this project.

- Local municipality Torre-Pacheco.
- Province state of Murcia.
- The cities and their municipalities downstream of the rivers the ZAC structure is located in, like Los Alcázares and San Javier.
- Local farmers and citizens located on the construction site of the ZAC structure. Farmers represented by the agricultural lobby.
- People living in the near vicinity of the structure.
- Nature organisations.
- Spanish national administration (Confederación Hidrográfica del Segura).

In Table 2.1 below, an overview of all the stakeholders, including their interests, influence and involvement is given. Their influence on and their involvement in the project will be rated from 1 to 5, where 1 indicates a low, and 5 indicates a high influence or involvement on the project.

Table 2.1: Overview of the stakeholders with their interests, influence and involvement.

Stakeholder	**Interests**	**Influence**	**Involvement**
Local municipality and the province state of Murcia	The structure will be located on their land. They are responsible to ensure the safety of citizens. The local municipality is also responsible for providing permits.	5	5
Los Alcázares and San Javier	These cities benefit the most from the construction of the ZAC, as it reduces the impact of heavy rainfall events in these cities.	5	5

Local farmers and citizens located on the construction site of the ZAC structure	These people must be bought out. The agricultural lobby is strongly against this, and from expert advise by Bermejo (2022) it is known that their influence is large. Therefore the number of bought out people must be limited. They will have to be offered a respectable offer and are against the plans for the structure.	4	4
People living in the near vicinity of the structure	They are mostly interested in their safety and want guaranties that chance of failure of the structure is acceptably low.	2	4
Nature organisations	Nature organisations focus on preserving the local ecology.	3	3
Confederación Hidrográfica del Segura	Confederación Hidrográfica del Segura are in favor of this project as it increases the water safety	4	2

The interests of the local farmers and citizens are partially against the interest of the project, as they would need to move their house or company in favour of reduced flood risk. Therefore, the decision of the location must minimise the number of people who are disturbed in their lives. When this is not possible, they must be financially compensated by buying their land.

Nature organisations could obstruct construction works if they know about rare species on the location of the ZAC. When design is more detailed and actual construction is more realistic, contractors are advised to get in contact with nature organisations in an early stage. Then, a study of the location could be conducted to see if endangered species live on either location A or B. This could be financed by the contractor. Details on this subject are out of the scope of the project, yet it is attempted to minimise environmental disturbance in this initial design, see Chapter 4.

Municipalities and authorities are involved by requesting construction permits and organising biweekly meetings by engineering firms, informing them about engineering decisions and opening discussions about potential changes in design.

The interests of the stakeholders are taken into account by assigning weight factors in the multi-criteria analyses in Chapter 4 in order to determine the optimal location of the ZAC structure.

2.2 Process and function analysis

2.2.1 Function analysis

The function analysis shows the main, preserving and additional functions of the ZAC structure and is used for the total design. All functions for this structures are mainly hydraulic functions due to the remoteness of the area. There are no nautical functions because the river is most of the time dry, which means there is no flow. Therefore, freshwater life is rare to not present and will not be taken into account. Below, the main functions of the ZAC structure are given.

Principal function:

- Lowering of peak discharge of the river during heavy rainfall to reduce water levels downstream by temporarily storing water in the reservoirs. This includes spreading the outgoing water volume over time.

Preserving function:

- Protect infrastructure, companies and households in the downstream area of the town Balsicas.

2.2.2 Process analysis

For the process analysis, an event tree is created. This event tree is given in Figure 2.1 on the next page, where all the effects of the processes are given. To present an even clearer picture of the processes of the ZAC-structure, a filling process of the ZAC is made to show all occurring steps during operation. This sequence can be found in Appendix C.

In the event tree, either there is no rain, regular rain or heavy rain. During no rain or regular rain the ZAC is not in use. During heavy rain, water fills up the ZAC. Then, the ZAC capacity can be exceeded, upon which an evacuation plan of the area should be started. If the ZAC capacity is sufficient, the water slowly gets discharged from the ZAC after the rainfall event. When the ZAC is empty, trapped sediment can be too much for the ZAC to provide the volume of water storage for the next rainfall event. Then, maintenance is applied by removing the deposited sediment. The ZAC must include a buffer volume for sediment, so maintenance is not required after every rainfall event, and also fulfills the structural requirement of maintenance once every 10 years as described in Section 3.1.3. Sediment trapping is not further considered in this design.
This event tree is used to define which situations can occur and what happens to the structure. One path is selected for the design process. The path is: heavy rain, high discharge - ZAC capacity not exceeded. Sediment transport is neglected, as mentioned in Section 1.5 From this, the event tree is used to define the functional requirements in Section 3.1.2.

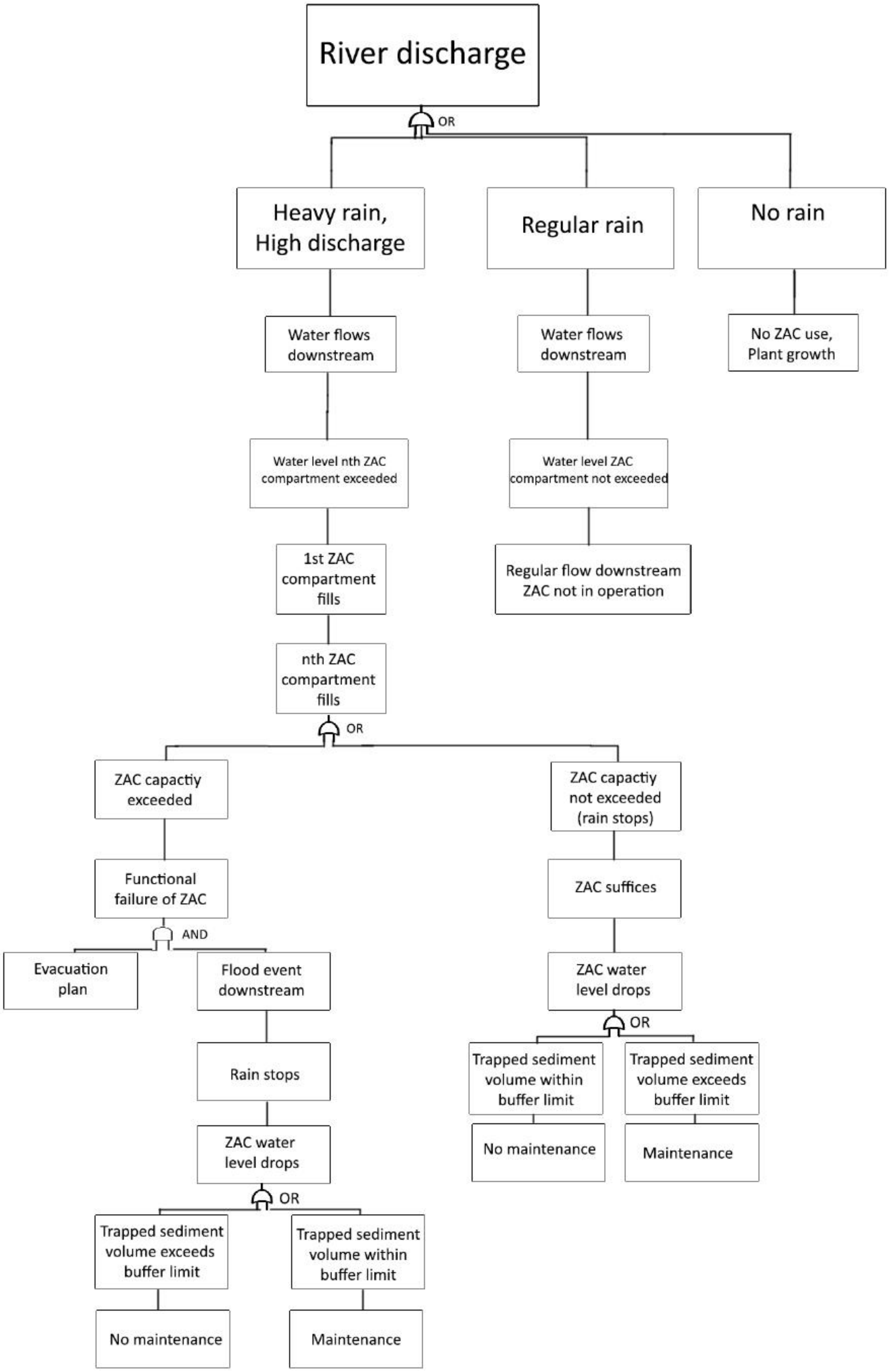


Figure 2.1: Event tree of the implementation of the ZAC.

3 | Basis of design

This chapter shows the start of the design of the ZAC structure. A list of requirements, boundary conditions and preferences are presented. These are the guidance during the design stage in Chapter 6 and 7.

3.1 Project requirements

The functional analysis stated the functions, which lead to different requirements for the ZAC to satisfy. These are categorised in location, functional and structural requirements. In consultation with the client the requirements are determined. Furthermore, the preferences of the project are determined in the same way.

3.1.1 Location requirements

Minimal environmental disturbances
The given area of the project has a dry climate, meaning extensive dry periods and therefore less vegetation. Therefore, the impact on the environment must be limited at the location of the to-be-built structure and must be integrated in the surrounding. Richly vegetated areas must be avoided when possible and a location with less plants has to be used. Furthermore, a ZAC structure could store water which could result in an enrichment of vegetation in the area.

Maximum effect on flood risk reduction
The impact of the ZAC structure regarding flood risk has to be as large as possible. As can be seen from Figure 1.4, the river in location A confluences in location B with a very small dry tributary. This means that potentially more water flows through location B than through location A. So based on this criterion alone, location B might have a larger impact on the reduction of flood risk in the downstream area.

Minimal disturbances for inhabitants at the project location
The impact of the ZAC on the people living in the near vicinity of the structure must be as low as possible. For both project costs and overall project support, it might be desirable to minimize the number of people that have to be bought out and the amount of farmland that has to be transformed to make place for the structure.

3.1.2 Functional requirements

Storage capacity
Being able to store the amount of water complementary with the design rainfall event is one of the structure's main requirements. It is important to understand the way the ZAC functions, regarding the amount of water the ZAC basins have to store during an extreme discharge event. For this design, it is chosen that the basins of the ZAC will only be used during extreme events. An extreme event is defined as the discharge being higher than the maximum discharge capacity at any location in the river downstream of the ZAC. The required amount of storage is the difference of maximum discharge capacity of the river and the actual maximum occurring discharge multiplied by the time the exceedance occurs. This is $3.3 * 10^6\ m^3$ and is indicated in the dashed area in Figure 3.3a. However, the storage capacity can be lower in case of additional measures downstream.

Inundation level
The structure must lower the impact of the floods in the downstream area. The acceptable inundation level is set to be 0.10 meter due to the drainage capabilities of the urban areas.

Structure life time and number of rainfall events
The structure has to fulfill its function for 50 years. This is given by PIANC (2016) guidelines for breakwaters and sea dikes. The link to this type of structures with a ZAC is that they are both flood protection structures, however breakwateters are exposed to sea, as opposed to inland ZACs that are prone to rivers. Since no information is found on inland reservoirs, it is assumed that the values from these guidelines can be extended to this project. This results in a design rainfall event with a return period of 1 in 474 years, based on an acceptable probability of failure of 10% during its life time. The determination of the probability of failure and the calculation for the return period is provided in Appendix A. In Appendix H it has been disproved that the occurrence of 2 consecutive rainfall events can be the determining factor for the storage capacity. In essence, for 2 rainfall events to have the same cumulative volume as the 474-year rainfall event, the return period reduces from once every 474 years to once every 66 years. The probability of 2 66-year rainfall events in 2 days has been calculated as $2.05 * 10^{-5}$, which is acceptably low, such that the once in 474-year rainfall event is the determining factor. This calculation has been performed under assumption of independent meteorological events.

No side overflow
To protect the properties in the near vicinity of the structure, no side overflow of the ZAC is allowed. This means that for functional failure (i.e., the amount of water is larger than the storage capacity), the water overflows downstream into the river, as shown in Appendix C.

Channel must not be clogged
For the ZAC to function properly, it is important that the channel, and more specifically, the culverts, do not get clogged with sediment or rubble, preventing water to properly flow through the channel. This can increase the filling speed of the ZAC, jeopardising the functionality of the structure.

3.1.3 Structural requirements

Stability and strength of the structure
The structure must be statically and dynamically stable in both normal and extreme circumstances. Furthermore, the structure must withstand all forces during the maximum discharges. Also, downstream of the structure, a scour hole might develop. This might negatively impact the stability of the structure. Therefore, a scour protection downstream of the structure might be necessary and should be considered in the design phase. Further, to ensure stability during large flow velocities, those must be limited at 4.8 m/s when the structure is made out of concrete. For greater velocities up special provision should be made but are limited to 6.0 m/s (CCPPA, 2022).

Constructability
The structure must be constructable with ready-to-use equipment. Construction equipment should not travel too large distances to arrive at the project location. This is because the project location is quite remote and no major transport routes have good access to the project location.

Maintainability
The project area is located in a remote area and is hard to reach. Therefore, the ZAC structure must be designed in such a way that there will be a low amount of maintenance needed.

3.2 Preferences

No downstream riverbed changes
The current downstream river is going through agricultural areas and is at multiple locations used as an orchard. A picture of this location is shown in Figure 3.1. Changes of the river geometry must be designed carefully and are not preferable, due to the potential impact to the agriculture. The agricultural lobby has strong influence on changes in the region, and therefore the least disturbances are preferred.

Expandability
The approximate area of the structure is given and results in a potential limit capacity of the structure. Although, the structural dimensions are not yet determined, one could think of an extra added basin in the future. Therefore, area in front, behind or at the sides of the ZAC could have the option to be excavated and to be used as an extra reservoir to increase the capacity of the ZAC.

Water storage
In region of the ZAC there are dry periods and large agricultural areas. Therefore, access to water supply is important. Thus, it would be useful that the ZAC structure could be used as a water storage for the surrounding farmlands after an extreme discharge event.

Locations for plants and trees
Various plants and trees might have a positive effect on the aesthetics of the structure and its location and it will also implement the Building with Nature philosophy. Bringing some artificial nature to the area can be beneficial to the biodiversity in the ZAC.

Construction materials
To reduce transport times and to lower the emissions, it is desirable to use as many of the construction materials from manufacturers in the region of the project and use local soil as a construction material.

Figure 3.1: Trees are located on the dry river bed in an orchard.

3.3 Boundary conditions

Boundary conditions are site specific parameters given by the terrain and surroundings of the project location. A list of boundary conditions is given below.

Maximum upstream discharge
The maximum upstream discharge is given by a hydrograph complementary with the return period of the design rainfall event. The maximum discharges for various return periods are provided by the client. From Appendix A it follows that the return period of the design rainfall event for this specific ZAC structure is equal to 474 years, with a structure life time of 50 years. By means of interpolation, the upstream hydrograph can be calculated and is displayed in Figure 3.2. As can be seen, the maximum discharge at the peak of the flood wave is equal to 207 m^3.

Maximum discharge capacity
The weakest link in the dry river is defined as the point with the critical cross-section of the river that can handle the extreme discharge without overflowing (i.e., the maximum capacity of the downstream river). In Section 5.4 the 1D HEC-RAS model is used to determine this value. This discharge is equal to 51 m^3/s. From this it follows that the maximum allowable outflow of the last culvert of the ZAC is equal to 51 m^3/s as well, on the condition that no changes are made to the downstream channel. The rainfall event that has this discharge as peak discharge has a return period $R = 24\,years$, as determined by the hydrograph scaling method in Appendix B. This means that there is no overflow of the river in this situation. So during an extreme event, the river downstream of the ZAC will maintain a maximum discharge of $51\,m^3/s$ while the excess actual discharge must be stored in the reservoirs of the ZAC. Only in case of extreme events where the last spillway is used, the maximum outflow discharge increases, and a downstream flood might happen. To reduce this, additional measures to increase the capacity of the weakest link have to be applied. The total volume of the reservoirs, which is the storage capacity of the ZAC, can be calculated as the total volume of water under the hydrograph minus the total outflow

volume of the ZAC during the same amount of time. For clarity, this is also displayed in Figure 3.2.

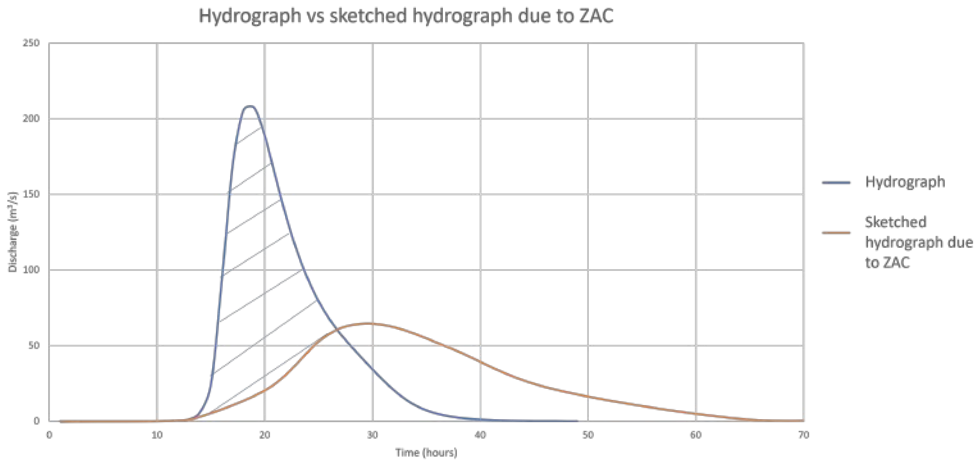


Figure 3.2: Hydrograph in current situation (blue) and sketched hydrograph by implementation of a ZAC structure. Key aspects are elongation of the curve, as well as flattening of the curve and the delay of the peak discharge. The total volume capacity of the reservoir is the difference between the current hydrograph and the new hydrograph of the ZAC (arched in blue).

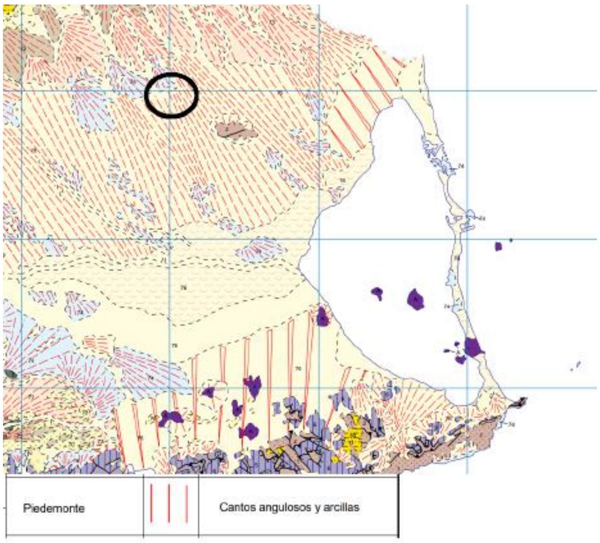
Soil type and groundwater level

Soil data directly from the project location A and B is not present. Therefore, rough data about the soil in the Murcia region is used and is shown in the figures below. The first one in Figure 3.3a (Instituto Geológico y Minero de España, n.d.), indicates the region of the ZAC in the black circle. The type of soil is illustrated by red stripes, which indicates pebbles and clay. This is supported by Wright and Wilson (1979), which give a estimate of the distribution of soil types in the soil in Murcia. The used soil type is sand - greatly silty clayey.

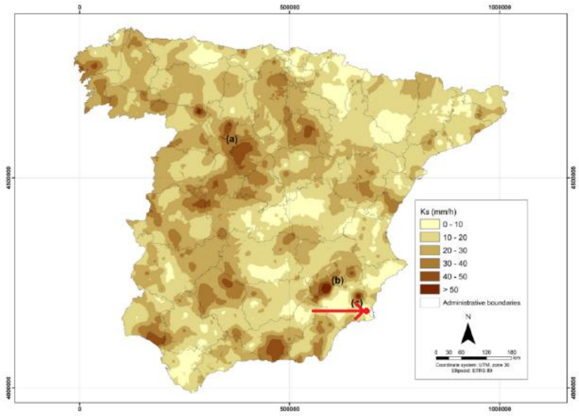
Table 3.1: Distribution of soil types in sand - greatly silty clayey from Murcia in Spain. Values taken from Wright and Wilson (1979).

Soil type	Range [%]
Sand	35-50
Clay	30-45
Silt	10-30

In Figure 3.3b (Jiménez-Martínez et al (2012)), the soil hydraulic conductivity of Spain is shown and at the ZAC between 0 to 10 mm/h. Furthermore, this soil type at the project location has a very low capacity to store water (Sierra et al, 1992) and by interpolating the available data of groundwater measuring stations, which are shown in Appendix F, it can be concluded that the groundwater is approximately 68.8 meters below the project location and can be neglected for this project. A photograph of the local soil is shown in Figure 3.4.



(a) Soil map from the Instituto Geológico y Minero de España



(b) Hydraulic conductivity map from Jiménez-Martínez et al (2012). At the ZAC location, indicated by the red arrow, the hydraulic conductivity k is in the range of $0 - 10\ mm/h$.

Figure 3.3: Soil maps of the Murcia region. The black circle and red arrow indicate the project location.

Figure 3.4: Photo of the local soil on location B.

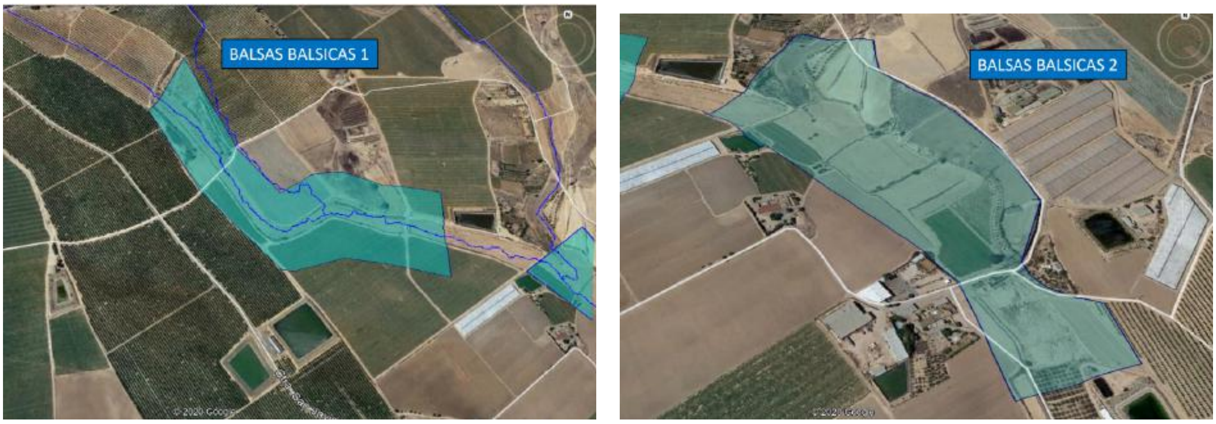
Land boundaries

Figure 1.4 gives the initial proposal of the project boundaries. This is an indication, as the structure does not exactly take this shape as a final design. For location A, the borders are neighboured by farmlands only. At the downstream end, there is a small water reservoir nearby, managed by a farmer. It is undesirable to place the ZAC at the location of the reservoir, as it would cause unnecessary buy-out costs. The upstream area of location 1 is suitable to place the ZAC in terms of available space.

For location B, the ZAC has to narrow a bit in the middle, because there is a farm complex on the west side and a warehouse on the northeast side. Furthermore, there is a small farm on the northwest side of the western farm complex. To avoid buying out these companies, the shape of the ZAC has to be constricted. In this location, there are also multiple water reservoirs in between which the ZAC must be placed.

4 | Selection of the project location

The structure can be created in 2 locations, as given by the client. Location A and B can be distinguished on multiple aspects. Figure 4.1 shows the project locations. A multi-criteria analysis (MCA) is performed, which is used to quantitatively choose a location for the ZAC structure.



(a) Overview of location A

(b) Overview of location B

Figure 4.1: Overview of the project locations.

The MCA is done by going through the following seven steps and the results for location A and B are collected in Table 4.1 and 4.2 on page 20 respectfully:

1. Criteria are divided into main categories.
The main criteria are taken from the project requirements. The main criteria are: impact on local environment, impact on society and water storage. This is the first column of Table 4.1

2. Each criterion has been assigned a weight factor.
The sum of the weight factors (A) of the criteria is 1.0. The influence of the water inflow and the potential storage area are considered the largest (0.6), as the incoming design rainfall event volume is the main cause for the design of the ZAC. The interests of the local population and companies is then considered most important (0.3), because of the impact of the ZAC on the lives of local population. It is not estimated higher, because no houses are built directly in the areas. However, with regard to potential future expansion the interest of people living within a 200 meter reach are still considered. After a first area assessment using satellite images, it was concluded that there are no endangered species, and limited numbers of trees and plants, leading to the conclusion of an environmental weight factor of 0.1. An extensive research by a nature organization could change the weight factors of this multi-criteria analysis, but this is used for a first estimate. This is the second column of Table 4.1.

3. Criteria are subdivided into subcriteria, which all have their own weight factor.
The same approach holds for sub-criteria, where the sub-criteria weight factors (B) summed amount to 1.0. The subcriteria are:

- *Loss of farmland area:* The area of used farmland is measured for both locations using Google Earth. Then, the relative contribution of each location is calculated, giving the lower score to the largest in-use farmland location. This is because more land needs to be bought, which negatively affects the MCA-score. This makes location A less favourable, scoring 45.7, whereas B scores 54.3.
- *Number of removed trees:* The number of trees in each location is estimated using satellite imagery from Google Earth. When more trees need to be removed, it leads to a lower score for the location.
- *Area percentage of plants:* From the same imagery, the percentage of area that is filled with plants on the unused land is estimated for both locations.
- *Number of company buildings (reach 200m):* The number of company buildings within a reach of 200 meter of the project area are counted for both locations, and calculated the same way as farmland.
- *Number of houses (reach 200m):* See above.
- *Existing water storage area:* The cumulative area of all water storages is measured, which indicates how much of this type of property needs to be removed to make space for the ZAC. Calculation: see above.
- *Road-household-kilometers* The number of houses that are affected by either location A or B are counted. Then, the number of kilometers that they need to drive extra to get home, assuming they come from the main roads, are measured. These values are multiplied and summed for each house, so the total house-reroute-kilometers can be compared.
- *Inflow volume of water:* The inflow in the locations can be different, as location B can also experience inflow from the eastern branch. Since no data is available on the potential discharge of this branch, an assumption is made on the maximum value. From Avigueras Rodríguez (2022), shown in Figure 4.2, the red dashed circle includes ZAC 7 and ZAC 8, which are locations A and B respectively. Since this map does not even show a branch on the east between ZAC 7 and 8, it is assumed that the branch contributes to maximum 10% of the inflow at the main branch. This leads to a favourable score at location B, because the total inflow there could be larger. Thus, increasing the effectiveness of the ZAC.
- *Potential storage area:* The project location of A has a larger area ($250,000m^2$) than location B ($180,000m^2$). In the future, this could be a bottleneck for possible expansion, which gives location A an advantage over location B. The same calculation as farmland is applied.

4. The relative weight factor C is calculated by multiplying the main criterion weight factor with the subcriterion weight factor.
The relative weight weight factor $C = B * A$.

5. Each subcriterion has been assigned a score V_c.
V_c for farmland location A is calculated by looking at what percentage of the total to-be-bought farmland is in location A. In location A $113,000\ m^2$ of land must be bought, in location B this is $95,000\ m^2$. Location A is less favourable to choose regarding this aspect, so it must have a lower score than location B. The formula used is:

$$V_{c,A_i} = 100 * \frac{ValueB_i}{ValueA_i + ValueB_i} \tag{4.1}$$

Where:

- $ValueA_i = 113,000m^2$
- $ValueB_i = 95,000m^2$
- 100 to get scores between 0 and 100.

This leads to $V_{c,A_{farmland}} = 45,7$. The same way, $V_{c,B_{farmland}} = 54,3$. Both values are each other's complement and amount to a score of 100. The value of location A is lower than B.

6. The relative score D per subcriterion is calculatd by including the weight factor of each item.
The relative score is $D = C * V_{c_i}$.

7. Finally, the scores of each subcriterion are summed and they form the final score of location A or B.
The relative scores are summed, leading to a total score for each location. Both locations score relative to each other, so the sum of both scores is 100. When this procedure is followed for all sub-criteria, the score of location A is 57.4 and location B is 42.6. Location A is the complement of location B and has the highest score, meaning location A is the best suitable location for the construction of the ZAC.

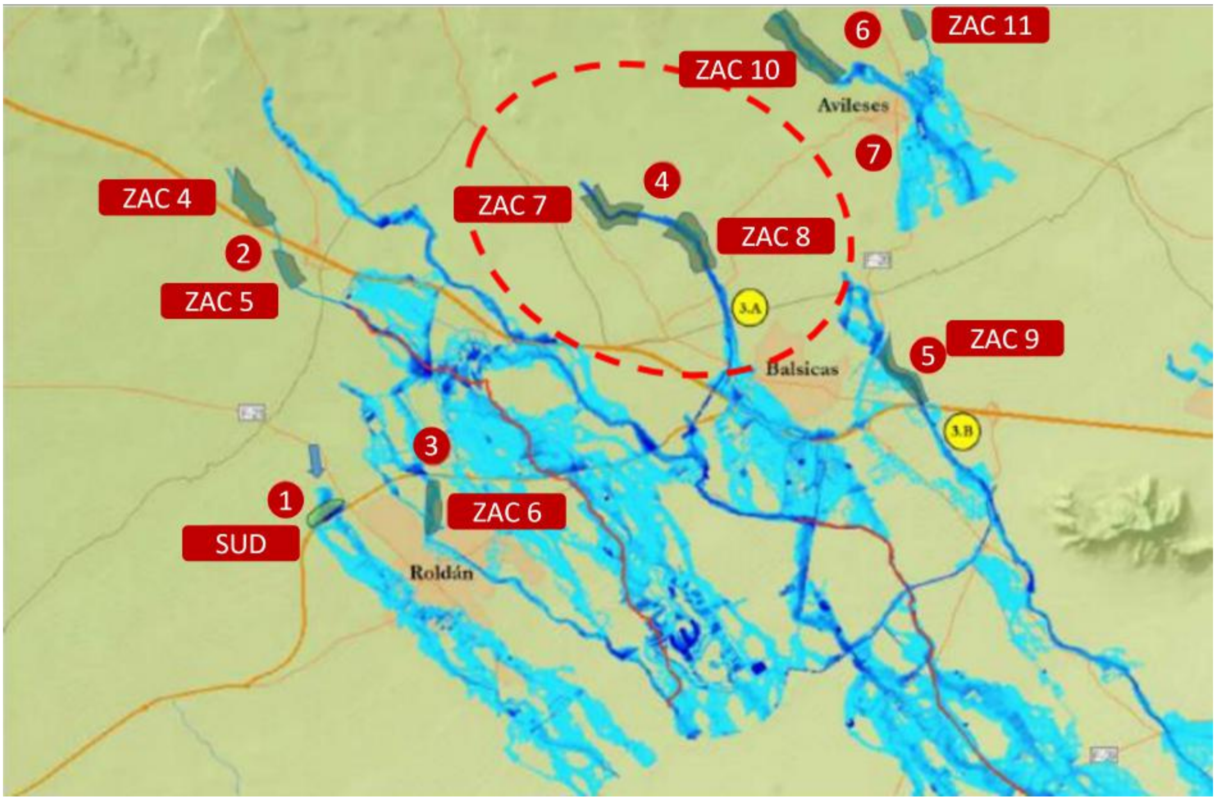


Figure 4.2: Overview ZAC system. Between ZAC 7 and ZAC 8 there is no branch indicated, leading to the assumption of a maximum inflow of 10% of the maximum inflow by the main dry river.

Main Criterion (A)	Weight Factor A	Sub-criterion (B)	WF B	Relative weight (C=A*B)	Score Vc	Relative Score (D=C*Vc)
Environmental	0.1	a. Farmland	0.8	0.08	45.67	3.65
		b. Trees	0.15	0.015	75.22	1.13
		c. Plants	0.05	0.005	52.47	0.26
Societal	0.3	a. Close Work buildings (200m reach)	0.3	0.09	82.86	7.46
		b. Close Houses (200 m reach)	0.45	0.135	81.25	10.97
		c. Storages	0.15	0.045	44.55	2.00
		d. Roads	0.1	0.03	26.31	0.78
Water	0.6	a. Inflow	0.6	0.36	47.61	17.14
		b. Potential storage area	0.4	0.24	58.13	13.95
					Total	**57.36**

Table 4.1: Multi criteria analysis of location A. The total score of A and B summed is 100 for every value in the Vc column. The total score of 57.4 means location A is preferable over location B.

Main Criterion (A)	Weight Factor A	Sub-criterion (B)	WF B	Relative weight (C=A*B)	Score Vc	Relative Score (D=C*Vc)
Environmental	0.1	a. Farmland	0.8	0.08	54.32	4.34
		b. Trees	0.15	0.015	24.77	0.37
		c. Plants	0.05	0.005	47.53	0.23
Societal	0.3	a. Work buildings (200m reach)	0.3	0.09	17.14	1.54
		b. Houses (200m reach)	0.45	0.135	18.75	2.53
		c. Storages	0.15	0.045	55.45	2.49
		d. Roads	0.1	0.03	73.68	2.21
Water	0.6	a. Inflow	0.6	0.36	52.38	18.85
		b. Potential storage area	0.4	0.24	41.86	10.04
					Total	**42.63**

Table 4.2: Multi criteria analysis of location B. The total score of A and B summed is 100 for every value in the Vc column. The total score of 42.6 means location B is less preferable than location B.

5 | Set-up of a 1D & 2D hydraulic flow model and its application

A computer model of the river is needed of the project locations with and without the ZAC structure. First, the river is modelled without the structure to check the initial river flow. Thereafter, the ZAC structure is implemented and its dimensions are optimized to aim for the lowest flood impact downstream. To model the river in the design area, HEC-RAS software is used.

5.1 Theory used in computational software HEC-RAS

1D computational method

The two following laws of physics form the basis of the 1D model computations: the principle of conservation of mass (continuity equation) and the principle of conservation of momentum (momentum equation), (CivilGEO Inc., 2022). These equations are then mathematically represented by partial differential equations. As follows from the HEC-RAS unsteady computational set-up menu, the 1D numerical solution is calculated using the Skyline/Gaussian finite difference method. This is the default computational method for 1D.

2D computational method

There are two possible computational methods that can be used for the 2D model: the diffusion wave method (which is faster, but is less widely applicable) and the 2D shallow water equations (which is a more time consuming method, but increases the accuracy of the computation under more specific cases). For this project, the diffusion wave method is used as this is the fastest method, and only a limited amount of computational power was available. In section 8.4 it is explained why the choice of the 2D diffusion wave method might not be ideal for the modeling of a ZAC structure.

5.2 Setting up HEC-RAS in 1D

The 1D HEC-RAS model has been setup in 7 steps with as goal to create a flow depth map of the flood area during a 474-year rainfall event. These are described in detail in Appendix G. The key aspects of these steps are:

- Local terrain near Balsicas is loaded into the model, and the correct projection, land cover type layer and according Manning value coefficients are assigned to the locations. Manning values from different land cover types are taken from the US Coastal Change Analysis Program (C-CAP) (2016).
- The flow path, the dry river banks and cross-sections are drawn to represent the river. HEC-RAS interpolates flow between cross-sections. Choosing correct spacing of cross-sections is important

to get reliable results. The model gave unrealistic flood maps for a spacing of 200m, and it did not give noticeable more accurate depths for a spacing smaller than 80m. The trade-off of a smaller spatial step is larger computation time. Therefore, the spacing has been set at 80-100m, varying in straight channels and curves.

- At the upstream boundary an unsteady flow hydrograph is imposed to represent the 474-year design rainfall event that flows into the system. This flow is given by Bermejo (2022) 3.3a and was adapted with a base flow of 10 m^3/s to avoid numerical errors. The need for a base flow follows from the 1D solver that HEC-RAS uses to compute the flow. The flow characteristics are such that the Froude number is smaller than 1, so it was concluded that subcritical flow is present. Therefore, an up- and downstream boundary condition needed to be imposed. This was done by implementing the normal depth downstream. Even though there is no normal depth downstream due to the unsteady flow, it is considered acceptable, because the downstream boundary condition has been imposed a few kilometer downstream of the area of interest. The numerical error gets smaller the further upstream you go from the downstream end and it is sufficiently small according to expert judgement from J.M. Carrillo-Sanchéz gives a good indication.
- The time interval for the computations has the same consideration as spacing. Too small timesteps of 1 sec yield no significant extra detail, while too large timesteps of 5 min created errors in the resulting flow depth maps. Therefore, the time step was chosen at 1 min.

5.3 Limitations of the 1D model

After the setup an attempt was made to recreate the flood area as depicted in Figure 5.1. This flood map is the map of a once in 500-year rainfall event as given by Centro de Descargas - the Spanish national institute of geographical information - which is a near perfect resemblance of the once in 474-year flood map. This recreation was very complex to do in a 1D model, because of the following reasons:

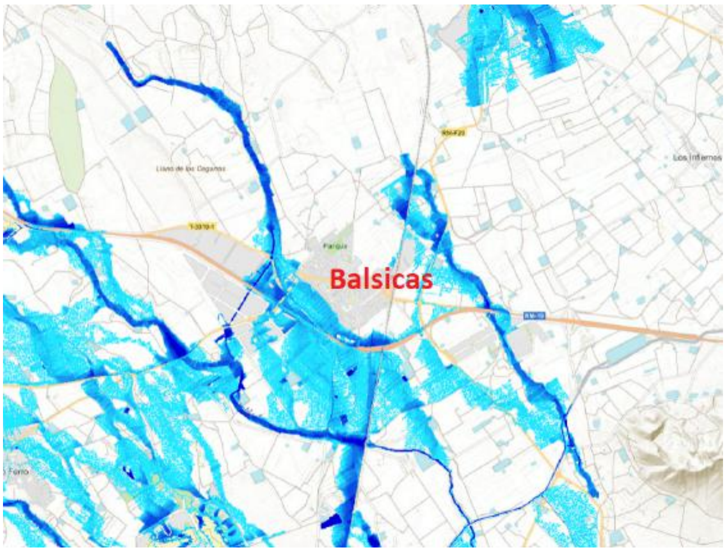


Figure 5.1: Dry river reach near Balsicas that is the reference flood area of the result of the 1D model.

- The model was setup in such a way that the water getting out of the banks did not have their own flow path branches. The flow path bathymetry, as shown in Figure 5.2 is lower outside the river banks. This means that when water gets out of the banks during a flood, it starts to find another flow path toward the coast. This is through the town of Balsicas. This indicates that the existing dry river channel location is ill-designed. The 2D model also shows more clearly where the water starts coming out of the branch (see Figure 5.4 in the next section), so this is where the river branch would have to be implemented. These branches have not been applied however, because the length of the modelled reach is sufficiently small to apply a 2D model instead. Besides, the application of the 2D model turned out to be more straightforward. Therefore, the flood maps that got out of the river banks indicated that the lower-laying floodplains would fill up completely, as HEC-RAS aims to satisfy the continuity equation.

- The problem of water getting out of the banks in the model can be avoided if at the upstream location a river bifurcation is implemented, which leads water through the city. This is done by applying a branch in the HEC-RAS flow path. However, there is no information available on the division of water per reach and the flow path is unknown in this situation too. Therefore, it was much more difficult to apply a 1D model compared to a 2D model.

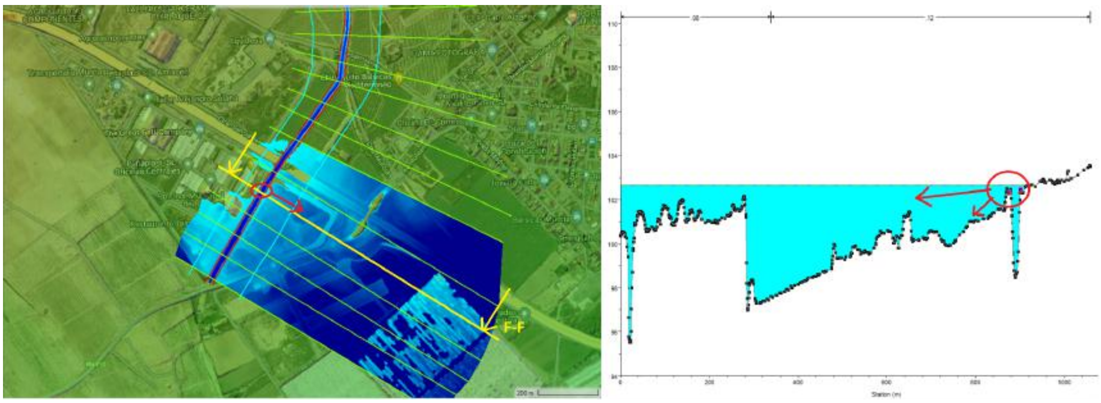


Figure 5.2: Flooding is instantaneous. This figure shows what happens in the 1D model if the water level exceeds the banks. The left figure locates the cross-section F-F, which is pointing downward. The cross-section is shown on the right. The right part of the flood in the left figure is displayed by the left part of the cross-section of the right picture, so it is flipped horizontally. Outside the red circle in both figures the complete lower lying floodplain area is instantaneously filled. This indicates that a branch must be applied at the location where water starts flooding the area to have a correct 1D model.

5.4 Application of the 1D model

The 1D model still proves useful for the determination of the maximum discharge capacity of the river. In Appendix B this is worked out in detail. The maximum discharge capacity is found by iteratively scaling the flow hydrograph to a new maximum and checking in the flow depth map whether the resulting water level from this flow hydrograph stays between the banks of the current bathymetry. This means that the water stays within the applied flow path and the existing channel can be used for the model. The maximum discharge capacity of the current downstream geometry turned out to be $51\ m^3/s$ at the location as indicated by Figure 5.3. In this figure no overflow occurs. The bottle neck is the transition point from an orange orchard to the channel, see the right part of the figure. Not all water flows into the channel if the discharge is larger than $51\ m^3/s$. The accompanying flow depth map is shown on the left in Figure 5.3. The capacity of the channel downstream of this point is larger.

Figure 5.3: Left: Depth map resulting from a scaled flow hydrograph with a maximum of $51\ m3/s$. No overflow occurs, which means it determines the maximum capacity of the dry river, if no additional measures are taken to increase the capacity. The flow direction is from the top to bottom of the figure. Middle: satellite image of the terrain, zoomed. The terrain shows that there is an abrupt transition between the farmland and the start of a channel. Right: photo of the river bed taken at upper point of the middle figure, pointed southward.

5.5 Setting up the 2D model

Next to the 1D model of the river response, a 2D model is set up. This is done to validate the two results of the models. The main difference between the two types of model is the calculation method. For the 1D approach, an interpolation between each cross-section is made, whereas the 2D model uses a computational grid for its result. The interpolation between different cross-sections along the river reach is limiting, because in the flood area water can travel orthogonal to the river reach, by getting out of the banks.

A 2D-model has the following advantages over a 1D model:

- No base flow is required for the upstream boundary condition to get a working numerical model. Therefore, the actual occurring flow can be used in an unaltered manner, leading to more representative boundary conditions.
- The flood flows take different paths along the flood plains and inundate sideways.

The goal of the 2D model is to determine and verify the dimensions of the various parts that make up the ZAC structure. Designing in the 2D model goes hand in hand with the functional design in Chapter 6, creating an iterative process. The most important parameters to design and test in the model are: the culvert-spillway structure and the number of reservoirs. Also, a reference model without a ZAC structure is modelled to compare the effect of the ZAC with the initial conditions. Figure 5.4 shows the result of the initial situation without a ZAC.

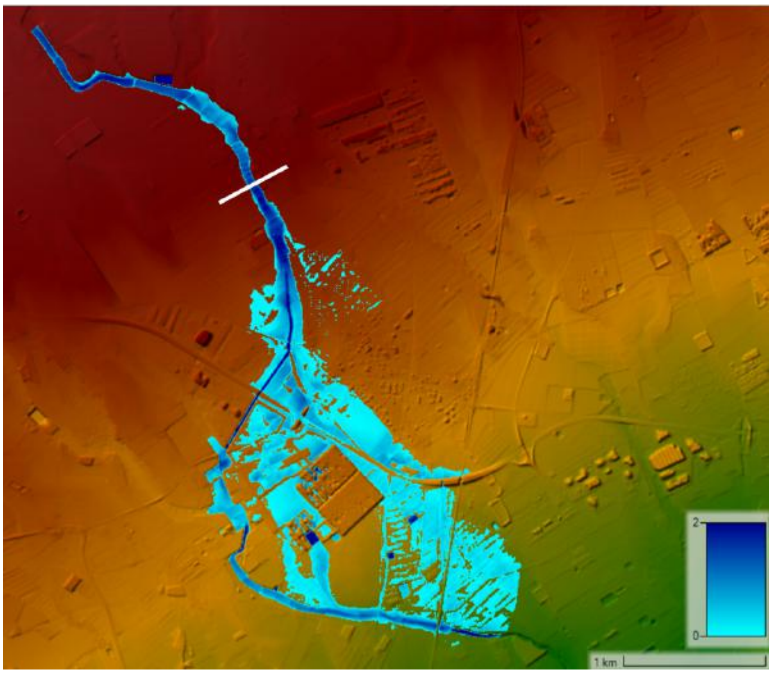


Figure 5.4: Result of a simulation without a ZAC structure. After the white line the river starts to overflow. This is also the location from which the hydrograph is taken. Downstream, the river gets out of the banks and shows a flood area in the city of Balsicas. The light blue values are depths close to $0.0\ m$. and dark blue values close to $2.0\ m$.

The designing and testing for these parameters is done in a structured manner, starting with the parameters with the largest influence on the reduction of the peak discharge. This is the number of reservoirs, because they determine the size of the storage capacity. The effect of the number of reservoirs is quantified by comparing the hydrographs just downstream of the ZAC. Figure 5.5 shows the result of these simulations. Every simulation, an extra reservoir is added to the ZAC, increasing its footprint and storage capacity.

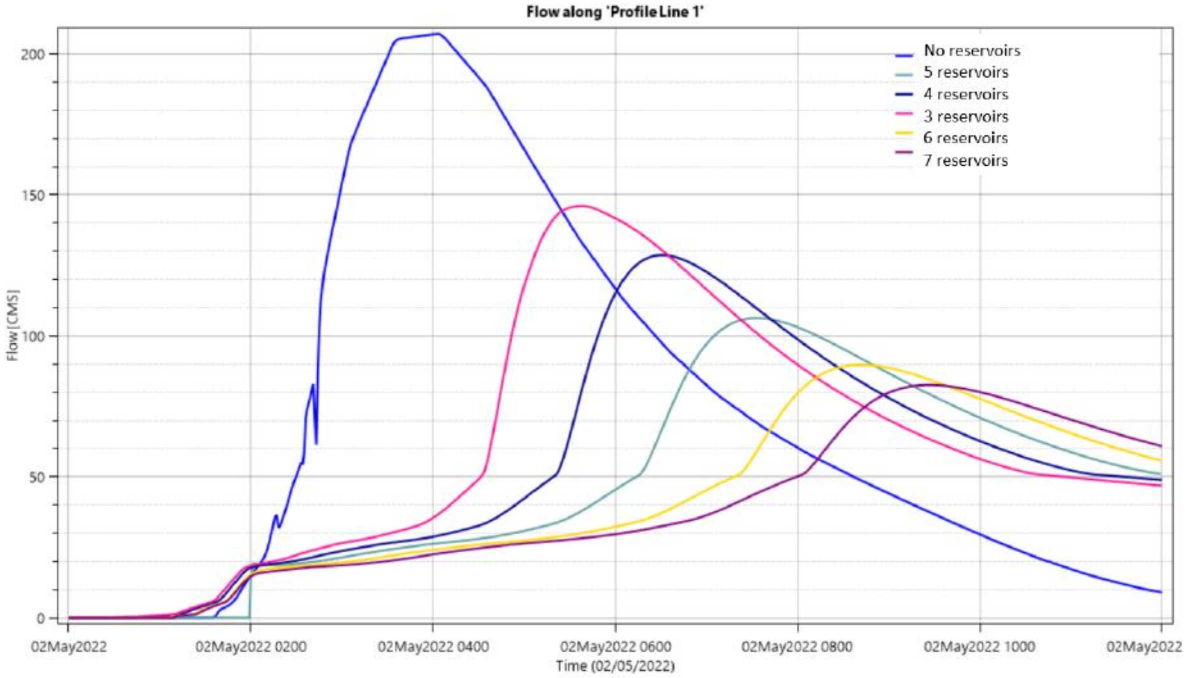
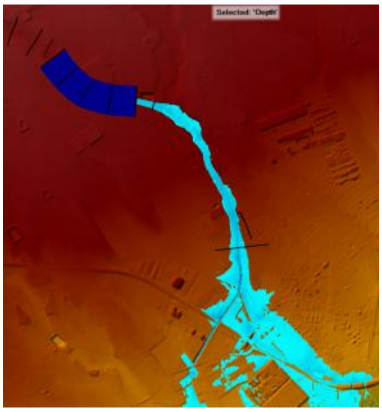
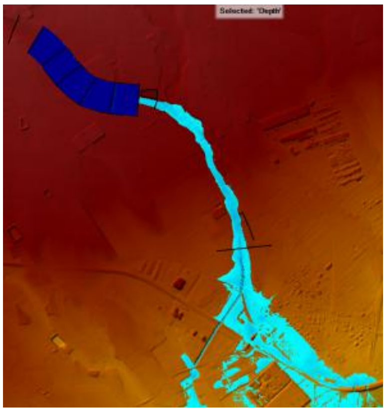


Figure 5.5: This figure shows the outcome of the simulations that shows the effect of adding extra reservoirs.

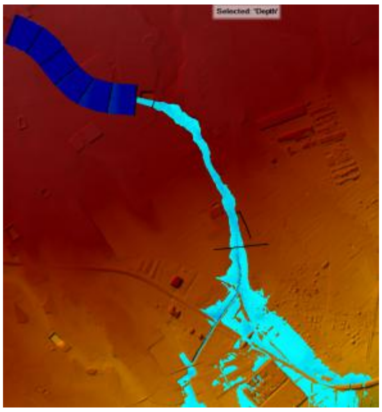
Another option to compare the effect an extra reservoir has on the peak discharge reduction, is by means of flood maps. Figure 5.6 shows the flood maps for four, five and six reservoirs.



(a) Situation: four reservoirs without a downstream channel. The flooded area is covered by roughly 70 cm of water.



(b) Situation: five reservoirs without a downstream channel. The flooded area is covered by roughly 50 cm of water.



(c) Situation: six reservoirs without a downstream channel. The flooded area is covered by roughly 40 cm of water.

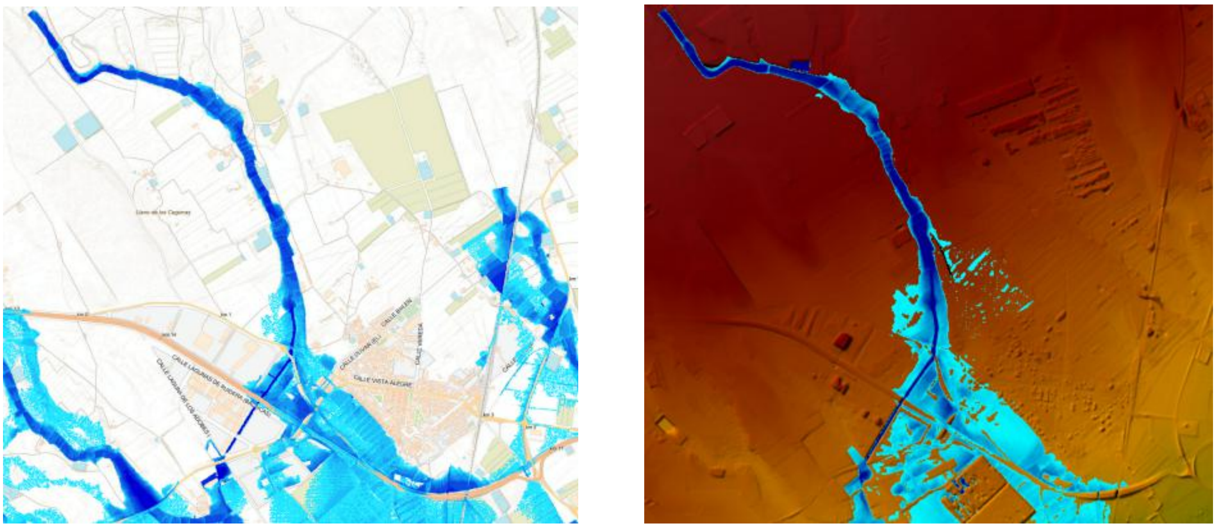
Figure 5.6: Flood map of the area downstream of the ZAC at the time the flood is maximum for each situation.

Note that assessing the effect of an extra reservoir by means of a flood map is less quantitative than using a hydrograph and is better used as visual impression.

Further modelling was done on the culvert spillway structure. By first leaving out the spillway entirely, the culvert was modelled. This way, the entire dike was able to overflow and thus making it possible to read the maximum capacity of the culvert from the hydrograph. The spillway was later added to control and concentrate the overflow of the dike through a smaller and armoured part.

5.6 Validation of the 2D model

Validating the model is an important step to check whether the output of the model is reliable and can be used for solving the problem. Validating the model is done qualitatively using the maps of the Centro de Descargas del CNIG (Centro de Descargas del CNIG (IGN), n.d.). These are flood maps of the same area as the project location. Figure 5.7 shows the flood map created by HEC-RAS and the flood map from Centro de Descargas del CNIG.



(a) Flood map from Centro de Descargas del CNIG

(b) Flood map from HEC-RAS

Figure 5.7: Qualitative validation of the model

The flood maps from the HEC-RAS model and from Centro de Descargas del CNIG show a similar result. From this it can carefully be concluded that the 2D HEC-RAS model is reliable enough to start to make alterations and to implement a ZAC structure in the model. It should be noted that the flood map from Centro de Descargas del CNIG is the only source that allows for the comparison of results, resulting in a rough comparison between the two models. Due to the lack of data, it is difficult to refine the validation of the model.

The difference between the two models is mostly due to extra input from other rivers in the case of the model from Centro de Descargas del CNIG.

6 | Functional design

The design choices for the ZAC structure are explained in this chapter. This includes all the global dimensions, shapes and the design choices for the various components of the structure. Three design loops are made: first design loop is the placement of the ZAC in the surrounding, second design loop is about the storage capacity and holds the number of reservoirs, dikes and connection to the river. The third loop is made for the culvert - spillway structure. The hydraulic model of Chapter 5 is used for the verification of potential concepts in these design loops.

6.1 ZAC placement in surrounding

The aim in this design loop is to find the best design of the placement of the ZAC in the surrounding. As stated in the requirements in Section 3.1, the environmental impact must be low and the design must be integrated in the surroundings. Therefore, there are concepts made to show potential placements the surrounding dikes or the excavations. In this design loop three ZAC placement concept are evaluated and the options are shown in Figure 6.1:

1. Constructing dams surrounding the area at the ground level. The slope in the channel bed level remains the same as the current bed slope. There is no excavation to go deeper than the current riverbed level. The maximum crest height compared to the outer ground level is around 9.5 meters.
2. The ZAC is totally excavated to 7.5 meters depth. This can results in a long change in slope in the river bed upstream and downstream of the ZAC. The upstream river becomes steeper to reach the bottom of the ZAC. The downstream connection to the river becomes milder.
3. The ZAC has been excavated for a couple of meters into the ground combined with surrounding dikes. So only partly excavated, and partly above the ground level. This is a combination of the two concepts above and results in a smaller excavation depth of approximately 4 meter and a maximum crest level of 3.5 meter. At connection, the water reduces in height by approximately 2m to the bed of the ZAC. This can be done by a drop of a few meters, a small spillway, or via an upstream channel. In the downstream channel, the slope is made a bit milder, but due to the smaller excavation depth the slope is steeper than option 2. Furthermore, the slope of the ZAC itself can also be designed to be milder to connect directly to the current downstream river bed. This changes the storage capacity and therefore that design choice is made in the next Section 6.2 and is for now not yet reviewed.

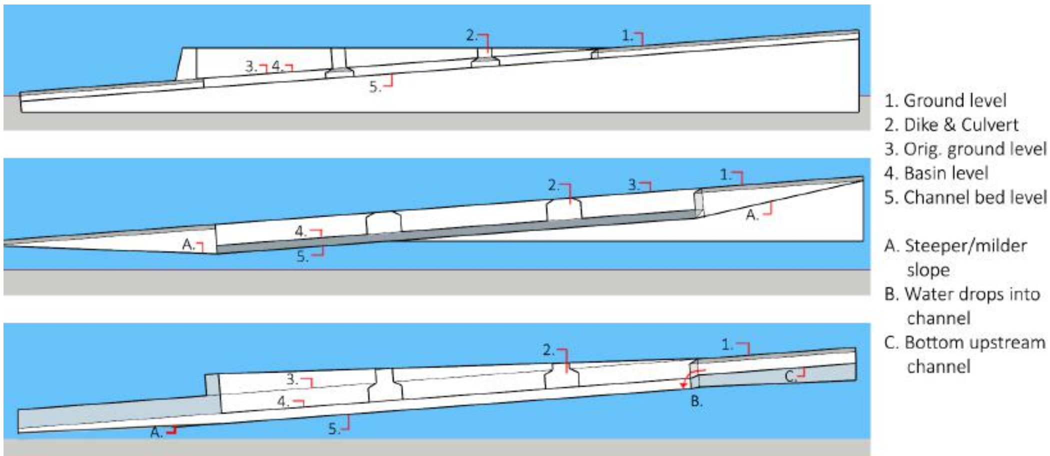


Figure 6.1: The sketches are cross-sections from upstream to downstream along the river bed. Top: The ZAC is placed on top of the ground level with maximum dike crest level of 9.5 meter compared to outer ground level. Middle: The ZAC is excavated 7.5 meters into the ground. Connection to river bed could be made by increasing the upstream slope and decreasing the downstream slope (A). Bottom: The ZAC has been excavated 4 meter into the ground and is surrounded by dikes with a maximum crest level of 3.5 meter to outer ground level.

By the comparing the concepts with each-other and comparing them to the requirements stated in the begin of the section, are the following conclusions made:

- In concept 1 no soil is excavated from the project location. This means all material for the project needs to be imported, while local soil could also be used to construct the surrounding dikes and dikes in the reservoir. This would conflict to the preference to use local soil materials as much as possible to reduce transport time, costs and emissions.
- In concept 1 the ZAC is emerged out of the ground level. This results in a wall of several hundred meters long sticking out of the area which could be seen from afar. This conflicts with the environmental integration. Local population may find such structure disturbing, so excavating the ZAC partly into the ground reduces this effect.
- In concept 2 the soil is excavated everywhere. This means the downstream end is lowered by a lot (7.5 meter). To compensate for this deepening, the downstream slope of the connection to downstream river must be milder. This leads to large distances of downstream river bed which has to be changed and would be conflicting with the preference to do no changes of the river bed downstream the ZAC. Furthermore, a too mild slope may also lead to floods directly downstream of the ZAC, as the discharge capacity downstream of the ZAC could become smaller.

Concept 3 is going to be used instead of Concept 1 and 2. The excavated soil can be used to design the dikes which reduces the transport times and emissions. Further, the connections to the up and downstream river bed can be made smaller in length due to the higher reservoir level compared to Concept 2. Also, in this concept the slope within the ZAC can be changed compared current river slope to have an even better connection to the downstream river. This would be harder in Concept 2 because it would lead to a larger elevation at the upstream reservoir to the outer ground level.

6.2 Determination of the ZAC configuration

The ZAC must fulfill the requirement to store $3.3 * 10^6\ m^3$ or measures downstream must be taken. In order to satisfy this requirement, a design of following elements of the ZAC is made: Number of dikes and reservoirs, dike crest levels and the connection of the structure to the current river. All these elements are directly interacting with each-other and are design steps within one design loop.

6.2.1 Number of dikes and reservoirs

Effect of number of dikes

It is important to denote the difference between the effect of giving the ZAC either one, two or n reservoirs. In Figure 6.2 the ZAC side cross-section is shown, with an upstream channel, the channel in the ZAC, and a downstream outflow. Q_{US} is the actual flow discharge entering the ZAC at the upstream end. $Q_{channel,max}$ is the maximum channel capacity discharge. The ZAC is shown for 3 cases:

1. $Q_{US} < Q_{channel,max}$, the water reservoirs do not get filled up. The water flows downstream without obstruction.

2. $Q_{US} > Q_{channel,max}$ in 1 reservoir. The reservoir gets filled gradually and the water head near the outflow is the largest.

3. $Q_{US} > Q_{channel,max}$ in 2 reservoirs. The first reservoir gets filled faster than when there is 1 reservoir, as the volume is smaller. However, the filling of the upstream reservoir keeps the water head on the second reservoir lower, meaning a smaller discharge at the outflow of the ZAC.

When the discharge is lower in the situation of 2 reservoirs, this means the maximum discharge capacity is maintained for a longer time. This explains the delay in when the peak of discharge occurs in the hydrograph, as already shown in Figure 5.5. The difference in the pressure head between 1 and 2 reservoirs is denoted by $\Delta h_{res1,2}$ in red. It is important to mention that in this figure the volume of the separating dikes is assumed to be 0. According to this assumption, it would be beneficial to create an infinite amount of reservoirs. This is to keep the water level in the last reservoir as low as possible for as long as possible during the design rainfall event. Then, it would be beneficial to create as many reservoirs as possible in the project location.

However, the volume of the reservoir separating dikes is not 0, and therefore the volume of the ZAC gets smaller with an extra dike orthogonal to the flow direction. Besides, during construction more space is required than the final dimensions to get to the final design. Consequently, the maximum number of reservoirs is limited by constructability of the dikes. It is also limited by the costs of 1 dike.

Finally, implementing multiple reservoirs is beneficial to the height of the dikes. If each reservoir has approximately the same volume, considering a constant slope for the bed level, the downstream reservoir dike crests can be at a lower level than the upstream reservoir dikes. This means the dikes are sticking out of the landscape with a constant height, instead of an increasing height. This is less of a viewing disturbance for the local population. The increasing height has been displayed in Figure 6.2, and in the case of reservoirs with equal volume, the series of reservoirs looks like a staircase with steps going downstream. For clarity, compare Figure 6.2 and Figure 6.8.

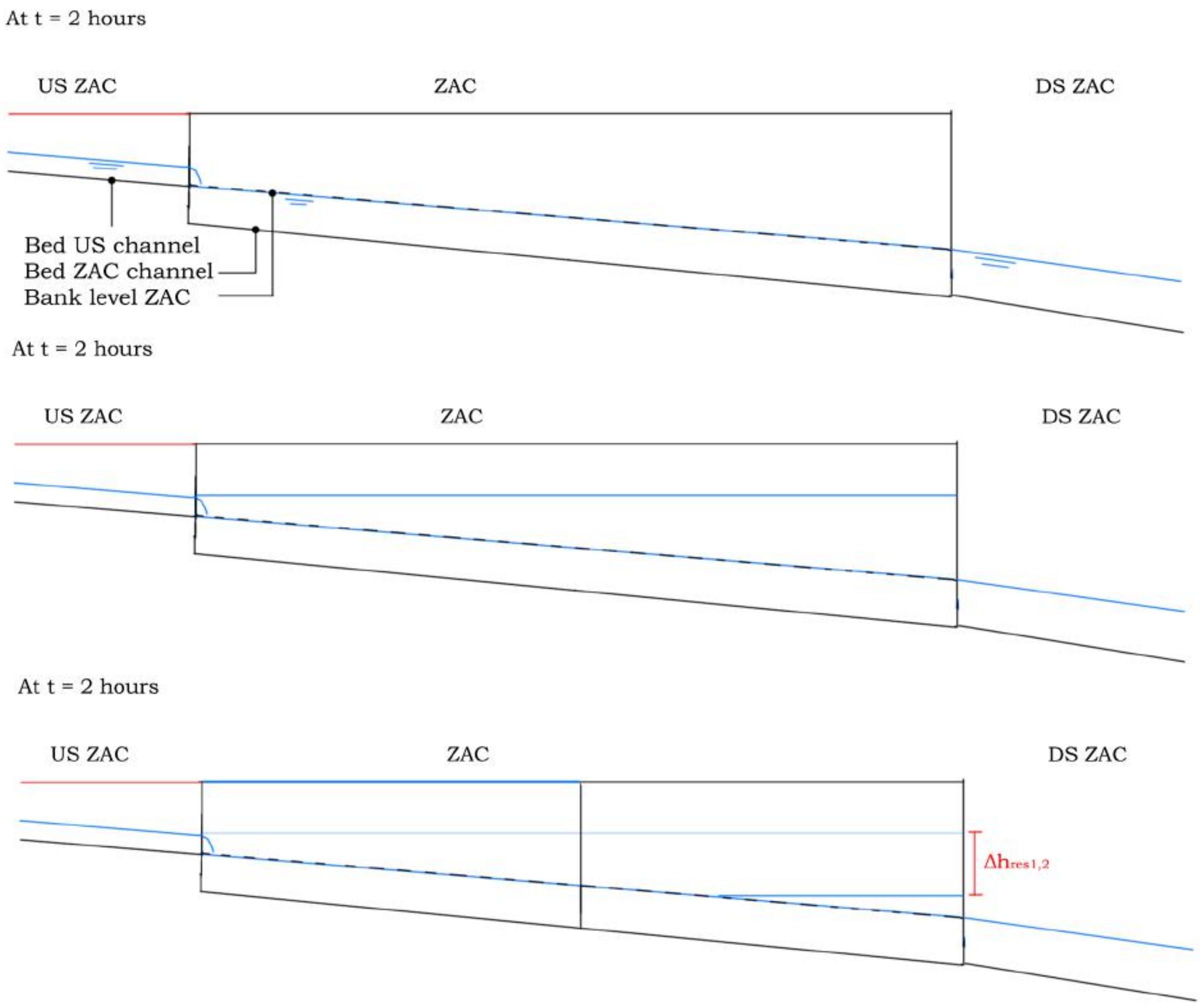


Figure 6.2: There are multiple numbers of reservoirs possible for the ZAC. Top: the water level does not exceed the Bank level ZAC. This means that the water reservoirs are not in use and the flow towards downstream is unhindered. The maximum discharge is equal to or smaller than the discharge capacity of 51 m^3/s. Middle: When Q_{US} > $Q_{channel,max}$, the reservoir starts to fill. At t = 2 hours, the h_{res1} is horizontal in the whole ZAC, as there is only 1 reservoir. Bottom: use of 2 reservoirs. At t = 2 hours, the same amount of water has entered the ZAC, only the US reservoir is filled up firstly. This means the second reservoir has less water in it at the same time compared to the middle ZAC with 1 reservoir. At t = 2 hours, h_{res2} is lower compared to 1 reservoir. The difference is indicated by $\Delta h_{res1,2}$. This leads to a lower pressure head on the last reservoir, meaning a lower outflow compared to a ZAC with 1 reservoir.

Optimizing the number of reservoirs

As explained in the previous section, the maximum number of dikes is limited by the combination of constructability and the extra costs of an extra dike. The ratio needs to be large enough to make it worthwhile to construct additional dikes. To optimize the number of reservoirs, an extensive cost-analysis would be required, where the risk of damages per year is quantified per situation with a different number of dikes versus the investment costs of the extra dikes. Due to time-constraints, this cost-analysis has been omitted and the number of reservoirs has been set to 5. Appendix G.3 shows an extensive explanation of the reasoning behind 5 reservoirs. By placing more reservoirs leads to a marginal effect on the peak discharge reduction, which is too small to be cost-efficient. This marginal effect from 5 to 6 reservoirs has been shown in Figure 5.5 to be. It is not opted that 4 or less reservoirs are implemented, because otherwise the storage capacity is beforehand too small, leading to a too large peak outflow discharge of the ZAC. This means the ZAC would not satisfy its function.

The cost-analysis would come down to a sketch of optimization in Figure 6.3. The optimal costs would lead to an acceptable level of safety which could be translated to a value for the peak discharge value.

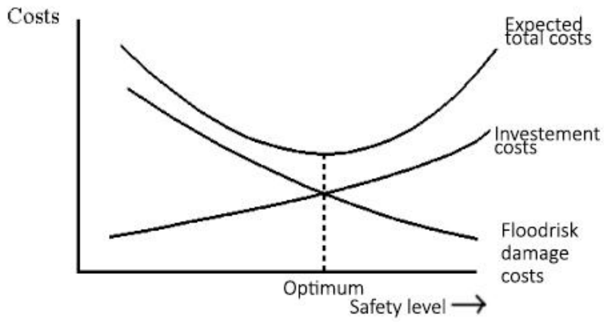


Figure 6.3: The graph displays the total costs for different combinations of safety levels and costs. The total costs are a summation of the total investment costs and flood risk damage costs. When the total costs are minimal, the accompanying level of safety could be translated to a peak discharge value. Subsequently it must be tested if this peak discharge value is acceptable.

6.2.2 Dike crest levels determination

The number of reservoirs is equal to 5 which corresponds into 8 dikes. These are separated in 3 categories; Inner dikes (2, 3, 4, 5), Side dikes (7, 8) and Begin (1) and End (6) dike. The layout of the dikes are shown in Figure 6.4. Each category has a different function. The begin, end and side dikes protect the dry land around the ZAC and must not overflow, as stated in the requirement in Section 3.1.2. The inner dikes separate the reservoirs which are filled if during discharge events occurring once in 24 year (Q_{max} = 51 m^3/s) to once in 474 year (Q_{max} = 207 m^3/s), as determined in Appendix B. Events occurring more often than once every 24 years causes water levels which stay in the ZAC-channel.

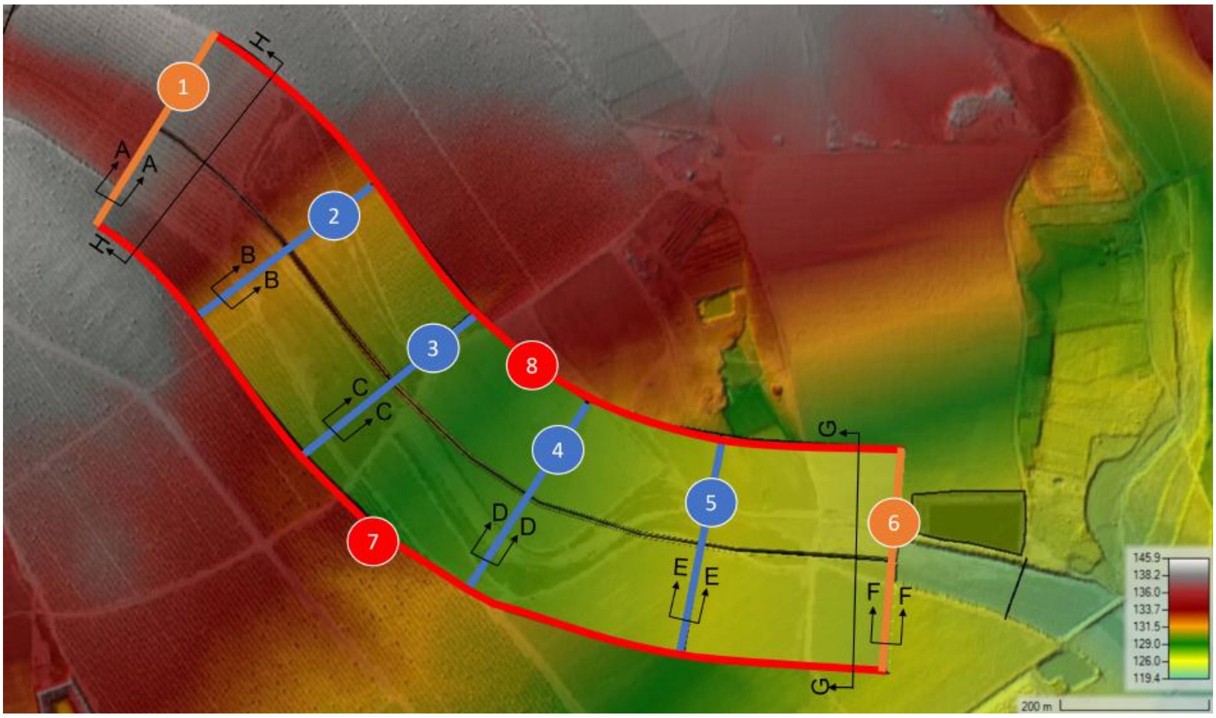


Figure 6.4: ZAC structure: begin dike (1), inner dikes (2,3,4,5), end dike (6) and side dikes (7,8). On the right the legend indicates the height of the terrain with regard to mean sea level (further mentioned as SNM). The indicated cross-sections A-A to H-H are shown in Figure 6.5

Inner and end dikes concepts

The inner and end dikes (2, 3, 4, 5 & 6) must withstand the potential overflow when an upstream reservoir has reached its maximum water level. Two measures can be taken in order to ensure the stability during the occurrence of the overflow. First option is to armour the total inner and end dikes, and these dikes can be safely overflown. Placing the armour can be costly and labour intensive due to the width of 300 meters per dike. Furthermore, the maintenance of an armoured dike is larger compared to a non-armoured dike, which is unfavorable as stated in the requirements 3.1.2.

Therefore, the second option is going to be designed: a spillway structure. This structure can be combined with the culvert structure which results in a lower footprint compared to two separate structures (as shown in Figure 6.12). It results in inner and end dikes which can be made out of the excavated soil. These are so-called earth dikes which are less complex to construct compared to convectional dikes with several additional soil layers. Furthermore, a spillway can be designed in such a way that it has a low maintenance. The width of the slopes of the dikes is set to be 15 meters and a crest width of 3 meters, as presented in Figure 6.5. The stability calculation and tested failure mechanisms of the dikes are shown in the next chapter Structural design.

Dikes crest levels

Each reservoir of the ZAC is reducing the incoming peak discharge of the flood wave. Therefore, the first reservoirs have a faster water level rise compared to the last reservoirs. In order to keep the dimensions of the channel, culvert and spillway more or less equal for each dike, the storage capacity of the first three reservoirs must be larger compared to the last reservoirs.

Therefore, a distinction is made between dikes 1,2,3 & 4 and dikes 5 & 6. The first four dikes have a crest level of 7.5 meter and the other two downstream have a crest level of 7.0 meter compared to local basin level, and are shown in Figure 6.5. The side dikes (7 and 8) are $0.5\ m$ larger than the inner dikes and therefore have a varying crest height from 8 upstream to 7.5 meters downstream.
To ensure the side dikes (7 & 8) are not overflown, the design choice is made to increase these dikes by 0.5 meter compared to the inner dikes. It means that the water levels should rise more than 0.5 meter above the whole length of a inner dike (300 meters), which then acts as a extra spillway with a discharge of roughly 176 m^3/s. This discharge combined with the discharge of the to be designed spillway above the culvert is expected to be larger than the incoming design peak discharge (207 m^3/s), which indicates that the 0.5 meter increase is a safe design choice.

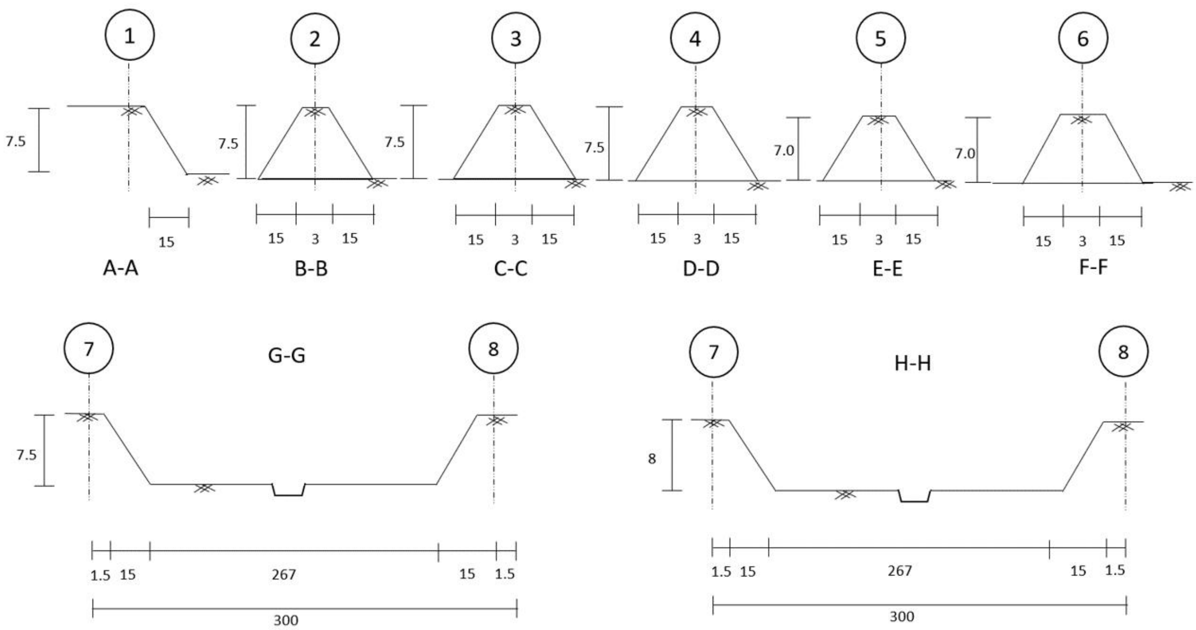


Figure 6.5: Cross-sections of the dikes 1 to 6 with corresponding crest height and width. Dikes 7 and 8 have a decreasing crest level from 8 meters to 7.5 meters above the local basin level.

6.2.3 Connection of reservoirs to river bed

The ZAC structure is placed in the current river bed. The bottom level upstream is 7.5 meter lower than the upstream ground level. Therefore, this elevation change has to overcome. This design step explains the applied measures to lower the bed level of the current river to the reservoir level.

Connection of first reservoir to upstream river

The first reservoir is placed in the current river ($i_{up,riv}$ = 0.007 [-]). This connection can be made by steepening the upstream riverbed or by placing an extra spillway structure at the entrance of the ZAC. If the second option would be implemented, an additional spillway structure must be designed which has other dimensions compared to the spillways at the inner and end dikes. This complicates the construction and would not be favorable. Furthermore, by placing a spillway structure at the entrance, an additional impact load acts on the bottom of the first reservoir. By steepening a part of the upstream river slope these problems are eliminated. Therefore, this option is chosen to be implemented in the design of the upstream connection.

The basin level at dike number 1 is located at SNM + 132.10m, which is 7.5 meters lower to compared to local ground level. The upstream channel collects the water from the upstream river with a slope of 0.008 [-] which results in a total length of 1450 meters and is shown in Figure 6.6. This channel is placed in the current upstream riverbed and unlike the downstream river, there are no orchards situated in the riverbed. Therefore, an upstream channel can be implemented without any interference with the agricultural land which is a preference of the client.

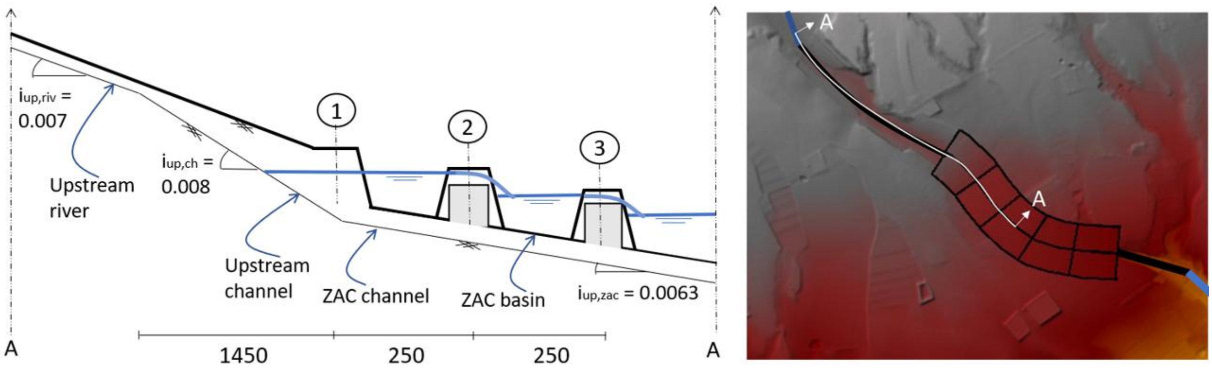


Figure 6.6: Upstream channel configuration via channel cross-section A-A. Vertical direction is not scaled to clarify the configuration of the structure. The upstream bottom slope of the river (0.007) and the upstream channel (0.008) are steeper than the ZAC slope (0.0063).

Water flow through ZAC in normal conditions

The ZAC structures must not be acting as an obstacle during normal conditions ($Q_{max,nor}$ = 51 m^3/s, as stated in the requirements Section 3.2. Therefore, the water has to flow through the ZAC and this is done by a channel. The channel bottom is set to be at the same level as the culvert bottom level to have the lowest value of obstruction in normal conditions and has a trapezoidal shape. Furthermore, to connect the channel with the culvert-spillway structure, the top width of the channel is equal to the width of the spillway. A total overview of the connection of the channel with the culvert-spillway structure is presented in next Section 6.3. It leads to the following dimensions of the channel:

- Top width = 7 m
- Bottom width = 5 m
- Channel depth = 2 m

Connection of outflow ZAC and downstream river

The last reservoir ground level is located at the same level (SNM + 124.24m) as the downstream river level, as presented in figure 6.7. Section 5.6 shows in Figure 5.7 the modelled flood map during once in 474 year discharge event. It shows that the downstream river has the capacity to withstand such events and flood arise only when the water enters Balsicas. Therefore, no additional measures are taken at the connection of the downstream river and the outflow location of the ZAC because the river capacity at the connection point is sufficient for a situation without a ZAC structure.

However, the outflow velocity of the ZAC structure at dike number 6 could be large due to the large water level difference of the last reservoir compared to the outside water level. By modeling the structure in HEC-RAS, it predicted flow velocities of 4.7 m/s and therefore bed protection must be implemented. The calculation of the bed protection and filter layer is presented in the Chapter 7. The length of the bed protection is for now approximated to be 100 meter and the required stones are placed in the current river bed.

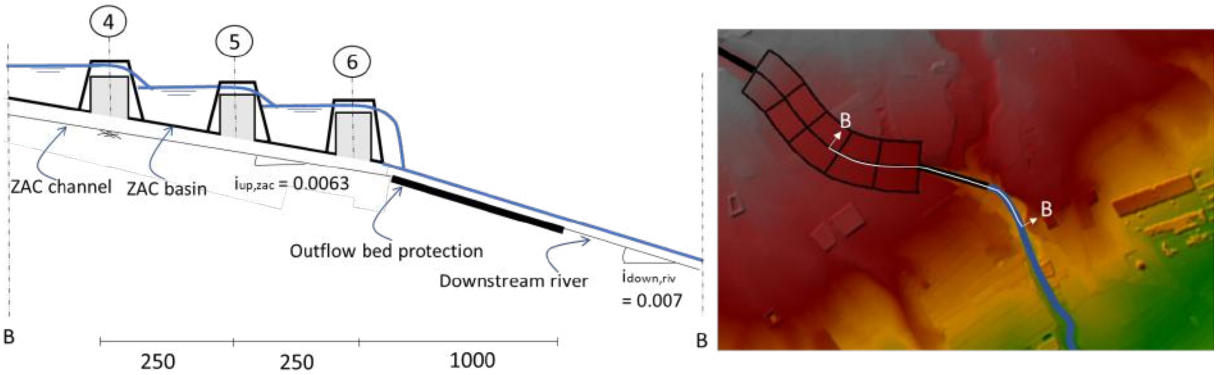


Figure 6.7: Outflow channel configuration via channel cross-section B-B.The current downstream bottom slope stays the same but is steeper than the ZAC. It is important to notice that the 6th dike shows water flowing over the spillway, but the culvert flow at the its bottom is not shown. This means the water also passes for water levels lower than the dike crest height.

6.2.4 Verification of the ZAC storage capacity requirement

The ZAC structure must fulfill the requirement to store $3.3 * 10^6\ m^3$ water or additional measures downstream have to be implemented to compensate for a too small storage capacity. The storage of the ZAC depends on the reservoir level, slope and dike height, and are determined in the previous subsections. To check the storage capacity of the ZAC, the total storage volume must be calculated. This is done by calculating the volume per reservoir. For overview, the side view of the ZAC and its reservoirs are depicted as in Figure 6.8 including the levels upstream and downstream the ZAC.

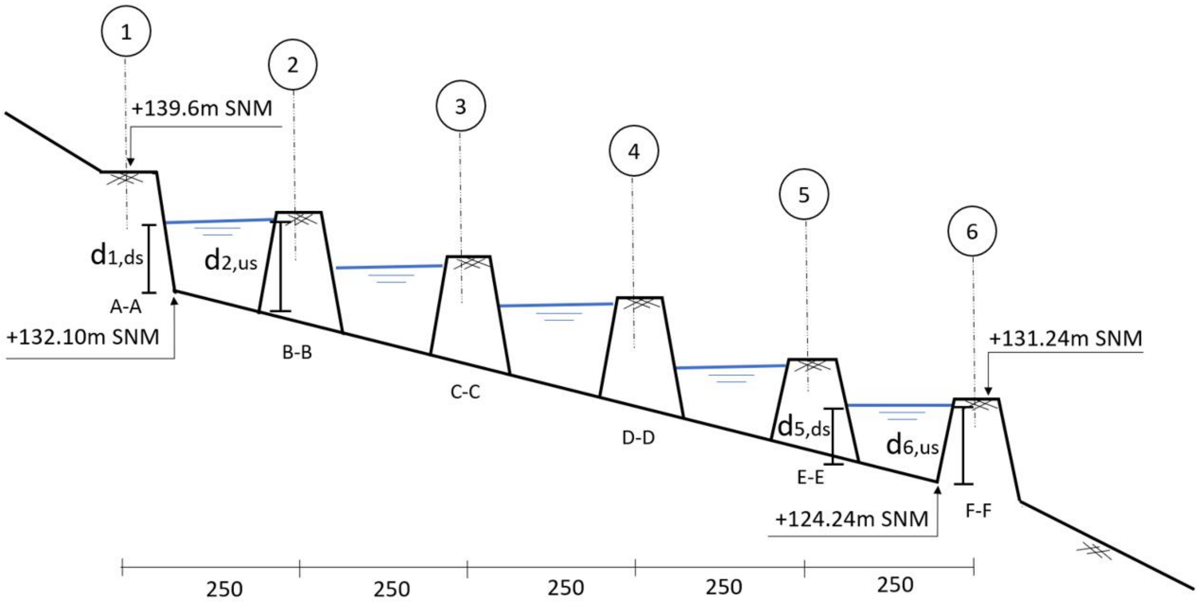


Figure 6.8: The 3 upstream reservoirs are bordered by dikes (1,2,3,4) with equal dimensions on the upstream and downstream end (h = 7.5m). The 2 downstream reservoirs are bordered by 2 equal smaller dikes (5,6) on the upstream and downstream end (h = 7.0m). This is an unscaled schematic representation of the dike design and the upstream and downstream connection is not shown

The width of all reservoirs is $300m$. The length of each reservoir is $250m$ from core to core. $d_{2,us} = 7.50m$, this is the maximum depth downstream of the first three reservoirs, as the maximum depth is roughly equal to the dike height. The height difference between the upstream (SNM + 132.15m) and downstream bottom level (SNM + 124.22m) of the ZAC-channel is $7.93\,m$. This means the slope of the ZAC basin and channel is:

$$i_{zac} = \frac{\Delta y}{\Delta x} = \frac{132.10 - 124.22}{1250} = 0.0063[-] \tag{6.1}$$

Using this slope, the upstream depth of the first three reservoirs is:

$$d_{1,ds} = d_{2,us} - L_{res} * i_{res} = 7.50 - 1250 * 0.0063 = 5.93m \tag{6.2}$$

For the smaller dikes (5) and (6) the dike height is $d_{5,us} = 7.0\,m$. Applying the same procedure leads to $d_{5,ds} = 5.43m$. The volume of a reservoir is calculated by multiplying the average depth of the trapezoid by the length and width of the reservoir. At this point, the volume $V_{no\ dikes}$ is calculated without the presence of the dikes:

$$V_{no\ dikes,1,2,3,4} = \frac{d_{1,ds} + d_{2,us}}{2} * N_{res} * L_{res} * B_{res} = \frac{5.93 + 7.5}{2} * 3 * 250 * 300 = 1.51 * 10^6 m^3 \tag{6.3}$$

$$V_{no\ dikes,5,6} = \frac{d_{5,ds} + d_{6,us}}{2} * N_{res} * L_{res} * B_{res} = \frac{5.43 + 7.0}{2} * 2 * 250 * 300 = 0.93 * 10^6 m^3 \tag{6.4}$$

and the total volume without dikes is:

$$V_{no\ dikes,tot} = V_{no\ dikes,1,2,3,4} + V_{no\ dikes,5,6} = (0.93 + 1.51) * 10^6 = 2.45 * 10^6\ m^3 \tag{6.5}$$

The total volume of all dikes needs to be subtracted. This is done using Table 6.1. First, it is determined how many dikes of each type are present in the current storage. Dike 1 to 4 can be set to 3.5 dikes, as at the upstream end only the right half is part of the calculated storage volume. There are 1.5 dikes present for dike 5 and 6. Since there is 1 dike on each side, the net volume is $2 * 0.5 = 1$ for side dikes 5 and 6. Second, the cross-sectional area of each dike is calculated and subsequently, the volume per dike and the volume reduction per dike type.

Table 6.1: Net volume of dikes that is subtracted from total storage volume (including the dikes).

Dike type	Nett number of dikes	Dike cross-section $[m^2]$	Length dike $[m]$	Volume per dike $[m^3]$	Total volume $[m^3]$
1-4	3.5	$(15 + 3) * 7.5 = 135$	300	40,500	141,750
5-6	1.5	$(14 + 3) * 7 = 119$	300	35,700	53,350
7-8	1	$(16 + 3) * 8 = 152$	1250	190,000	190,000
				Total	**385,100**

The total volume reduction is the sum of these contributions. Finally, $V_{excl\ dikes,ZAC}$ is:

$$V_{excl\ dikes,ZAC} = V_{inc\ dikes,tot} - V_{dikes} = (2.44 - 0.385) * 10^6 = 2.06 * 10^6 m^3. \tag{6.6}$$

This is lower than the requirement of storage capacity from Section 3.1.2, which is $3.3*10^6\ m^3$. However, this is acceptable when a combination with additional measures downstream is made, as mentioned in the same section. This is done by relatively simple measure to increase the inlet discharge capacity of the current downstream channel near Balsicas.

6.2.5 Downstream river measures

The maximum capacity of the downstream river bed is at its critical point equal to 51 m^3/s, as determined in Appendix B and is located at the entrance of the already existing man-made downstream channel. The location of this inlet is shown in Figure 6.9 together with the lay-out of the other parts of the ZAC in the river. To prevent floods downstream, the maximum culvert capacity is also set to be at 51 m^3/s. The culvert achieves this discharge just before the spillway starts to discharge water. At the moment the last spillway is used, it starts to deliver an additional discharge into the downstream channel (in total 106 m^3/s), which is a larger discharge than the maximum capacity of the current downstream river at the critical point. Therefore, additional measures must be taken.

As stated in the preferences in Section 3.2 an excavated downstream channel is not a preferred option. A downstream channel from the outlet of the ZAC toward the end of the design area uses agricultural land, in which the river bed is used for multiple orchards or farms. In order to keep this current situation the same, the bottle neck location (inlet of the existing downstream channel) from Figure 5.3 can be upgraded to handle larger discharge capacities. The current inlet of existing channel suddenly starts in the middle of a orchard. Currently, no guidance structure is used for the inlet of the downstream channel.

By using 5 reservoirs in the ZAC structure, the out flowing peak discharge of the ZAC is 106 m^3/s for the design rainfall event with a return period of once in 474 years. The current critical cross-section is at the inlet of the downstream channel and is 51 m^3/s. By improving the inlet to a capacity of 106 m^3/s the total capacity of the downstream river is improved.

Guiding the flow is done by making a inlet structure. This is done by adding a wall to either side of the beginning of the existing channel and funnels the water flow from the river into the channel. After optimization in the HEC-RAS 2D model, the height of the walls turned out to be 1.0 meter high and around 120 m long under an angle of 45 degrees. Figure 6.10 shows the results when these walls are implemented in HEC-RAS. Without the guiding walls is a flood of 0.16 meters predicted. With the guiding walls no flood is predicted by the HEC-RAS 2D model, as presented in Figure 6.10.

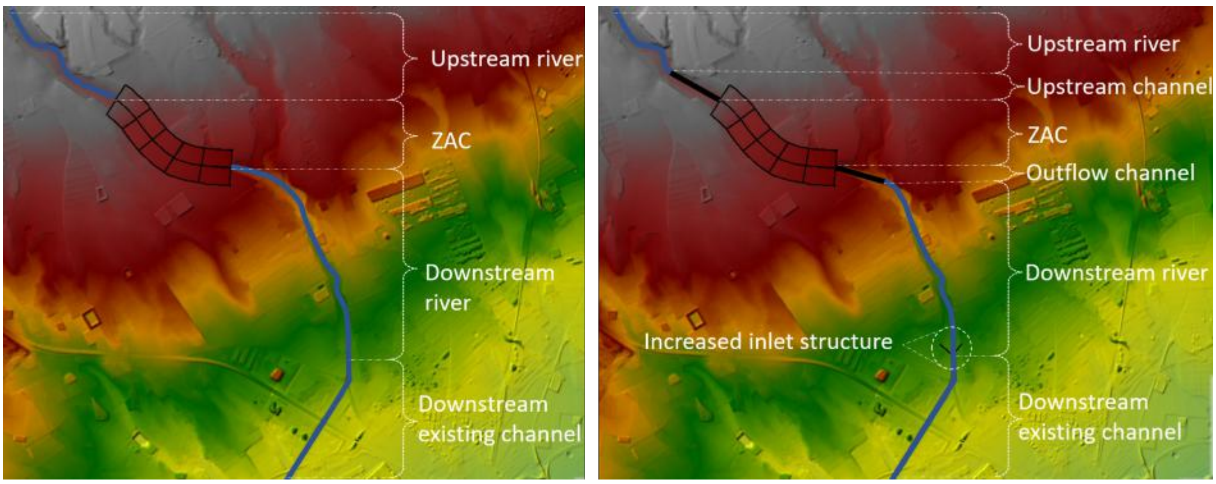
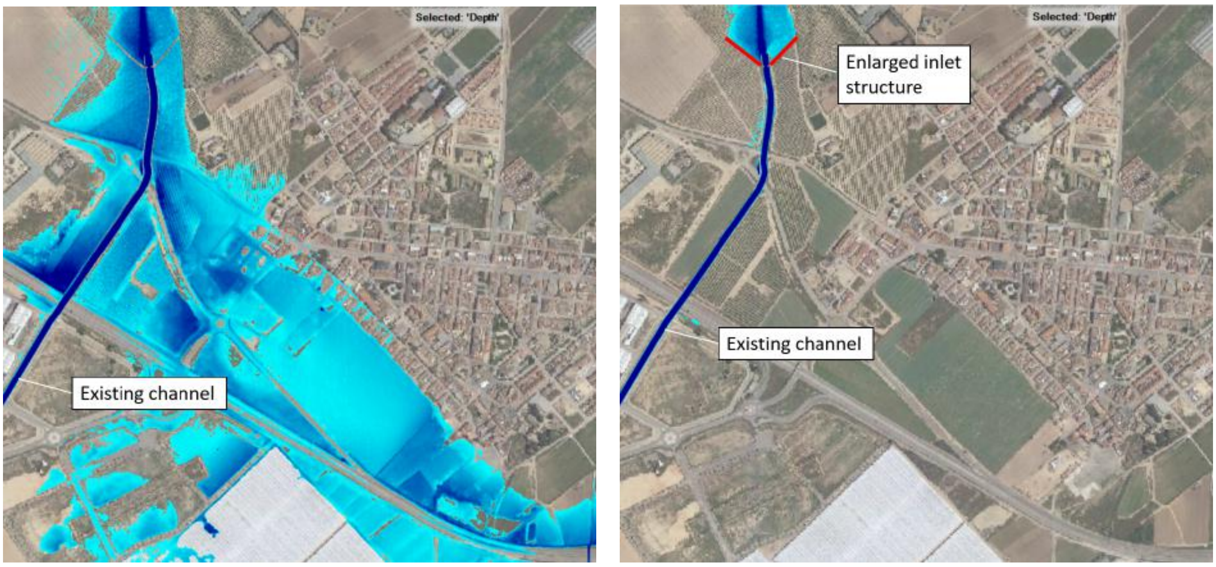


Figure 6.9: Left: A ZAC in the river system. Right: Channel layout and ZAC in the river. This type of ZAC is smaller in terms of storage capacity, and therefore requires additional measures up- and downstream. Both an inflow channel and an increased capacity inlet structure at the bottle neck location are applied.



(a) Situation without guiding walls. There is a flood occurring

(b) Applying these guiding walls results in no flood.

Figure 6.10: Model results of the situation downstream with and without guiding walls.

By implementing all the designs given in the previous sections of Chapter 6, no flood is predicted via the HEC-RAS 2D model. Figure 6.11 shows the cross-section along the flow path from the upstream river to the funnel-shaped inlet structure of the downstream channel.

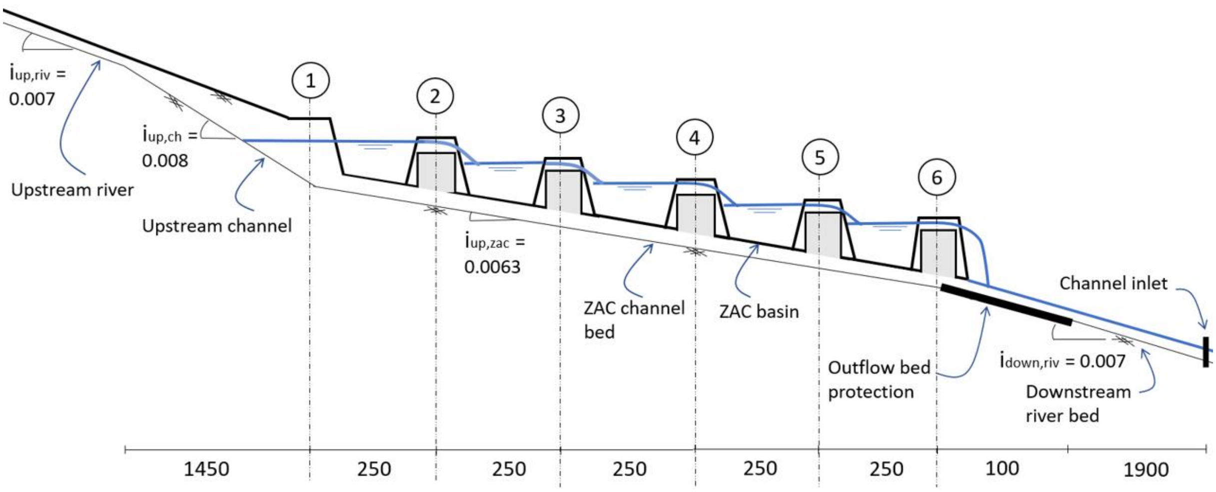


Figure 6.11: Cross-section along the water flow from upstream river to downstream river. The upstream river is connected by a upstream channel to the ZAC reservoirs. The begin (1), inner (2,3,4,5) and end (6) dikes are shown with the culvert-spillway structure (grey). Water flows over the spillway and goes through the culverts to the downstream channel with 100 meter bottom protection. After 1900 meter it enters the current existing channel through the increased channel inlet

6.3 Design of the Culvert-Spillway structure

The third and last design loop is made for the culvert-spillway structure. It is placed in the middle of dikes 2, 3, 4, 5 &6. It provides the connection between the reservoirs and regulates the discharge. The culvert-spillway structure is designed as one structure. The following structural components are considered and are the design steps in this design loop: Spillway, Culvert, Retaining walls, Flared walls and the Foundation. All elements are shown in Figure 6.12.

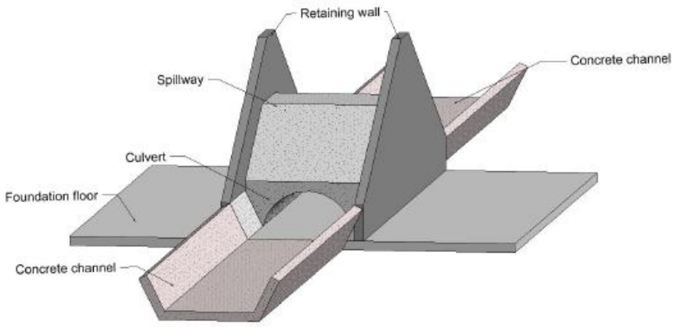


Figure 6.12: 3D view of a culvert-spillway structure with all its elements

6.3.1 Design approach of the Culvert-Spillway structure

For the structural elements, multiple alternatives can be applied to create a culvert-spillway structure. It is therefore important to first select the optimal shapes of the individual elements. The design approach overview is shown in Figure 6.13 and is applicable until Section 6.3.4.

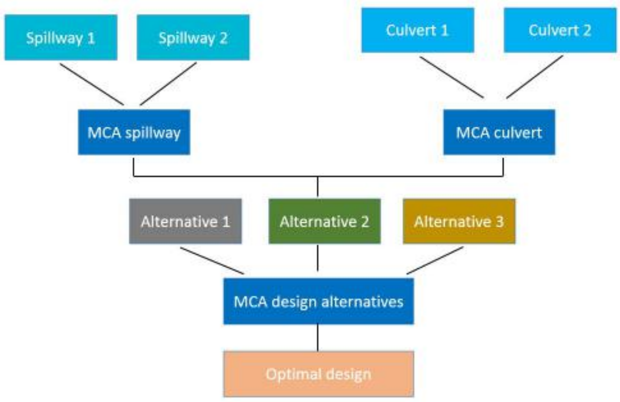


Figure 6.13: In the first step after identifying the required structural elements the different types of alternatives per element are considered. This is done for different types of spillways and culverts. In Section 6.3.5, the second step of the design overview is performed. Then the optimal design is verified at the requirements.

For the functional design, the foundation and the retaining walls are not considered because it is assumed to be a supporting structure and has no additional impact on the function of the ZAC structure. Therefore, the these are designed in Chapter 7.

Based on the requirements indicated in Chapter 3, multiple design alternatives are plausible for the culvert-spillway structure. Further, the requirements of the structural elements are also defined by the

HEC-RAS model. The design alternatives are determined by the following aspects:

- Building materials
- Spillway structure
- Culvert structure

For each structural element (spillway and culvert), concepts are made for which a multi-criteria analysis (MCA) is used to determine the optimal one. This allows for a qualitative consideration of the different options. From these options, three designs are created. Subsequently, the designs are evaluated with an MCA consisting of varying criteria and checked at the stated requirements. Then an optimal design is selected. Appendix I shows all steps of the MCAs and gives a further explanation.

6.3.2 Selection of the building materials

For the construction of the structure, different structural materials are considered: steel, timber and reinforced concrete. For each structural element the selection of the building materials is based on the durability, environmental impact, and execution of the materials.

Durability
Durability is resistance of the building materials to the surrounding environment, without them experiencing damage or regular maintenance. Both steel and timber are susceptible to the surrounding environment, which requires the need for regular maintenance. However, if designed correctly, concrete can be a much more durable building material. Due to this high durability, it has low maintenance costs compared to steel and timber.

Environmental impact
This criterion takes into account the energy consumption and the expelled emissions. Both steel and concrete are highly energy and emission-intensive compared to timber. Thus, timber has the lowest environmental impact.

Execution
Both steel and timber connections are susceptible to the surrounding environment in terms of corrosion or fungus growth, which requires the need for regular maintenance. On the other hand, if designed correctly, concrete connections are durable, and require low maintenance compared to steel and timber.

Most suitable building material
The structure has a service life of 50 years and lies in a remote location. Based on that, durability and maintainability are taken as the governing criteria for the structure, therefore having the highest weight factors in the MCA. Staying in line with the European green deal to become the first climate-neutral continent by 2050 (Norton Rose Fulbright, 2021) and in line with the preference of limiting emissions in the project, the environmental impact criterion has a significant influence on the selection of the building material. From the MCA, shown in Appendix I.1 follows that concrete is the optimal building material for the structural components.

6.3.3 Structural element: Spillway type and shape determination

A spillway structure is chosen over the option to armour the total dike, as stated in Section 6.2.2. If the armouring option was chosen, the total outer area of inner and side dikes must be reinforced and this is costly. Furthermore, the need for maintainability of armour is larger. Due to the requirement to have a low maintainability of the ZAC structure, the option to armour the total dike is not favorable. A spillway structure is relatively small and can be combined with the culvert structure. Therefore, a spillway structure is chosen to be implemented in the design of the ZAC.

Different types and shapes can be used for the spillway. Regarding the remote location of the ZAC structure, an uncontrolled spillway is selected. Also, the uncontrolled spillway is more affordable than a (remote) controlled spillway, because no mechanical structures are needed. Furthermore, for a conceptual design of the ZAC in this report, a chute spillway is applied, since this is a common and basic design. The considered shapes of chute spillways are shown in Figure 6.14 and are:

- Rectangular spillway
- Right-angled trapezoidal spillway
- Trapezoidal spillway
- Sharp-crested spillway

Figure 6.14: 3D view of the considered types of chute spillways. From left to right: Rectangular, Sharp-crested, Right-angled trapezoidal and trapezoidal spillway

The types of the spillways are further elaborated in Appendix I.2.

Requirements spillway

Modelling the spillway-culvert structure in the HEC-RAS 2D model resulted in the requirements for the spillway. The water depth over the spillway crest (h) is approximated to be 2.0 meter when the crest width (W) is set to be 7.0 meter compared to the floor of the culvert. The exact value of the spillway height (P) depends on the dimensions of the culvert, however for an initial approximation of the dimensions, the height is equal to 4.5 meter for all the spillway types. The thickness of the hollow spillways (t) are determined in the structural design phase. For an initial approximation, it is assumed to be equal to 0.5 meters. Based on these spillway requirements and the previously mentioned shapes, different spillway types are distinguished, which are presented in Figure 6.15, namely:

- Rectangular shape
  - Broad crested
  - Sharp crested
- Right-angled trapezoidal shape
  - Broad crested and positive slopes
  - Broad crested and downstream ramp

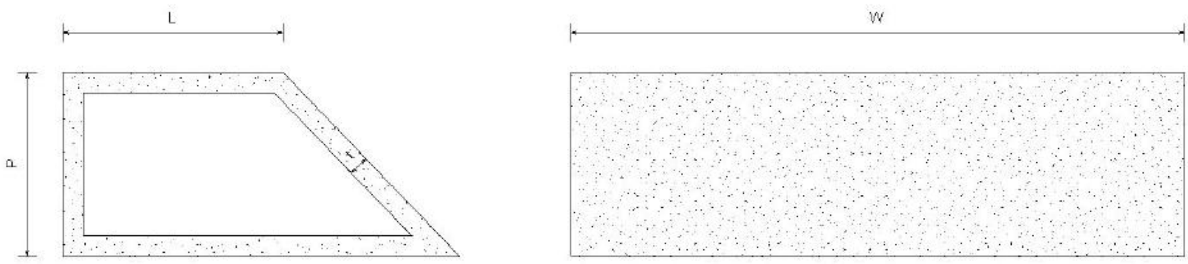


Figure 6.15: Parameters of a spillway. P = spillway heigth, L = spillway length, W = spillway width, t = thickness of concrete element. This figures shows the parameters on a right-angled trapezoidal spillway.

Dimensions of spillway types

A spillway is broad crested when the ratio between the water depth over the spillway (h) and the length of the spillway in flow direction (L) should approximately range between 0.1 - 0.4 (h/L = 0.1-0.4) (Azimi, Rajaratnam & Zhu, 2013). For a sharp-crested spillway, the ratio should be greater than approximately 2.0 (h/L > 2.0). The spillways with a ratio between 0.4 - 2.0 lie outside the scope of this report since only broad-crested and sharp-crested spillways are considered.

With the ratio for a broad-crested and sharp-crested spillway, the minimum and maximum length of the spillways are determined and are presented in Figure 6.16.

Minimum length broad-crested

$$h/L = 0.1 - 0.4$$

$$2/0.4 = 5\,m$$

Maximum length Sharp-crested spillway

$$h/L > 2.0$$

$$2/2 = 1\,m$$

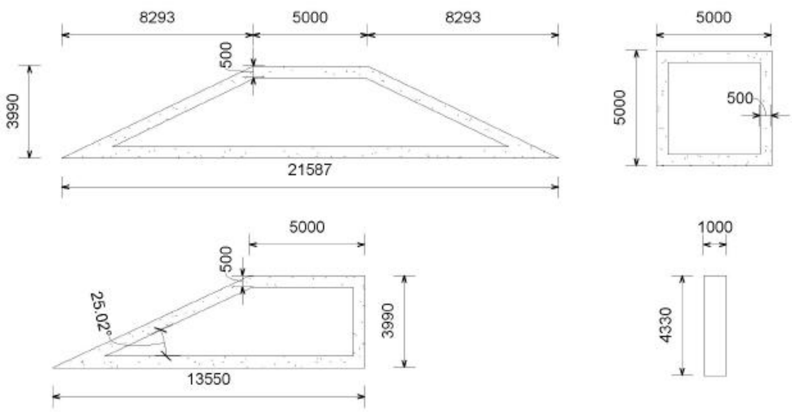


Figure 6.16: First dimensions in millimeters per cross-section per shape of spillway. Left top: trapezoidal spillway, Right top: rectangular spillway, Left bottom: Right-angled trapezoidal spillway, Right bottom: Rectangular sharp crested spillway.

Design choice of the spillway shape

In order to determine the best spillway shapes, an MCA is made and it evaluates the maintainability, environmental impact, level of difficulty in constructability and effectiveness of the structure on its function.

Maintainability
The probability of abrasion is elaborated per spillway, since this leads to a higher need of maintenance. The rectangular shapes have a higher probability of abrasion than the trapezoidal shapes, because the slopes are sharp-cornered.

Environmental impact
Only the amount of building materials are considered. After all, fewer building materials lead to an overall lower environmental impact. Based on the dimensions, the trapezoidal spillway has the highest volume of building materials and the sharp-crested spillway the lowest. This is confirmed by the volume per spillway.

Difficulty of constructability
This criterion depends on the selected building material of the spillway. It is expected that the rectangular spillways are easier to construct and are less labour-intensive compared to the trapezoidal spillways since it is essentially a box-girder. For the trapezoidal spillways, slope accuracy needs to be guaranteed, and reinforcement placement needs to be thought of in the sloped area. This leads to the trapezoidal spillways being more difficult to construct, hence being more labour-intensive.

Functionality
This criterion evaluates the hydraulic performance of the spillway. The hydraulic performance is based on the flow efficiency of the spillway, this efficiency is based on the discharge coefficient, C_d. Based on the discharge coefficient, the broad-crested trapezoidal spillway is the most efficient. This is logical, as the upstream slope reduces the separation zone at the entrance of the spillway.

Most suitable spillway shape
For shape of the spillway MCA the same criteria as the building materials MCA has been used, regarding the weight factors of functionality, maintainability, constructability and environmental impact. From the MCA follows that the broad-crested trapezoidal spillway is the overall optimal spillway. The overview of the spillway with dimensions is given in Figure 6.17.

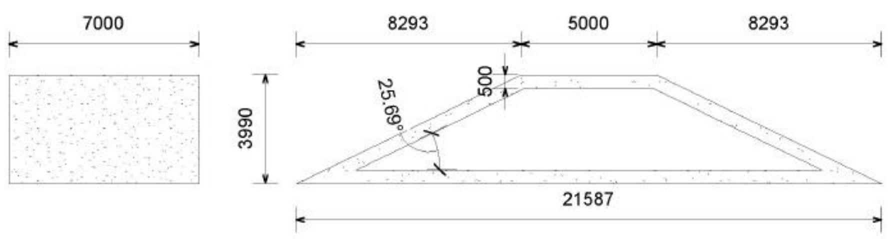


Figure 6.17: The most suitable spillway is the broad crested trapezoidal spillway. Left: the front view of the spillway is shown. Right: the longitudinal cross-section. Both show the rough first dimensions in millimeters

6.3.4 Structural element: Culvert shape determination

The culvert connects the reservoirs with each-other and when it is right designed it can lower the incoming discharge wave. This design step checks the best shape of the culvert to be used in the culvert-spillway structure.

Requirements of culvert

The reservoirs in the ZAC should start filling when the incoming discharge is larger than the maximum discharge capacity of the downstream river. During an extreme rain event, the maximum discharge through the culvert must be reached and the water level in the reservoir rises. Therefore, the maximum discharge through the downstream river without flooding the surrounding area is set equal to the discharge through the culvert. All discharges below the maximum discharge of the downstream channel are defined as normal conditions. The maximum discharge in the downstream river is $51\ m^3/s$ and the determination is explained in Appendix B. The discharge flowing through the culvert follows from the 2D HEC-RAS model. The limit of $51\ m^3/s$ through the culvert results in the cross-sectional area of the opening to be approximately 7.5 m^2 when it is combined with a spillway 4.5 meter above the top of the culvert.

Dimensions of culvert shapes

The shape of the culvert is tested based on the different effect it can have on the discharge capacity, combined with the use of the spillway. The following culvert shapes are considered and shown in Figure 6.18, namely:

- Rectangular culvert
- Arched culvert
- Multiple circular culverts

See Appendix I.3 for the full elaboration of the culvert shapes.

As an initial design, the width of the bottom of the culvert opening is set to the same width of the channel (the horizontal part), which is 5.0 meters. For an initial design, the concrete thickness of the culverts is set to 500 mm. This is evaluated further in the structural design phase.

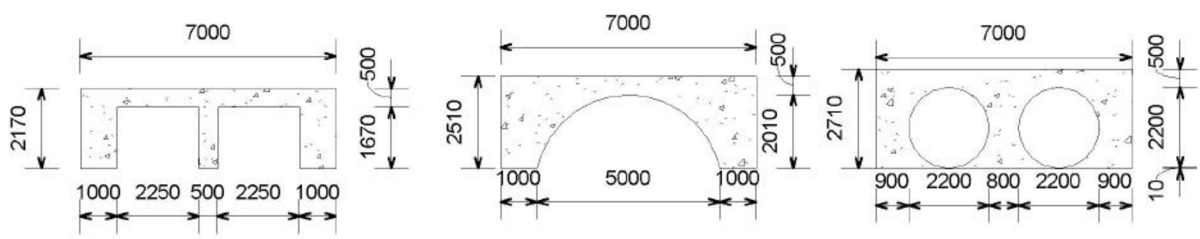


Figure 6.18: Shapes of culverts with the first dimensions in millimeters to have an area of 7.5 m^2.

Flared wall

With the 2D HEC-RAS model, the influence of the flared walls on the ZAC structure is calculated. The pre-determined requirements of the culvert and spillway, are kept constant during the simulations. An overview of the HEC-RAS inputs and outputs can be found in Appendix I.3. It is determined that the presence of the flared walls has no significant effect on the discharge or flow velocities of the ZAC structure. Therefore, in the case that flared walls are present, the construction mainly depends on practical reasons, naturally taking the acting loads on the structure into account.

Optimal shape culvert

The different types of culvert shapes are evaluated based on structural efficiency, maintenance and material use and the full explanation is shown in Appendix I.3.

Structural efficiency (Williams et al, 2012)
The structural efficiency of a culvert depends on the efficiency of distributing the loads acting on it. The concrete design uses strut and tie modelling for determining where compression and tensile forces occur. The most realistic model is the one that features the fewest and shortest ties. Based on the hydraulic structures manual, and keeping the strut and tie modelling in mind, a rectangular box cross-section is the least favourable option. An arched or circular-shaped cross-section would be the better solution.

Maintenance
A culvert with multiple barrels is more prone to the accumulation of sediment and debris between the different openings. This leads to a higher need for maintenance, which is not desirable. Also, this opposes the requirement of no clogging from Chapter 3. An arched shape is preferable over a rectangular cross-section because of fewer re-entrant corners in the cross-section. Application of corners is undesirable, as the corners are prone to large stresses, resulting in a higher risk of concrete cracks. This could lead to a higher need for maintenance or even failure in the worst case.

Material and environment
More material use, for the same function, leads to a larger environmental impact. A multiple barrel culvert can have larger spans. However, it requires more material because of the intermediate walls. With the same culvert opening area, the arched shaped cross-section is larger than a rectangular box section. However, it is expected that due to the larger spans of the arched shape, it still is more favourable with respect to material use. Based on the dimensions, it can be seen that the multiple circular culvert has the highest amount of building materials and the rectangular culvert the lowest. This is understandable, seeing the volume per culvert.

Most suitable culvert
For the culvert MCA the same criteria as the building materials MCA has been used, regarding the weight factors of functionality, maintainability, constructability and environmental impact. From the MCA follows that the arched culvert is the overall optimal shape for the culvert. Furthermore, it is assumed that the culvert shape has a sufficient size to allow flow of large soil particles. Thus, clogging of the culvert has a lower probability of occurring. This culvert is shown in Figure 6.19.

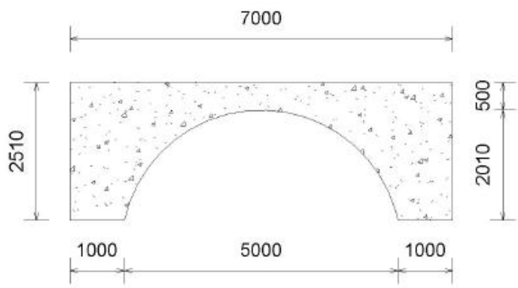


Figure 6.19: Most suitable culvert is the arched culvert. The figure shows the arched culvert with the first rough dimensions in millimeters.

6.3.5 Design selection

Based on the previously constructed MCA's per structural component, three design alternatives are constructed. This is the second step in the design approach, as indicated in the overview in Figure 6.20 Each design alternative focuses on a different criterion. The first design alternative focuses on the performance of the structure, which covers the flow efficiency of the spillway and the structural efficiency of the culvert. The second design alternative focuses on the maintainability of the structure. And the third design alternative focuses on the material use and environmental impact of the structure, hence the structural components with the minimal material use. For all the design alternatives, it is decided not to construct flared walls since it has no added value to the ZAC structure and would only increase the amount of building materials.

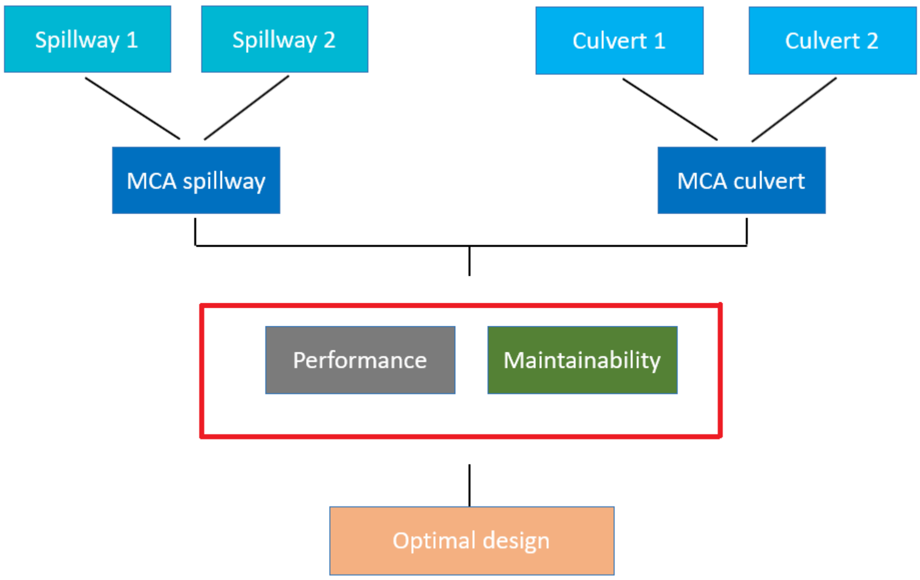


Figure 6.20: The second step in the design approach is to combine the design alternatives to a multiple combined alternatives, upon the optimal design alternative is chosen. Thereafter, this design is verified at the given requirements.

As indicated in appendix I.5, the structural components with the highest rank for performance are identical to that of maintainability. Thus, two design alternatives are constructed instead of three.

Alternative 1
The structural components with the highest rank for performance and maintainability are:

- Trapezoidal spillway
- Arched culvert
- Retaining wall

Alternative 2
The structural components with minimal material use are:

- Rectangular sharp-crested spillway
- Rectangular culvert
- Retaining wall

Verification of the alternatives
The alternatives are designed via the requirements from the HEC-RAS 2D model and the given requirements from Chapter 3. It results in alternatives which will fulfill all the requirement and therefore the optimal design is determined via the use of an MCA to check which alternative will preform better.

Optimal design
As previously indicated, the performance and maintainability of the structure are the governing criteria. Seeing that both the trapezoidal spillway and the arched culvert rank highest for both criteria, and as overall structural components, it is concluded that alternative one is the optimal design for the ZAC structure and is presented in Figure 6.21.

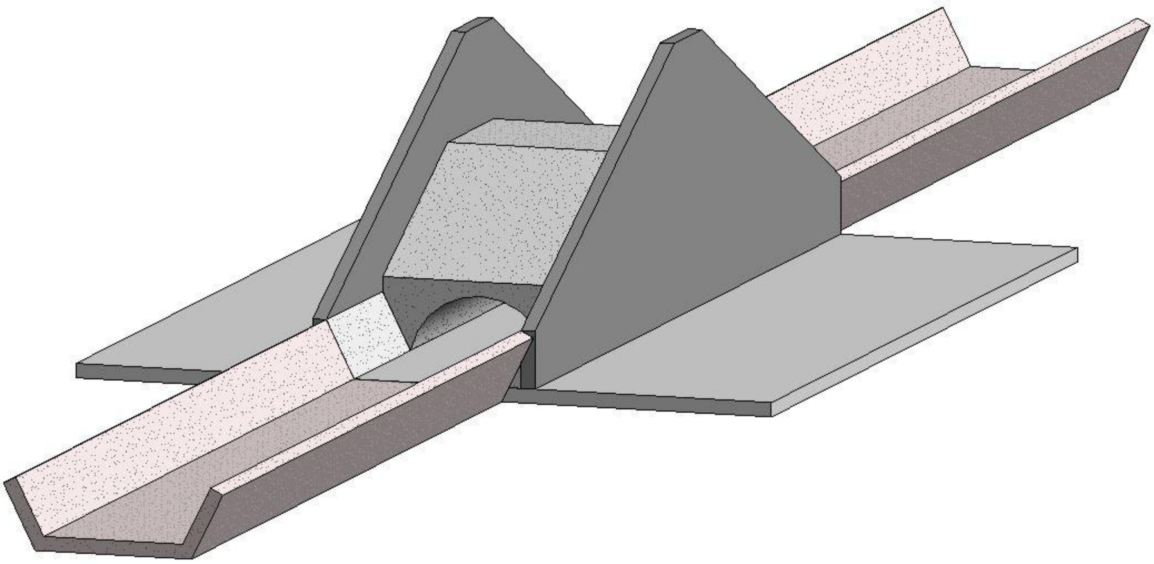


Figure 6.21: 3D view of the optimal culvert-spillway structure with arched culvert and trapezoidal spillway connected to the ZAC channel. The structure has two retaining walls and a foundation plate.

7 | Structural design

In Chapter 6, the global required dimensions of the structural components are determined. Based on these dimensions and general rules-of-thumb, a conceptual design is constructed. In this conceptual design, the reinforced concrete dimensions and governing load combinations are determined. From this, the strength and stability of the ZAC structure are evaluated.

Due to time constraints, only the use phase of the ZAC structure is evaluated based on stability and strength. Thus, for the construction phase, only a global construction sequence of the ZAC structure is determined. Furthermore, only one iteration of the conceptual design is documented. In practise, the design undergoes multiple iterations until an optimal design is determined which best satisfies the various requirements.

7.1 Construction sequence of the ZAC structure

The project execution consists of multiple steps. The sequence of all steps is shown in Appendix J. The steps can be summarized as:

1. Access to the project location is guaranteed by creating access ramps and an access road. Facilities are placed on site and space is allocated to materials and equipment.
2. Trees and bushes are removed from site. Then, the channel and the reservoirs are excavated. This soil is used to eventually create the outer dikes.
3. A temporary channel is excavated to keep water from flowing the culvert-spillway-structure that will be under construction. This is shown in Figure 7.1.
4. The structures at the up- and downstream end are created.
5. The outer dikes are constructed. Simultaneously, the temporary channel is connected to the up- and downstream structures.
6. The inner culvert-spillway-structures are constructed.
7. The inner dikes are constructed and attached to the structures, and the ZAC reservoirs are now finished.
8. The up- and downstream channel are excavated and scour protection is placed.
9. Fences are placed around the ZAC and all facilities are removed. The final result is shown in Figure 7.2.

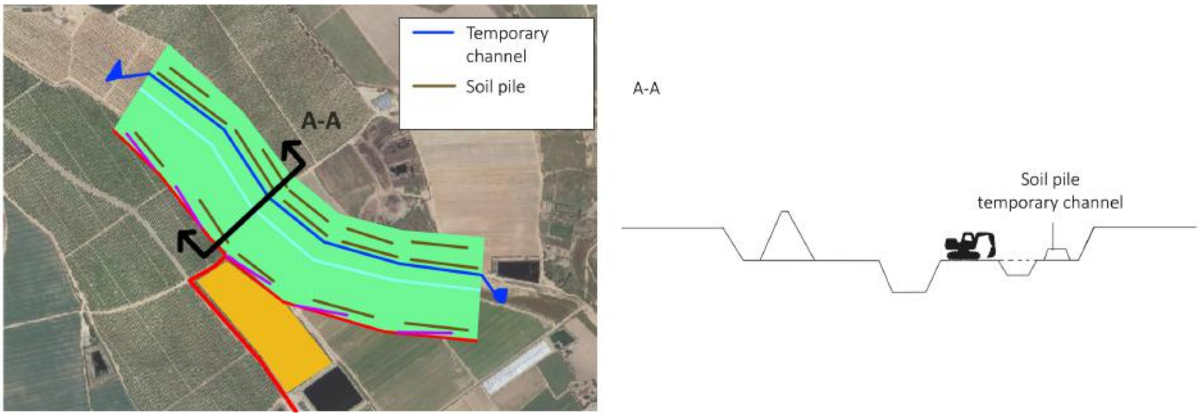


Figure 7.1: Left: plan view of the project area. Right: cross-section A-A as indicated in the plan view. The temporary channel is excavated, and its soil is placed in piles next to the channel. Initially, the temporary channel is not connected to the up- and downstream culvert-spillway-structure. These structures are not yet shown in this step, but are located at the up- and downstream end of the light blue line, which represents the main ZAC channel. The orange patch is space for facilities.

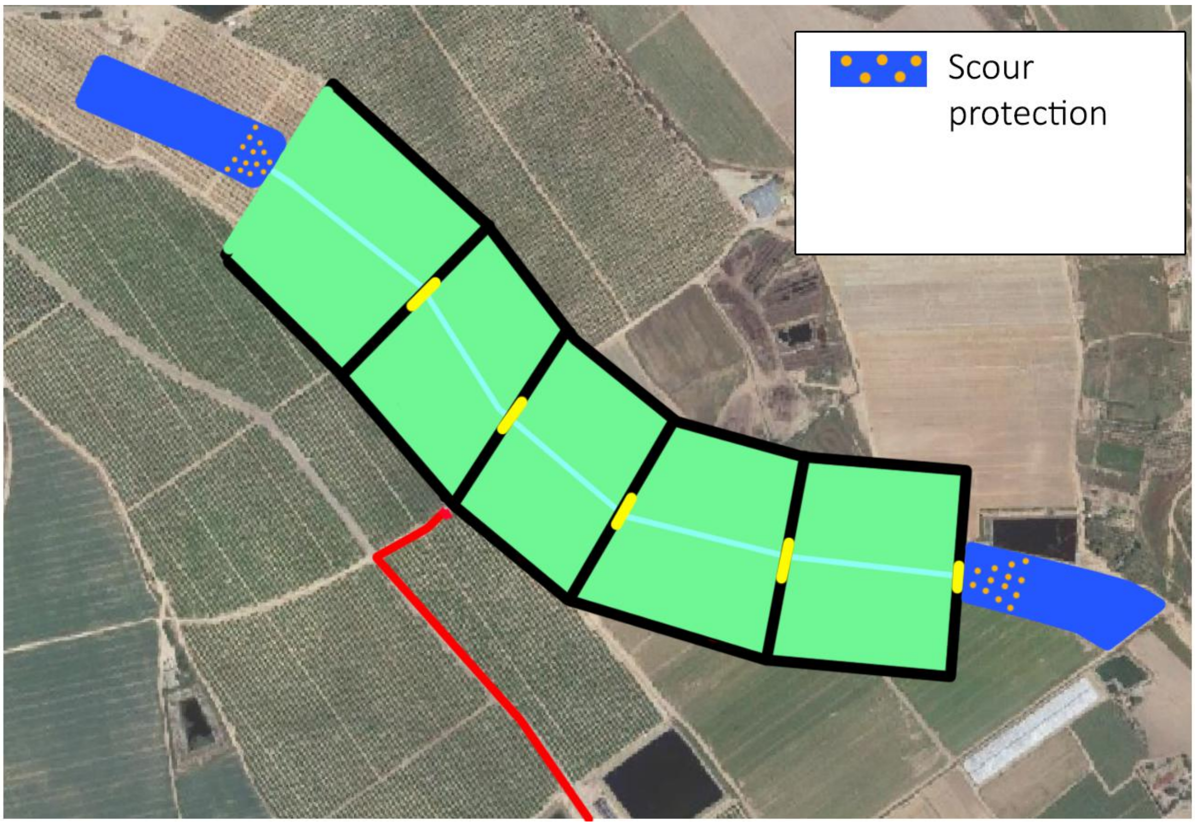


Figure 7.2: The final result is a ZAC structure with an up- and downstream channel. The black lines represent dikes and the yellow stripes represent the finished culvert-spillway structures.

7.2 Failure mechanisms of the ZAC structure

Prior to determining the conceptual design, the failure mechanisms for each element or system as part of the total structure are determined in this section. The failure mechanisms are used to determine the safe dimensions of the element. Each mechanism is further explained at the section where the element is checked.

Culvert-Spillway Structure

- Strength incapacity

Foundation

- Strength incapacity of foundation floor element
- Instability of structure
- Insufficient bearing capacity of soil

Retaining wall

- Strength incapacity

Dikes

- Macro stability (slip plane)
- Overflow
- Overtopping
- Piping

Channel bed

- Scour
- Stability

Other more detailed structural failure mechanisms are concrete damage due to too large flow velocities and steel reinforcement failure due to corrosion. The latter failure is covered by limiting the crack width of the concrete. A failure mechanism in the serviceability limit stage is when there is a lack of maintenance leading to sediment accumulation or poor conditions of the materials. This is not further elaborated in this report, since sediment transport and maintenance policy is not in the scope of this project.

7.3 Loads on the ZAC structure

In this section, a short description of the occurring loads is given. In the conceptual design, Section 7.5, the loads are elaborated and quantified.

Hydrostatic load
The water level on the two sides of the structure will generate a pressure that acts perpendicular to the structure.

Resulting force in culvert
Water will flow through the culvert and impose a resulting force in the flow direction based on the momentum balance.

Soil pressure
For the retaining walls, the soil is pressing on one side of the structural element, causing a pressure force in the horizontal direction. The soil pressure depends on the specific weight of the soil material and the height of the soil layer pressing on the structure. It is assumed that the soil pressure only acts on the retaining walls.

Self-weight
The weight of the structural components induces a vertical pressure on the substructures and also acts as a load on the structure itself. This has to be taken into account when designing the structural elements. Self-weight is the specific weight of the material of the structure multiplied by its volume.

Wind load
Wind pressure will cause forces acting on the structural elements. These wind forces will be determined by calculating the peak wind velocity. The wind force could be relevant for the stability of the spillway structure and the retaining walls during the construction phase. However, it is not included in the conceptual design of the structure, since it is assumed to be relatively small compared to the previously mentioned forces.

Wind set-up
The length of the reservoirs between the dikes is maximally 300-400 m, which is a small fetch length. For this reason, the wind-set is assumed to be negligibly small. Additionally, the ZAC structures are more land inwards, where wind set-up has a smaller effect in comparison with a coastal structure.

Wind waves
The waves generated by the wind also collide against the spillway-culvert structure, which induces stresses inside the reinforced concrete. However, it is assumed that the wind waves are relatively small compared to the hydraulic forces. Thus, the waves loads are neglected.

7.4 Governing load combinations on the ZAC structure

For the governing load combinations, the failure mechanisms for strength incapacity of the spillway-culvert structure and retaining walls are considered. The instability of the total structure and bearing capacity of the soil is elaborated in Chapter 7.6.2.

As indicated in Section 6.2.2, the inner dikes are numbered from 2 to 6, see Figure 6.4. It is also explained that there is a distinction between dikes 1, 2, 3, 4 and dikes 5 & 6, as the peak discharge is gradually decreased per reservoir. The first four dikes have a crest height of 7.5 m and the other two have a crest height of 7.0 m compared to the local basin level. Figure 6.9 gives an overview of this. Thus, to determine the governing load combinations, two dikes are considered, namely dike 2 and dike 5, as these dikes experiences the largest water level per dike group. The critical load combination for the design is based on two governing situations regarding the water levels in the system.

7.4.1 Load combination situation 1

The discharge through the culvert (Q_c) is at its peak level. In this case, one reservoir upstream of the structure is fully inundated, but the reservoir downstream of the structure is still in the initial phase of inundating, which means that the water level in the reservoir downstream of the structure is low relative to the upstream reservoir of the structure, see Figure 7.3. This is governing for the culvert because the flow velocities in the culvert are higher, leading to larger forces acting on it. This situation is also governing for the stability of the structure, since the forces coming from upstream are larger than the balancing forces from downstream.

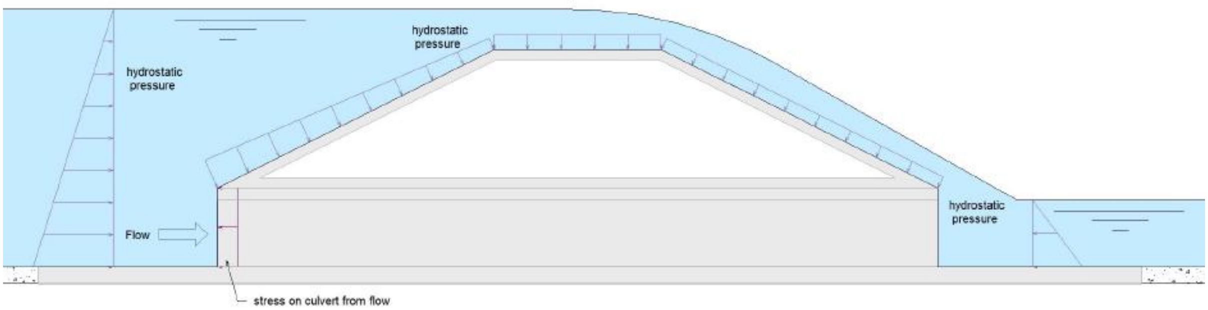


Figure 7.3: Governing load combination on ZAC structure in situation 1

7.4.2 Load combination situation 2

The discharge over the spillway (Q_s) is at its peak level and both reservoirs upstream and downstream of the structure are fully inundated. This is mainly governing for the retaining wall and the spillway, because of the maximal hydrostatic pressure downstream and upstream of the structure. It is assumed that situation 1 is governing for the stability of the structure. Thus, situation 2 is not considered for the stability verification.

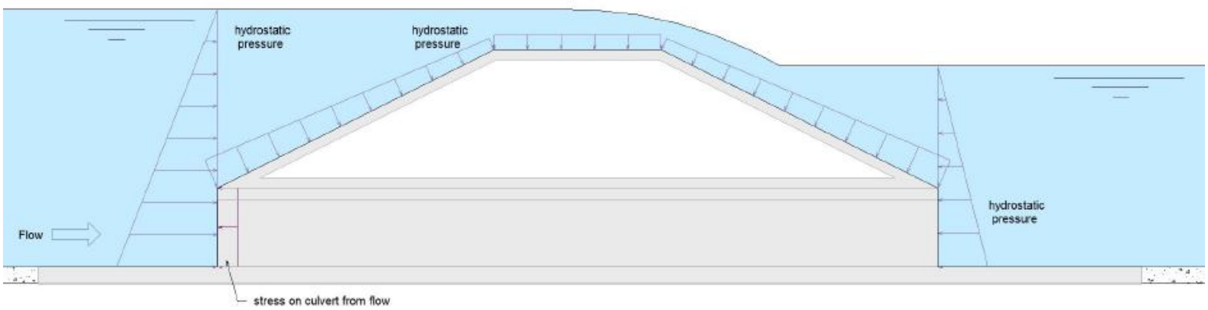


Figure 7.4: Governing load combination on ZAC structure in situation 2

7.4.3 Hydraulic characteristics for load situations

In Table 7.1 the hydraulic characteristics for the two dike types for each situation are given. These values are extracted from the 2D HEC-RAS model.

Table 7.1: Hydraulic characteristics dike 2 and dike 5

	Dike 2, 3 and 4				Dike 5 and 6			
	d_1 [m]	d_2 [m]	Q_c [m³/s]	Q_s [m³/s]	d_1 [m]	d_2 [m]	Q_c [m³/s]	Q_s [m³/s]
Situation 1	7.60	2.85	54.5	69.8	8.50	2.00	59.7	46.5
Situation 2	9.10	7.50	29.3	143.4	8.50	6.60	34.0	72.6

7.5 Conceptual design of the Culvert-Spillway structure

In the following section, a conceptual design is constructed for dike two and five. The calculated design for dike 2 is applied for dikes 3 and 4, and the design for dike 5 is applied for dike 6. The design of dikes 2 and 5 are determined per situation.

7.5.1 Assumptions for the conceptual design of the Culvert-Spillway structure

Before the ZAC structures are assessed with the previously determined governing load combinations, the reinforcement and the initial concrete thickness of the structural components need to be determined. But first, the main assumptions are made.

The reinforcement steel class B500B is applied for all the structural components. As an initial design, the concrete class for all structural components is assumed to be C35/45 and the reinforcement diameter is 32 mm. After the structural computation, the optimal concrete class and reinforcement are determined. For simplicity, it is assumed that the structural components are not prestressed.

Furthermore, based on Eurocode 1990+A1+A1/C2 the total structure falls under the consequence class CC2.

Consequence class : CC2

Reliability class : RC2

Load factor : K_{fi} = 1.0

Appendix M.1 gives an overview of the design parameters for the chosen concrete class and reinforcement class. These parameters are applied for both dike two and dike five.

7.5.2 Concrete element width

For an approximation of the initial concrete element widths, rules of thumb for designing a structure are applied. In Appendix M.2, the concrete element width per structural component is elaborated. Table 7.2 gives an overview of the chosen element widths.

Table 7.2: Initial elements width of the structural components

Structural component		**Initial element width [mm]**
Spillway	Slopped elements	300
	Upper slab	300
Culvert	Culvert roof (spillway bottom)	350
	Culvert beams	1000
Retaining wall		400

7.5.3 Concrete cover

For determining the concrete cover of the structural components, the method in Eurocode 2 is applied. The required thickness of the concrete cover, c_{nom}, is based on the minimum concrete cover, c_{min}, and the margin of construction tolerances, c_{dev}.

The minimum required concrete cover is the maximum of either the concrete cover based on the durability of the reinforcement, the concrete cover based on bonding or 10 mm.

Table 7.3 gives an overview of the determined required and chosen concrete covers. In Appendix M.3 the full elaboration for determining the nominal concrete covers is given.

Table 7.3: Required nominal concrete cover per structural component

Structural component	C_{min} [mm]	C_{dev} [mm]	Required C_{nom} [mm]
Spillway	37	5	42
Culvert	37	5	42
Retaining wall	37	5	42

Due to possible construction mistakes, a conservative concrete cover of 50 mm is chosen for all structural components. This is applied to all ZAC structures.

7.5.4 Determination of initial reinforcement for dike 5, situation 2

In this section, the initial design for the ZAC structure of dike two is made per governing load combination.

Quantification governing load combination

Hydrostatic pressure
The structure is subjected to hydrostatic pressure since it is submerged in water because of the high water levels in the reservoir. The hydrostatic pressure on the elements depends on the water level downstream, upstream and along with the structure. The hydrostatic pressure has a distribution which is dependent on the vertical water depth, which is calculated by:

$$p = \rho \cdot g \cdot d(z) \tag{7.1}$$

$p[kN/m^2]$ = water pressure at a certain point underwater

$\rho[kg/m^3]$ = density of water

$g[m/s^2]$ = acceleration due to gravity

$d(z)$ = water depth at a certain point in a vertical direction relative to the bottom of the water body

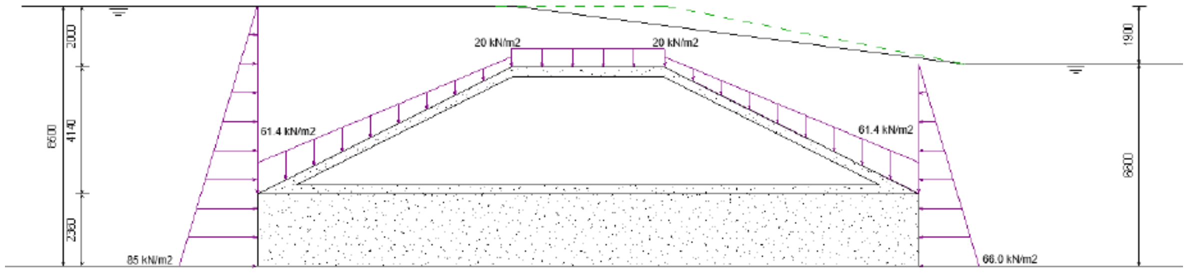


Figure 7.5: Hydrostatic pressure on ZAC structure for dike 5

The upstream parameters are invariable, where U_1 = 0 m/s and H_1 = 8.5 m. In Table 7.4, the values for the pressure at different locations are calculated with the use of the energy balance. The flow velocity is assumed to be 4.53 m/s which corresponds to the discharge through the culvert of 34 m³/s with a culvert opening of 7.5 m^2. The discharge through the culvert is extracted from the 2D HEC-RAS model.

Table 7.4: Pressure values at different locations of the culvert

	U_2 (m/s)	H_2 (m)	$p_i = \rho(H_2 - z_i)$
z=0 m	0.0	8.50	83.39 kN/m²
z=1.005m	4.53	7.45	63.25 kN/m²
z=2.010m	0.0	8.50	63.67 kN/m²

For simplicity reasons and to stay on the conservative side, the water pressure acting on the inside of the culvert, is assumed to be constant and equal to the largest value of the water pressure, which is 83.39 kN/m², see Figure 7.6

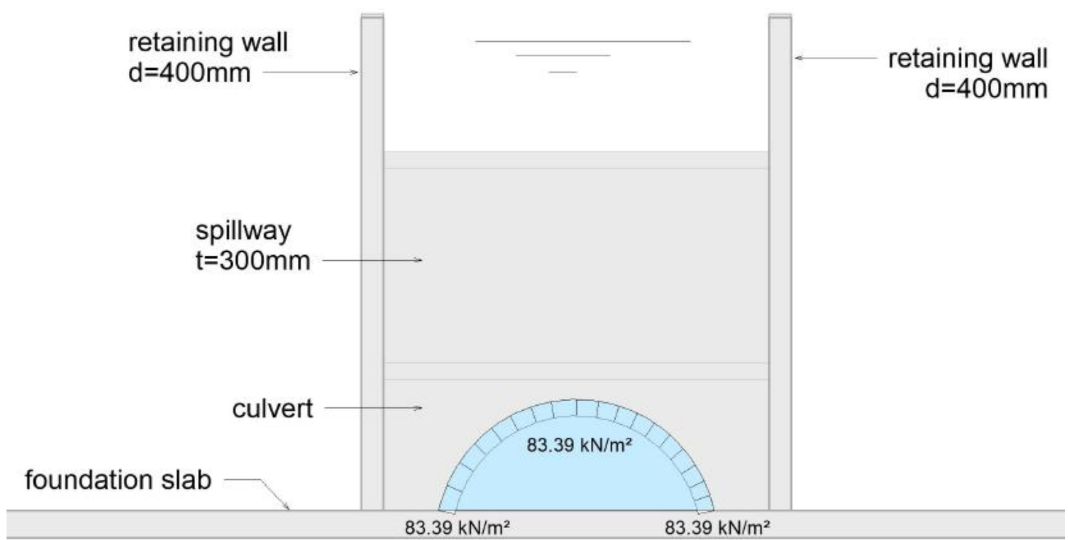


Figure 7.6: Water pressure in culvert for dike 5

Resultant force in culvert due to flow

The flow through the culvert imposes a resulting force in the culvert in the opposite direction. This can be determined with the use of a momentum balance. See Figure 7.7. For this calculation, the pressure inside and at the centre of the culvert (the highest velocity) is used, as determined in the previous section.

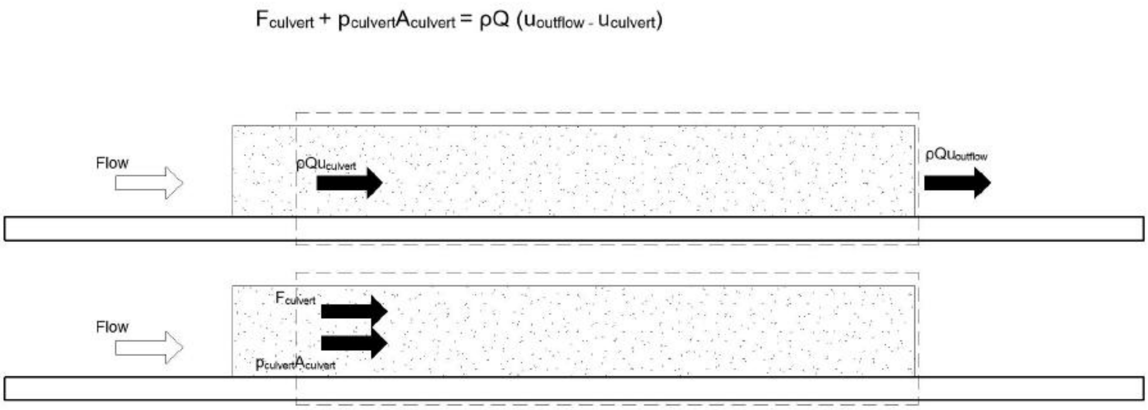


Figure 7.7: Resultant force in culvert

The velocities outside the culvert are extracted from the 2D HEC-RAS model. These are measured at a location just downstream of the culvert. The velocities and pressure in the culvert are calculated in the previous section. The input and result for obtaining the force in the culvert can be found in Table 7.5.

Table 7.5: Input and out for obtaining forces in the culvert

	Dike 5, Situation 2
$U_{culvert}$ [m/s]	4.53
U_{out} [m/s]	1.00
$P[kN/m]$	63.25
$Q[m/s]$	34.00
$F_{culvert}$ [kN]	4.53

Soil pressure
The surrounding dikes are supported by the retaining walls. Thus, the retaining walls are loaded by soil pressure. The total height of the dike is 9.0 meters.

As indicated in the Boundary Conditions, Section 3.3, the soil type is sand-greatly silty clayey. The groundwater table lies 68.8 m below the project location, therefore it is neglected. Due to no additional information about the soil, an average uniformly distributed soil is assumed over the total dike height. The assumed specific weight of this soil is gamma = 19 kN/m^3. It is also assumed that no objects are present in the vicinity of the structure, so the vertical soil pressure at the top of the dikes is zero.

The determination of the horizontal soil stresses are indicated in Appendix M.4. In Figure 7.8 the horizontal stresses for specific depths for dike 5 are given.

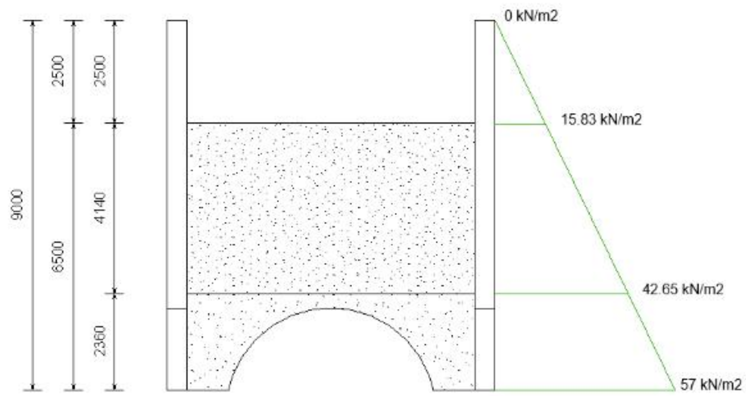


Figure 7.8: Horizontal soil pressure on the ZAC structure for dike 5

Governing load combination
In Figure 7.9 and Figure 7.10, the combination of all the loads acting on the structure is shown.

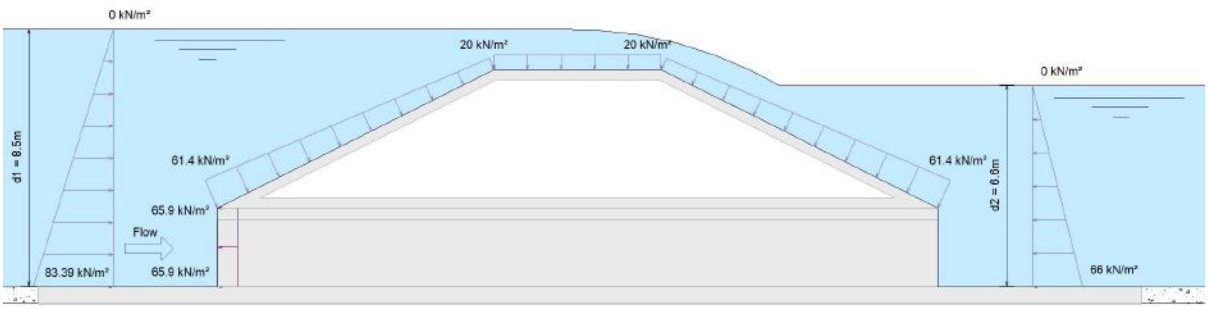


Figure 7.9: Governing load combination on ZAC structure for dike 5 in situation 2, side-view

This will be the input for the structural analysis in the FEM software DIANA. The method used to determine the previously mentioned loads is applied for all the following dikes. Thus, in the following section, only the main results are given.

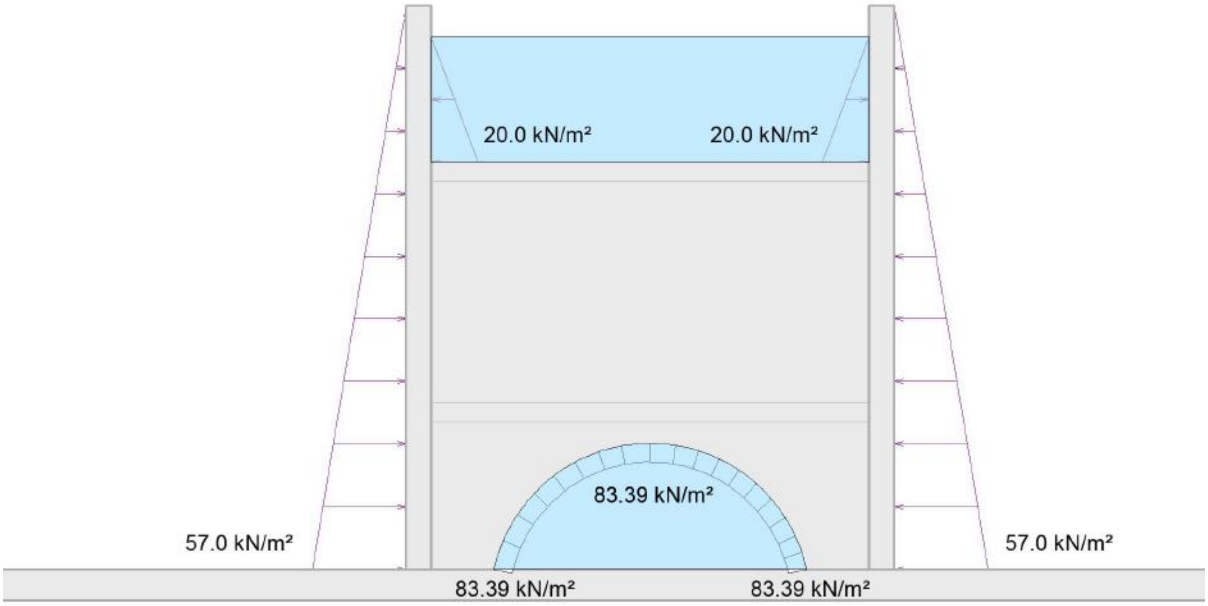


Figure 7.10: Governing load combination on ZAC structure for dike 5 in situation 2, front-view

Reinforcement

The initial design for the reinforcement is based on the method in chapter 35 of the Hydraulic structures Manual. The first approximation for the amount of reinforcement per structural component is based on its bending moments and tensile stresses. The acting loads per structural component are indicated in the previous section.

For simplicity, the loads acting on the structural components are simplified to loads per unit width. From this, the resulting loads are determined and subsequently the approximation of the reinforcement. This approach is reasonable since it is an initial design and the calculated reinforcement is examined in ULS. Furthermore, it is also assumed that there is no load transfer between the retaining walls and the spillway-culvert structure. The bending moments are determined with MatrixFrame.

This method for determining the reinforcement is also applied to all the remaining ZAC structures.

In Appendix M.5 the matrixFrame model per structural component is elaborated. Also, the governing bending moments with the corresponding required reinforcement of the structural components for situation 2 of dike 5 is given. An overview can be seen in Section 7.5.6.

7.5.5 Determination of initial reinforcement for dike 5, situation 1

As indicated in the previous section, only the main results of situation 1 for dike 5 are given.

Quantification governing load combination

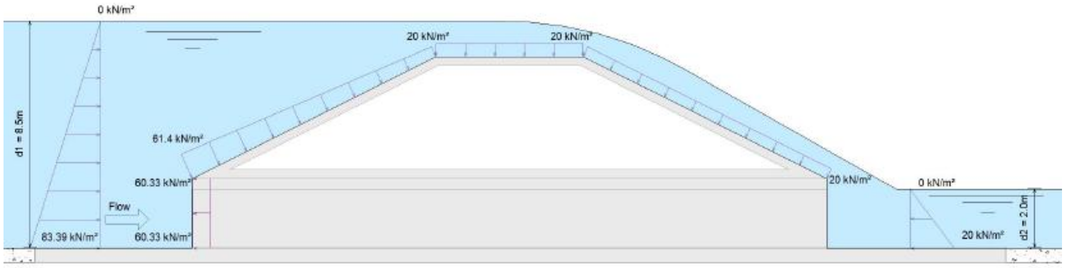


Figure 7.11: Governing load combination on ZAC structure for dike 5 in situation 1, side-view

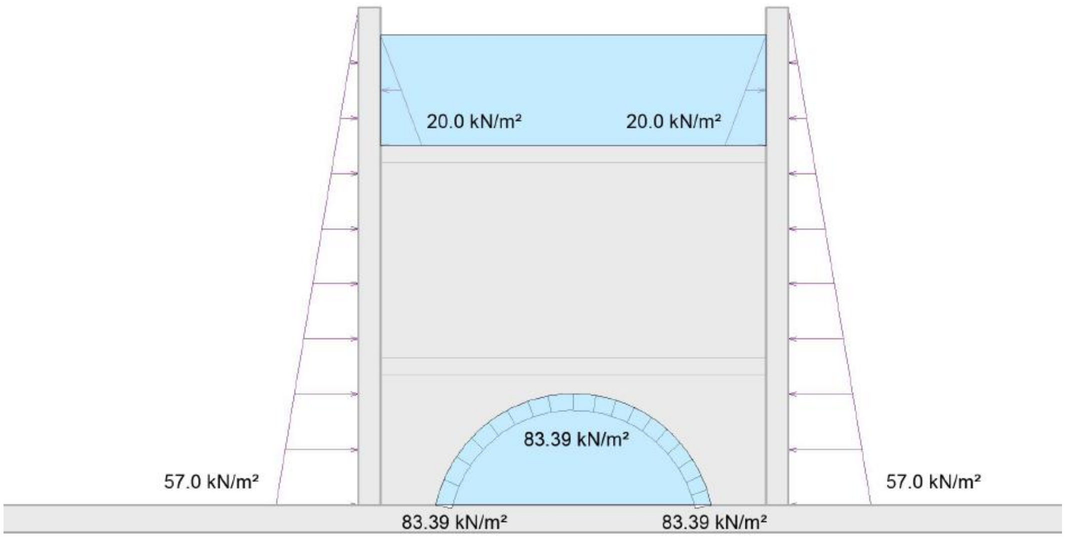


Figure 7.12: Governing load combination on ZAC structure for dike 5 in situation 2, front-view

Reinforcement

In Appendix M.5 the matrixFrame model per structural component is elaborated. Also, the governing bending moments with the corresponding required reinforcement of the structural components for situation 1 of dike 5 is given. An overview can be seen in Section 7.5.6.

7.5.6 Concept design dike 5

The following tables give an overview of the governing bending moments and required reinforcement per structural component of dike 5.

Spillway

Table 7.6: The governing bending moments and corresponding reinforcement per spillway element for dike 5. The values are per unit width.

Spillway element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2]
Dike 5 situation 2				
Top slab	-255	2097	0	0
Sloped slabs	-644	6163	298	2483
Bottom slab	-195	1330	195	1330
Dike 5 situation 1				
Top slab	-223	1816	0	0
Front Sloped slabs	-634	6036	276	2284
Back sloped slabs	-223	1816	112	884
Bottom slab/roof culvert	-163	1105	163	1105

Retaining wall

Table 7.7: The governing bending moments and corresponding reinforcement for the retaining wall for dike 5. The values are per unit width.

	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2]
Dike 5 situation 2				
Retaining wall	-77	448	55	318
Dike 5 situation 1				
Retaining wall	-72	418	50	290

Culvert

Table 7.8: The governing bending moments and corresponding reinforcement per spillway element for dike 5. The values are per unit width.

Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2]
Dike 5 situation 2				
Top slab (transversal direction)	-116	779	224	1537
Beams	0	0	224	518
Dike 5 situation 1				
Top slab (transversal direction)	-122	820	234	1609
Beams	0	0	234	541

Applied reinforcement
Based on the governing required reinforcement for dike 5 the applied reinforcement is determined. In Appendix M.5 the reinforcement spacing and placement is determined.

Spillway
A reinforcement net of $\phi 25 - 180mm$ is applied to all the elements. For the sloped element, additional reinforcement bars with a diameter of 32 mm with a spacing of 180 mm are added over a length of approximately 3.5 meters.

Culvert
The transversal reinforcement in the culvert top/spillway bottom is a net of $\phi 25 - 225mm$ .For the culvert beams, a net of $\phi 12 - 180mm$ is applied.

Retaining wall
A reinforcement net of $\phi 12 - 225mm$ is applied.

Transversal direction
According to NEN-EN 1992-1-1+C2 9.3.1.1 the reinforcement in the transversal direction should not be less than 20% of the reinforcement in the longitudinal direction. Thus, for simplicity, the amount of transversal reinforcement in all the elements, except that of the spillway bottom/culvert roof, is assumed to be equal to that of longitudinal reinforcement.

7.5.7 Determination of initial reinforcement for dike 2, situation 2

Quantification governing load combination

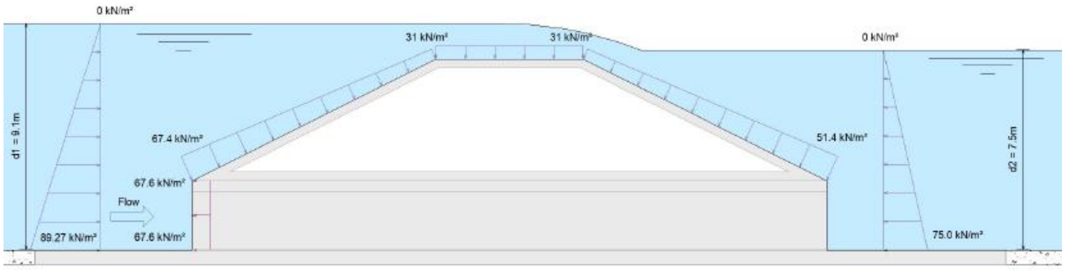


Figure 7.13: Governing load combination on ZAC structure for dike 2 in situation 2, side-view

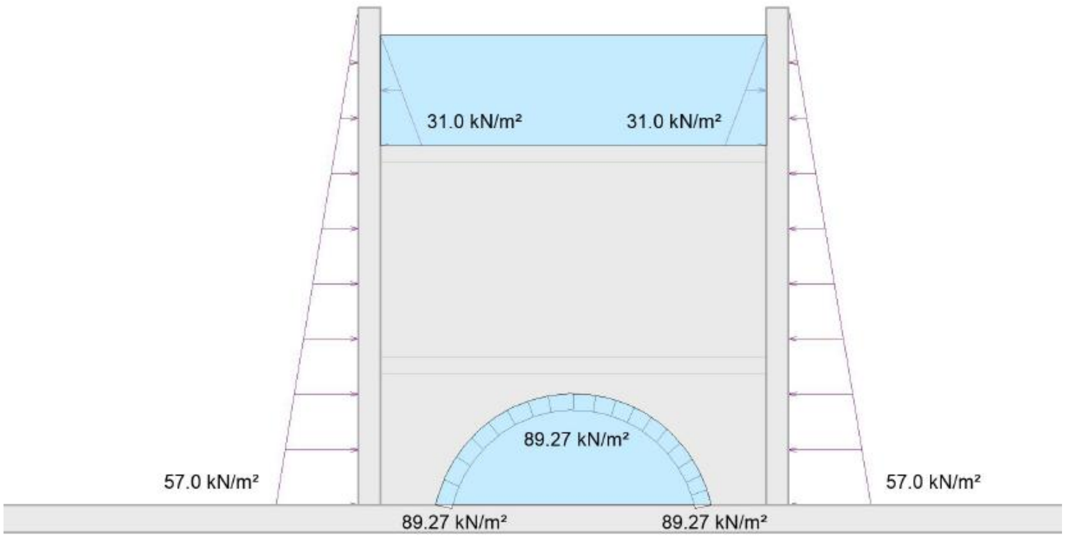


Figure 7.14: Governing load combination on ZAC structure for dike 2 in situation 2, front-view

Reinforcement

The same reasoning for all the MatrixFrame models of dike 5 are applied for dike 2. However, the dimensions and loading do differ. Furthermore, for dike 2, situation 2 is governing for all structural components. Thus, situation 1 is not included for determining the reinforcement.

In Appendix M.6 the governing bending moments with the corresponding required reinforcement of the structural components are given. An overview can be seen in Section 7.5.9.

7.5.8 Determination of initial reinforcement for dike 2, situation 1

Quantification governing load combination

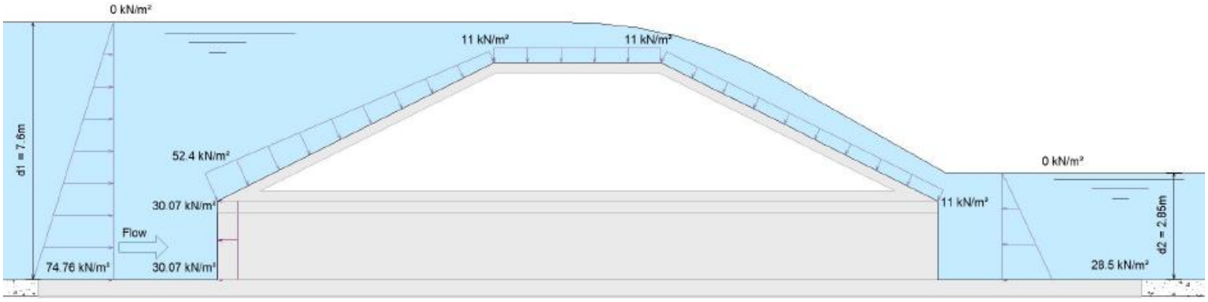


Figure 7.15: Governing load combination dike 2 for situation 2, side-view

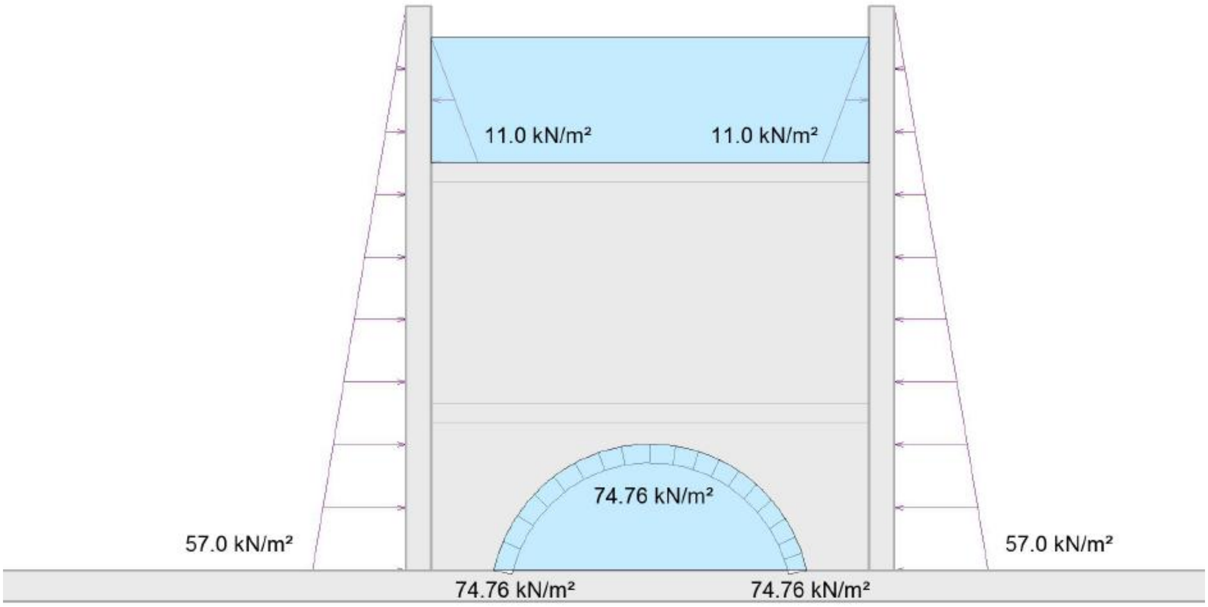


Figure 7.16: Governing load combination dike 2 for situation 2, front-view

7.5.9 Concept design dike 2

The following tables give an overview of the governing bending moments and required reinforcement per structural component of dike 2.

Spillway

Table 7.9: The governing bending moments and corresponding reinforcement per spillway element for dike 2. The values are per unit width.

Spillway element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Top slab	-236	1929	0	0
Front Sloped slabs	-545	4977	255	2096
Back sloped slabs	-545	4977	255	2096
Bottom slab/roof culvert	-173	1175	173	1175

Culvert

Table 7.10: The governing bending moments and corresponding reinforcement per culvert element for dike 2. The values are per unit width.

Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2]
Top slab (transversal direction)	-121	813	233	1602
Beams	0	0	233	279

Retaining wall

Table 7.11: The governing bending moments and corresponding reinforcement for the retaining wall for dike 2. The values are per unit width.

Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2]
Retaining wall	71	413	54	313

Applied reinforcement

Based on the governing required reinforcement for dike 2 the applied reinforcement is determined. In Appendix M.6 the reinforcement spacing and placement is determined.

Spillway

A reinforcement net of $\phi 25 - 225mm$ is applied to all the elements. For the sloped element, additional reinforcement bars with a diameter of 32 mm with a spacing of 225 mm are added over a length of approximately 3.5 meters.

Culvert

For the culvert beams, a net of $\phi 12 - 180mm$ is applied. The transversal reinforcement in the culvert top/spillway bottom is a net of $\phi 25 - 225mm$.

Retaining wall

A reinforcement net of $\phi 12 - 225mm$ is applied.

Transversal direction

The amount of transversal reinforcement in all the elements, except that of the spillway bottom/culvert roof, is assumed to be equal to that of longitudinal reinforcement.

The chosen diameters for the reinforcement of all the dikes do not correspond with the assumed diameter of 32 mm. This is not an issue, since conservative values for the concrete covers are selected.

7.6 Verification of the culvert-spillway structure

Based on the previously determined reinforcements, the structural components' ultimate limit state (ULS) are checked. For the ULS the reinforcement and concrete stresses are determined with the finite element program DIANA.

7.6.1 Strength verification of the structure (ULS)

Spillway-culvert structure

With the finite element program DIANA, the stresses in the elements and in the reinforcement steel are calculated. In Diana the structures are modelled with the chosen dimensions and thicknesses as mentioned in previous sections. The reinforcement steel bars are modelled as reinforcement sheets for which the diameter and spacing can be filled in, in the properties tab. After modelling the structure with right properties, the components are divided in sub-elements with a mesh size of 0.5m. This is the smallest mesh size that the educational version of the program is able to manage calculating the results. Furthermore, supports are attached to the structure. The culvert 'beams' and retaining walls are the line supports for the structure and transfer the load to the foundation. The loads are attached to the structure, such as indicated in Section 7.4 . From the analysis, the results of the stresses in the reinforcement and in the concrete elements follow. The extreme results are summarized in Table 7.12 and 7.13.

Results for dike 2 are not mentioned, since the results are similar to dike 5 and are also not governing.

Table 7.12: Minimum and maximum stresses in the concrete elements, where minimum indicates the compressive stresses and maximum indicates the tensile stresses.

	σ_{xx} (N/mm^2)		σ_{yy} (N/mm^2)		σ_{zz} (N/mm^2)		σ_{xy} (N/mm^2)		σ_{yz} (N/mm^2)		σ_{zx} (N/mm^2)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Dike 5 Situation 1	11.70	8.76	6.61	5.96	8.15	9.47	3.34	2.80	5.22	4.16	5.16	4.84
Dike 5 Situation 2	12.21	9.18	6.85	5.69	8.78	10.6	3.55	2.97	5.99	4.59	6.14	5.13

Table 7.13: Minimum and maximum stresses in the reinforcement steel, where minimum indicates the compressive stresses and maximum indicates the tensile stresses.

	σ_{xx} (N/mm^2)		σ_{yy} (N/mm^2)		σ_{zz} (N/mm^2)		σ_{xy} (N/mm^2)		σ_{yz} (N/mm^2)		σ_{zx} (N/mm^2)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
Dike 5 Situation 1	47.80	39.71	79.19	88.68	51.74	64.84	0	0	41.28	32.06	0	0
Dike 5 Situation 2	46.71	38.95	77.38	75.62	55.56	83.32	0	0	38.56	31.94	0	0

All resulting figures following from DIANA with stresses in x, y and z direction for dike 5 in the two situations can be found in Appendix M.

Resistance
As can be seen from Table 7.12 and Table 7.13 the stresses for dike 5 are lower than the strength of concrete and steel, which are 23.33 N/mm2 and 435 N/mm2 respectively. Thus, the chosen designs satisfy the strength (ULS) criteria.

Foundation
Due to time constraints, the strength criterion of the foundation is not elaborated in this report.

7.6.2 Stability verification of the ZAC structure

The soil mostly consists of sand material. Thus, it is decided to construct a shallow foundation. Since groundwater is neglected, the construction of an underwater concrete floor is not needed.

Only the main components are considered for determining the total weight on the foundation.

For the shallow foundation, the following criteria need to be satisfied:

- Soil bearing capacity
- Horizontal stability
- Vertical stability
- Rotational stability

The method for determining the previously mentioned criteria can be found in the Hydraulic Structures Manual. However, due to time constrains, these criteria are not elaborated in this report.

7.7 Conceptual design dikes

The dike dimensions are shown in Section 6.2.2. For these dimensions, the dikes are verified for the following failure mechanisms:

- Piping
- Overflow and overtopping
- Sliding inner and outer slope

7.7.1 Dike material properties

The dikes are constructed with the excavated soil on site. This requires information on the soil type. According to Jiménez-Martínez et al. (2012), the soil type is sand - greatly silty clayey. Actual data in or around the project location is not present due to the remoteness of the area. Therefore, sources are used that are assumed to be a rough estimate and a conservative approach has been applied. The distribution of silt, clay and sand in the total soil varies in Murcia. The ranges according to Wright and Wilson (1979) are shown in Table 7.14. The soil properties volumetric weight and friction angle are then taken from Eurocode 7 NEN-EN9997, and have to be verified by on-site soil investigation. This Eurocode does not mention a value for hydraulic conductivity k. The hydraulic conductivity of soil regions in Spain has been documented by Jiménez-Martínez et al. (2012) and is 0 - 10 mm/h. The value has been derived from Figure 7.20.

Table 7.14: Distribution of soil types in sand - greatly silty clayey from Murcia in Spain. Values taken from Wright and Wilson (1979).

Soil type	Range [%]
Sand	35-50
Clay	30-45
Silt	10-30

Table 7.15: Estimated soil properties of soil type at project location

Soil type	γ_{dry} $[kN/m^3]$	γ_{sat} $[kN/m^3]$	ϕ [°]	c $[kN/m^2]$
Sand - greatly silty clayey	19	21	30	0

7.7.2 Critical cross-sections of the dikes

The dikes are separated into three categories: inner dikes (2, 3, 4, 5), side dikes (7, 8) and begin and end dikes (1, 6). The last two categories separate the ZAC reservoirs from the surrounding ground level. The level of the crests compared to the outer ground level varies over the length of these dikes. Locations with the largest elevation difference on both sides are set to be the critical cross-section and are tested on the failure mechanisms. All the locations of critical cross-sections are shown in Figure 7.17 and the corresponding levels in Figure 7.18 and 7.19.

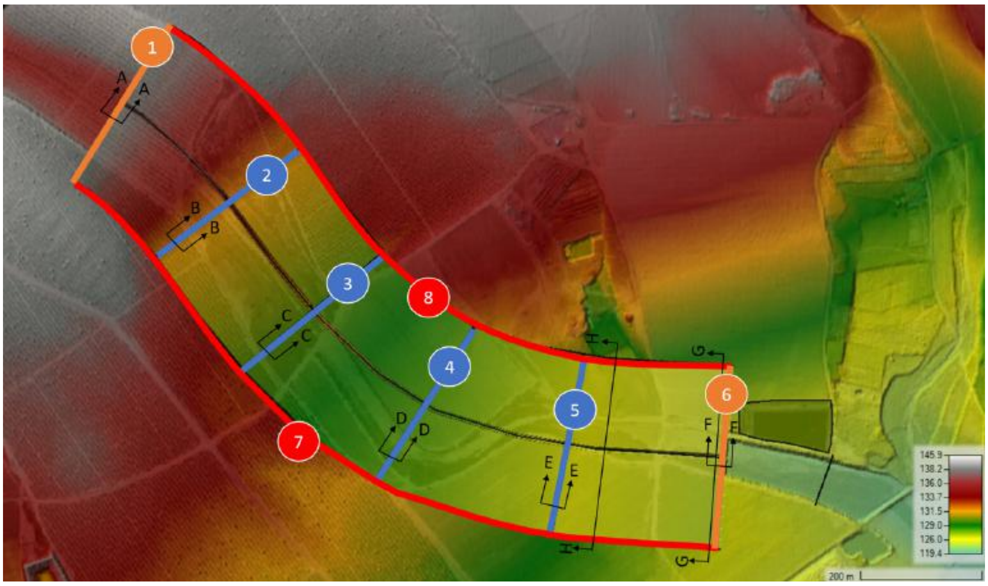


Figure 7.17: Critical cross-section map.

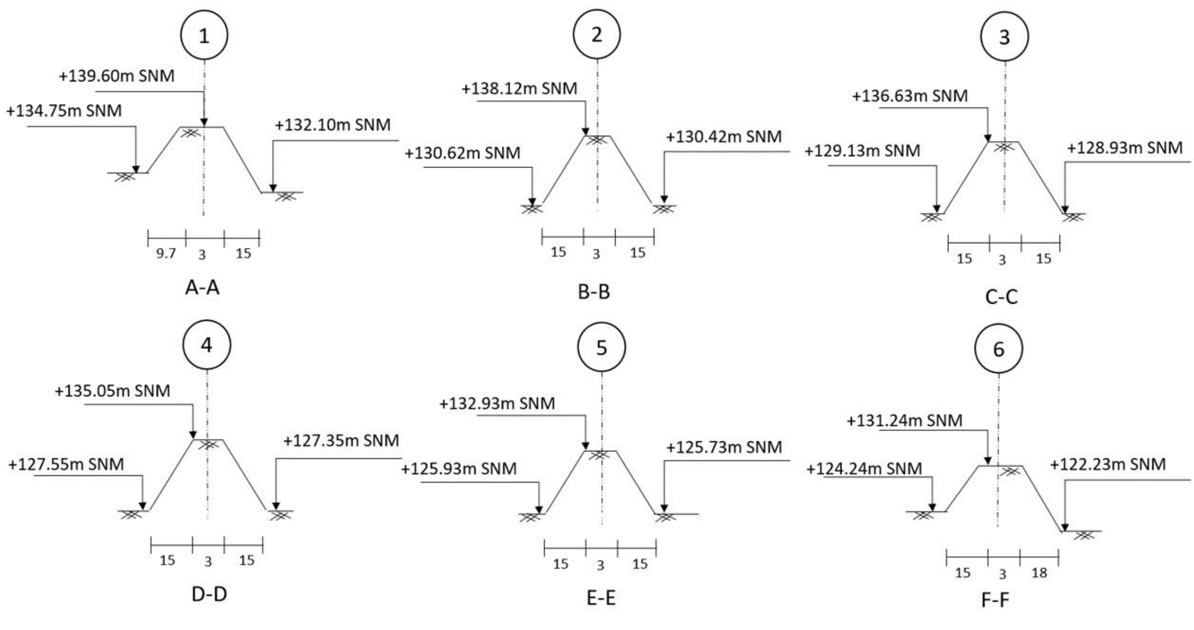


Figure 7.18: Critical cross-sections of dike 1, 2, 3, 4, 5 and 6.

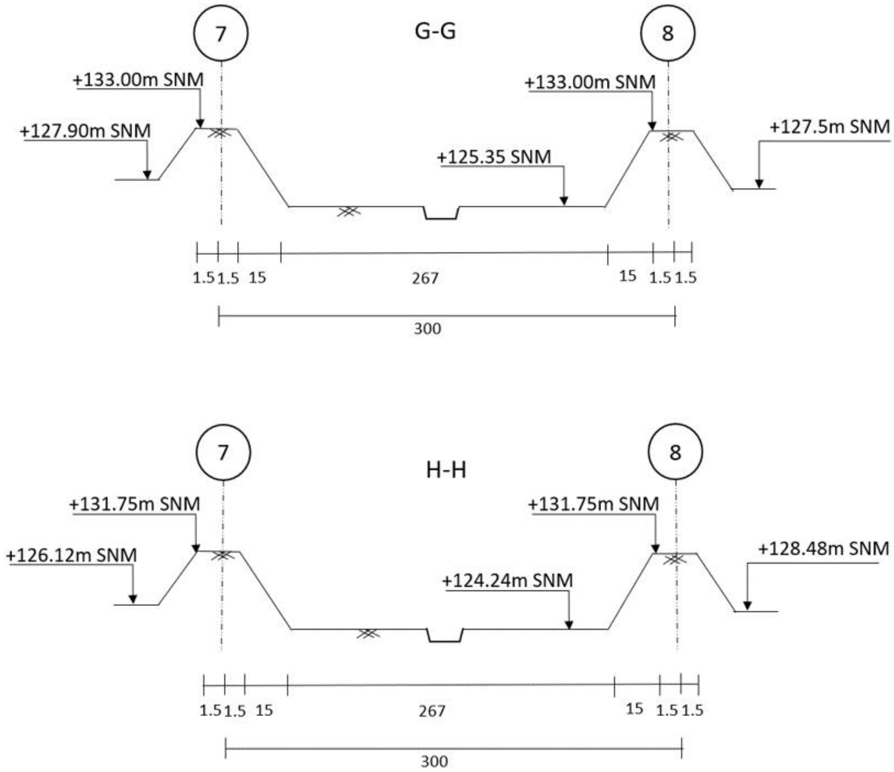


Figure 7.19: Critical cross-sections dikes 7 and 8.

7.7.3 Verification of flow and piping

By making the dikes out of the excavated soil, the failure mechanism of piping and flow through the dike must be checked. Eurocode 7 NEN-EN9997 does not mention a value for hydraulic conductivity k. The hydraulic conductivity of soil regions in Spain has been documented by Jiménez-Martínez et al. (2012) and is 0 - 10 mm/h. The value has been derived from Figure 7.20.

From the 2D HEC-RAS model, it follows that the flow gets out of the ZAC-channel banks at a certain time-step when it reaches a larger discharge than 51 m^3/s. When this happens, the toes of the dikes of the reservoirs get submerged in water. The total submerged time is the time between the ZAC basins are filling up and emptying again. The total time for the design rainfall event is 18 hours. The water then is able to penetrate the soil, so it must be checked whether through flow and piping is possible to occur. Despite the soil type being mixed and therefore having a relatively low permeability compared to sand, it is still being checked.

The length that the water is able to penetrate the soil is calculated with equation 7.2. The through flow is indicated with the arrows in Figure 7.21.

$$L = t * k = 0.18m. \tag{7.2}$$

Where:

- $t = 18$ hours.
- $k = 10mm/h$ in the most conservative estimation.

The dike is submerged in water on both sides, so the penetration in the worst case works from both sides.

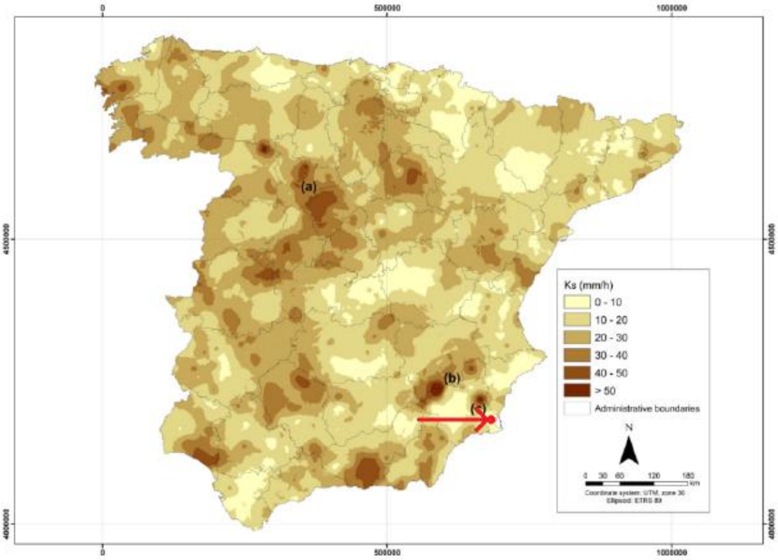


Figure 7.20: Hydraulic conductivity map from Jiménez-Martínez et al (2012). At the ZAC location, indicated by the red arrow, the hydraulic conductivity k is in the range of $0 - 10\ mm/h$.

Even if the water reaches the top of the dike for the entirety of the rainfall event, the total penetration length would be $2 * 0.18 = 0.36m$ which is smaller than the crest width of the dike of $3m$. This would not even be a realistic case, because from the start of the rainfall event till the end, the top is only submerged in water for a short duration. The toes (red arrows in the figure) are submerged for the entire time, but there the width between the toes is even larger, amounting to 33 m. However, since the top is safe during this time, the dike can be considered safe. Even if another large rainfall event would occur directly after the 474-year rainfall event, there would be enough protection against water penetration into the dike.

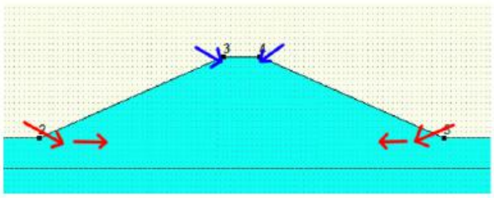


Figure 7.21: Cross-section of the dike. The crest width is $3m$. This is larger than the $2*0.18 = 0.36m$ of penetration that would occur during the whole submersion time of the dike.

The occurrence of piping is not calculated. This is because the duration of the submergence of the dikes is too short to be a danger. In a reference, research investigating the piping failure mechanism of a clay dike with similar dimensions as the current dikes on a sand foundation was tested, and the start of piping occurred after 100 hours in the worst case (Kramer, 2014). This was even on a sand foundation, which is much more permeable than the soil in this project (greatly silty clayey sand). In this project, due to a less permeable soil, the time before piping would occur is longer than the reference project. Since the duration of submergence of the dike is only 18 hours, the piping can be neglected.

7.7.4 Verification of overflow and overtopping

Overflow and wave overtopping are not to be considered for this design. The reason for this is that the excess water is discharged through a spillway, which is designed to prevent the dike from overflowing during design conditions. This means that the dikes does not need an armour layer. Wave overtopping is also neglected. The minimum difference between the crest of the dike and the maximum water level in front of the dike during the design rainfall event follows from the 2D HEC-RAS model and is around 20 to 30 cm. This is said to be enough, as a fetch of just 250 m will not create wind waves high enough to overtop the dike. This is also complemented by the fact that the ZAC designed to process a flow hydrograph, meaning that under design conditions, the maximum water level will only be present for a short period of time.

7.7.5 Verification of slope stability

The dikes have a crest level ranges from 7.0 to 8.0 meters compared to basin levels and a maximum slope width of 15 meters. Only dike 6 has compared to the outer ground level a crest level of 9 meters and a corresponding slope width of 18 meters. These dimensions are checked for stability via the method of slices and is performed via the computer program D-Geo Stability of the Dutch company Deltares. This method divides the soil mass above a circular sliding plane into vertical slices. For each slice, different soil parameters, effective stress an pore water pressures are determined (Deltares, 2019). Each dike profile is made out of the soil profile given in Table 7.15. It is stressed that the given soil parameters are a rough estimate and therefore the dike profiles are a very rough first design.

The inner dikes cross-sections (2, 3, 4, 5) are tested on the following load situation: upper reservoir filled combined with the lower reservoir empty. This is an overestimation of the load because the maximum water level is only reached when the lower reservoir is also partly filled. Due to the importance of no dike failure combined with the rough (not verified) soil data, this overestimation of the load is chosen. It results in a conservative preliminary dike design but after soil investigation the dimensions can probably be smaller, which increases the ZAC storage capacity and is favorable to reduce the impact of downstream floods. For the critical cross-sections of the begin (1), end (6) and side dikes (7 and 8) are two situations checked: filled and an empty reservoir. The following methods are used:

- Bishop
- Uplift Van

The Bishop method considers the driving moments due to the soil weight, loads and water pressures around the center of the circular slip plane. Stability is reached when the sum of driving moments is equal to the resisting moment due shear strength along the slip circle. By using the Bishop method, the safety is defined by the reduction factor that can be applied to the friction in order to reach the moment equilibrium (Bishop, 1955).

Next, the Uplift Van method is used to check for horizontal stability. Van's method assumes that the total slip plane is composed of two circular parts which are connected by a horizontal part. The safety factor is determined by using the equilibrium of the horizontal forces (Deltares, 1995).

Figures of the critical slip-planes are shown in Appendix L. The results are as followed:

Table 7.16: Safety factors slip-planes of dikes per cross sections

	Dike 1	Dike 2	Dike 3	Dike 4	Dike 5	Dike 6	Dike 7	Dike 8
Bishop safety factor	1.12	1.12	1.12	1.12	1.12	1.15	1.12	1.16
Uplift Van safety factor	1.79	1.79	1.79	1.79	1.46	1.31	1.77	1.93

7.8 Conceptual design of bed protection in the channels

Due to the high outflow velocities of the culvert, scour protection is needed in the main channel and at the outflow of the ZAC. At the upstream channel, water velocities are also increasing due to the steeper slope compared original river bed and bed protection is also needed. Three options are available for achieving a stable protection layer: geometrically closed, geometrically open and geometrically closed filters with geotextile. The base layer is described as sand - greatly silty clayey. Due to the lack of data available on this base layer, the following properties are assumed: $d_{50} = 180\,\mu m$ and $d_{85} = 300\,\mu m$. This is based on the available description of the soil layer. It is sand with clay and silt. So, the diameters will be smaller than pure sand. Because of the small diameter of the base layer, a geometrically open filter can already be ruled out. This is because the armour layer would have to become unrealistically thick in order to reduce the near bed velocity to prevent the base layer from washing away. First, a geometrically closed filter is designed. After that, it is checked whether a geotextile would have a positive effect on the layer thickness and amount of rock gradings that are used.

Armour size

From the HEC-RAS model, it followed that the maximum outflow velocity of the culverts is equal to $4.7\ m/s$ and the outflow water depth is equal to roughly $2.9\ m$. For the stone diameter of the armour layer, The following adaptation of the Shields formula is used.

$$d_{n50} = \frac{K_v^2 \overline{u}_c^2}{K_s \psi_c \Delta C^2} \tag{7.3}$$

The elaboration on this method is found in Appendix K. This method resulted in a d_{n50} of $37\ cm$, which corresponds to the $LM_A\ 60\ -\ 300\ kg$ rock class.

Geometrically closed filter design

For the filter layer to be geometrically closed, it must fulfill the three filter rules:

$$1)\ \frac{d_{15,F}}{d_{85,B}} \leq 5 \qquad 2)\ \frac{d_{60}}{d_{10}} \leq 10 \qquad 3)\ \frac{d_{15,F}}{d_{15,B}} \geq 5$$

When choosing a standard grading, it is assumed that the layer always fulfills filter rule 2. The filter layer is optimized to have as few layers as possible. Eurocode EN 13383-1 2013 was used for the properties of the available rock classes (EN 13383-1, 2013). This optimization is further elaborated in Appendix K. Figure 7.22 shows an overview of the final design of the filter layers.

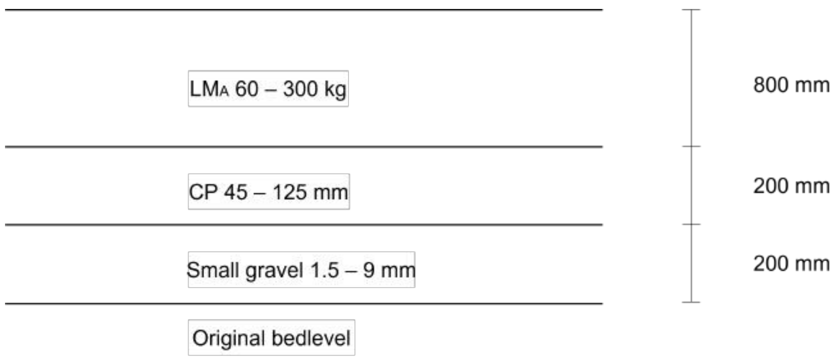


Figure 7.22: Overview of the geometrically closed filter used as bottom protection in the ZAC channel, downstream of the ZAC and in the upstream channel.

The total thickness of the filter is 1.2 m. This means that the channels have to be excavated 1.2 m deeper in order to maintain the design cross-section. A geotextile is said to be not preferable in this situation. This is because it will probably only replace the small gravel layer. The $CP\ 45/125\ mm$ layer is still necessary because the larger $LM_A\, 60 - 300\, kg$ rock class can damage the geotextile when placed directly on top of it.

Scour protection length

Downstream of the ZAC, the scour protection will have to extent far enough from the structure to minimize the chance of the structure falling into the scour hole. The following formula from Breusers & Raudkivi (1991) is used to calculate the depth of the scour hole. This is further clarified in Appendix K.

$$\frac{h_{se}}{2B} = 0.65\left(\frac{u_0}{u_{*c}}\right)^{0.33} \tag{7.4}$$

From this formula, it follows that the equilibrium depth of the scour is around $205\ m$. This large depth can be explained by the combination of a large outflow velocity and a small diameter of the bed material, though $205\,m$ is conservative. This also follows from the assumption of clear water scour from upstream and is probably not the case as such rainfall events will bring sediment from upstream. Furthermore, Breusers & Raudkivi (1991) also assume constant flow, which is not present as the out going flow will decelerate along the downstream river. In the case of this project, normal conditions represent a dry river. Breusers & Raudkivi (1991) state that the length of the scour hole is around 5 times as large as the depth of the scour hole. This would result in a scour protection length of around $1\ km$.

This scour protection is designed so that the velocity of the water is equal to or smaller than the critical velocity of the bed material at the end of the scour protection. This is a significant overestimation of the problem. It is reasonable to allow for a scour hole to develop downstream of the bed protection, on the condition that the scour hole is far enough away from the structure so that it does not compromise the stability of the structure.

For this design, no research is done on the sediment transport and therefore the length of the scour protection is based on the engineering judgement set to be equal to $100\ m$.

8 | Final design

The project location is located north-west of Balsicas in the region of Murcia in Spain. After an extreme weather event, floods may occur as the result of the dry river overflowing the riverbanks. At discharge events larger than 51 m^3/s, these floods occur, as this is the maximum discharge of the current river. By placing a so-called flood abatement control zone (ZAC) in the dry river upstream of Balsicas, the potential flooding is reduced for extreme discharge events. The design rainfall event has a return period of 474 years with a corresponding peak discharge of 207 m^3/s.

8.1 Proposed design

8.1.1 Location overview

The ZAC structure is placed around $2\ km$ upstream of the inlet of the already existing downstream channel near Balsicas. The ZAC has 5 reservoirs with each $250\,m$ in length by $300\,m$ in width and a total storage capacity of $2.06 * 10^6\ m^3$ over a total length of $1250m$. The implementation of the ZAC in the surrounding is presented in Figure 8.1. The location of the dikes in the ZAC are shown in Figure 8.2. Dike 2 up to 6 have a culvert-spillway structure to discharge water through the ZAC. An upstream channel connects the reservoirs with the upstream river bed. Water outflow velocities of the ZAC are up to 4.7 m/s and therefore bed protection in the channels and downstream area is implemented. The storage capacity of the ZAC is lower than the total volume of a once in 474 year discharge event and therefore downstream measures are taken. By increasing the inlet structure of the already existing downstream channel, the capacity of the downstream channel is fully utilized. This ensures that no flood will occur in the town of Balsicas.

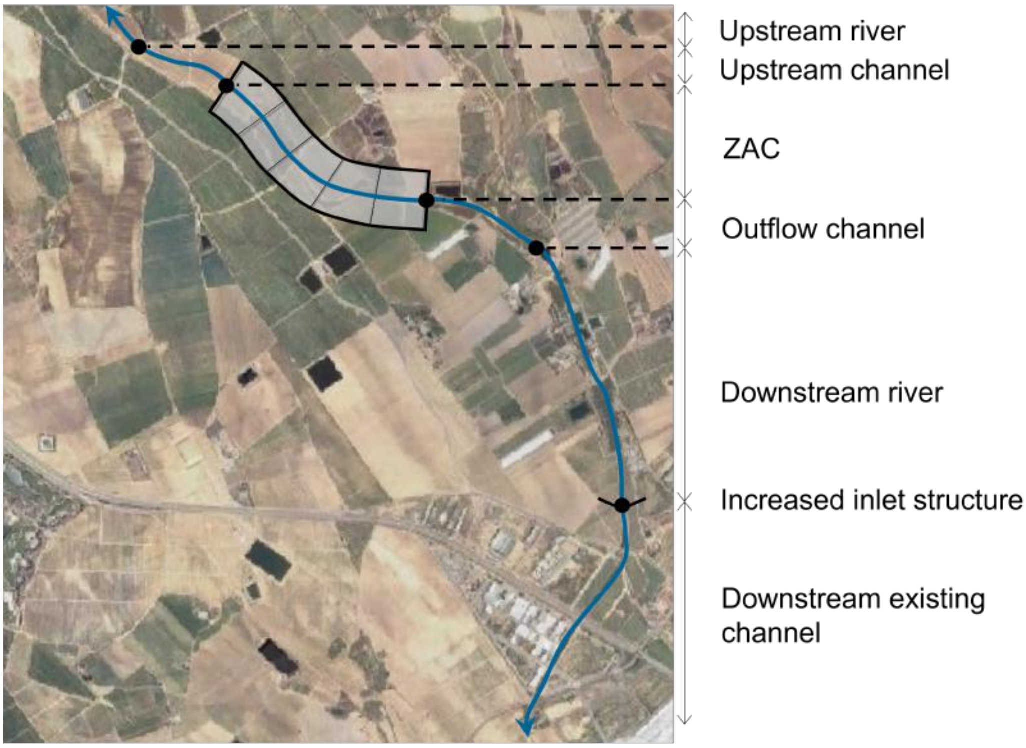


Figure 8.1: Overview of the location of the ZAC and rivers in project area

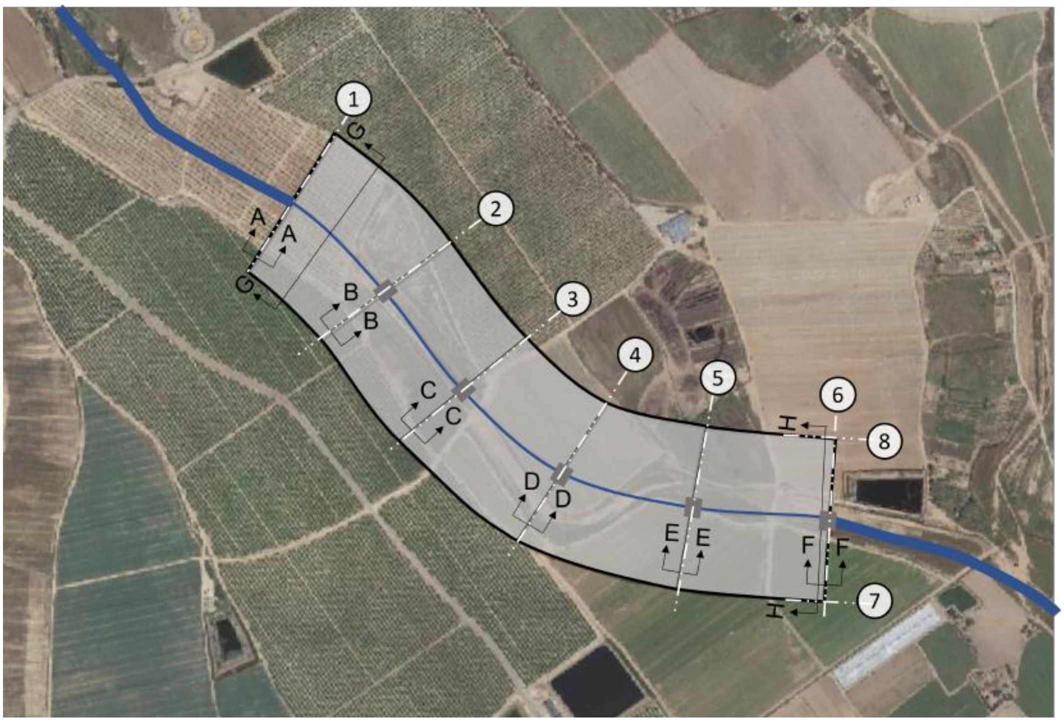


Figure 8.2: Close-up of the ZAC with cross-sections of the dikes and corresponding numbers

8.1.2 Integration of the ZAC in the project location

The ZAC reduces an incoming peak discharge of 207 m^3/s into 106 m^3/s for the design discharge event which occurs once every 474 years. At each reservoir the incoming peak discharge is partially reduced. This results in two different crest elevations of dike 1 until 6. Dike 1, 2, 3, and 4 have a crest height of 7.5 meters and dikes 5 and 6 have a crest height of 7 meters compared to basin levels. Therefore, the first 3 reservoir withstands a larger peak discharge and have a larger storage capacity compared to the lower two reservoirs. The increase of the upstream storage capacity is also created by implementing a milder slope ($i_{up,zac} = 0.0063$) for the ZAC compared to the river. The slopes of the river upstream ($i_{up,riv}$) and downstream ($i_{down,riv}$) are approximately 0.007 [-]. To connect the reservoir to the upstream river, an upstream channel is implemented with a slope of $i_{up,ch} = 0.008[-]$ and has a length of 1450 meters.

The outflow velocities of the ZAC is at maximum 4.7 m/s and therefore 100 meter bed protection is placed at the outflow with stones class of $LM_A\ 60-300kg$ at the top layer. Downstream of the protection is the current riverbed untouched, which means the current agricultural use can still be used. At 2 kilometers downstream the ZAC outflow, a funnel shaped inlet structure of the channel is constructed. The structure is 1.0 meter in height and 120 meter wide under a angle of 45 degrees. This structure guides the 106 m^3/s peak flow to the already existing channel to prevent floods in Balsicas.

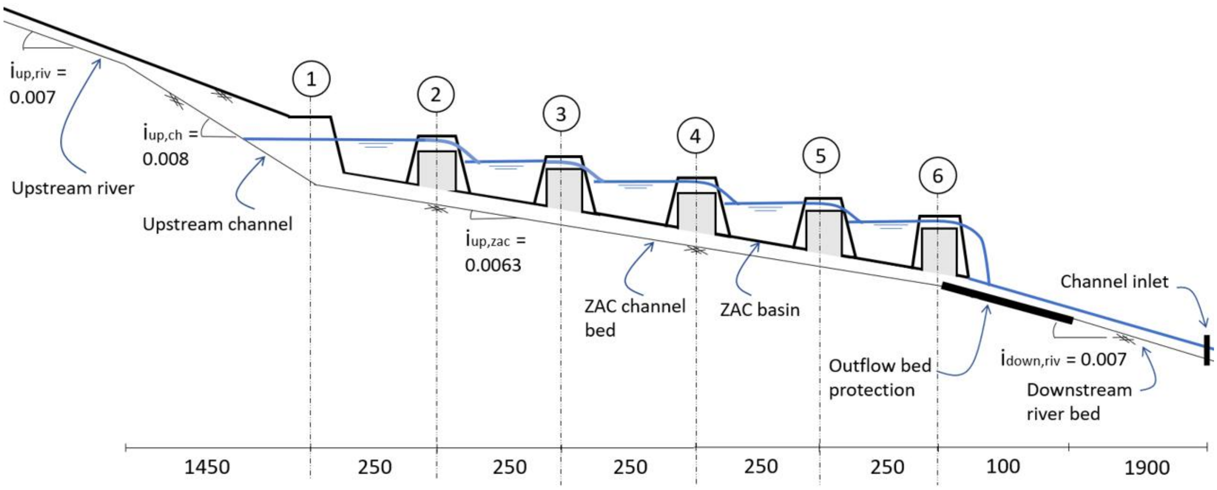


Figure 8.3: Cross-section along the water flow from upstream river to downstream river. The upstream river is connected by a upstream channel to the ZAC reservoirs. The begin (1), inner (2,3,4,5) and end (6) dikes are shown with the culvert-spillway structure (grey). Water flows over the spillway and goes through the culverts to the downstream channel with 100 meter bottom protection. After 1900 meter it enters the current existing channel through the increased channel inlet.

8.1.3 Cross-sections of the dikes and channel of the ZAC

Figure 8.4 shows the critical cross-sections of dike 1 to 6 including the local elevations. These are the cross-sections at the locations with the largest elevation differences compared to the crest level. Dike 1 to 4 have a crest level of 7.5 meters above the local basin level. For dike 5 and 6 is a crest elevation of 7.0 meters sufficient to withstand the water elevations in the reservoirs. Side dikes 7 and 8 have a gradually decreasing crest elevation compared to local basin level and goes from 8.0 meters upstream to 7.5 meters downstream. All the dikes have a slope of approximately of 1:2. The main channel in the ZAC has a base width of 5 meters, a top width of 7 meters and a depth of 2 meters. At main channel, upstream channel and at the outflow, bed protection is placed as shown in Figure 8.6b.

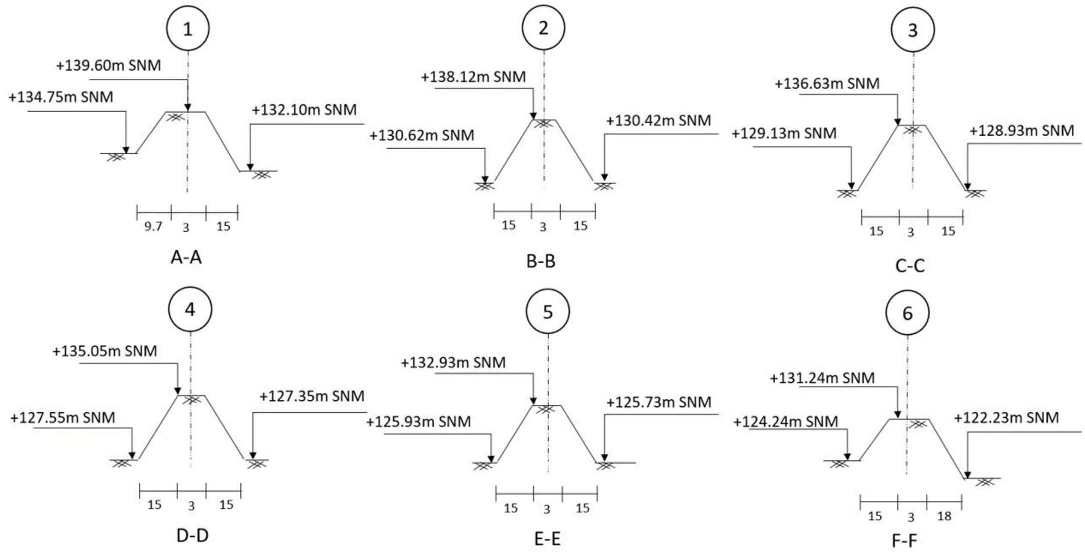


Figure 8.4: Critical cross-sections of dikes 1 up to 6. Local ground levels are shown and the width of the slopes.

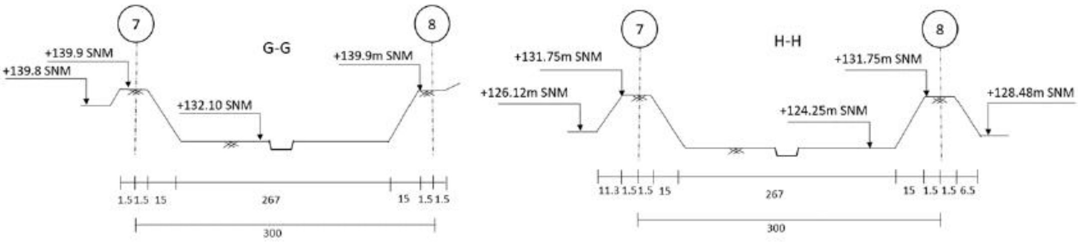
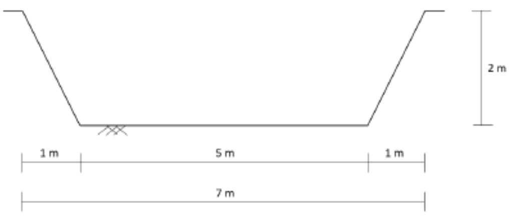
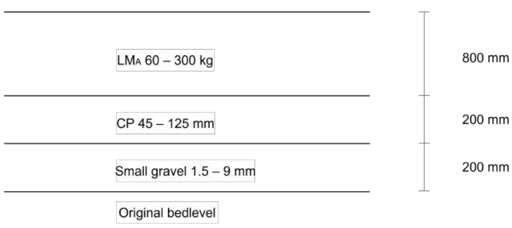


Figure 8.5: Cross-sections G-G (upstream) and H-H (downstream) of dikes 7 and 8.



(a) Cross-section of the main channel in the ZAC.



(b) Geometrically closed filter used at the ZAC.

Figure 8.6: Final design of the main channel in the ZAC

8.1.4 Culvert - Spillway structure

A culvert-spillway structure is implemented at dike 2, 3, 4, 5 & 6. This structure provides the connection between the reservoirs and regulates the flow. The culvert dimensions are designed to provide a maximum flow of 51 m^3/s as this is the maximum discharge downstream. It result is that the ZAC structure does not interact with the water flow during normal conditions. By larger discharges than 51 m^3/s the reservoir will fill and eventually the water level reaches the spillway. Each reservoir reduces the incoming flood wave and therefore the storage capacity of the 3 reservoirs upstream are that larger than the 2 reservoirs downstream, which results in two types of culvert-spillway structures. Whereas the culvert dimensions are the same at each dike, is the spillway bottom level lower at the last 2 downstream reservoirs compared to basin level. All dimensions and structural elements of the culvert-spillway structure are shown in the Figures 8.8 and 8.9.

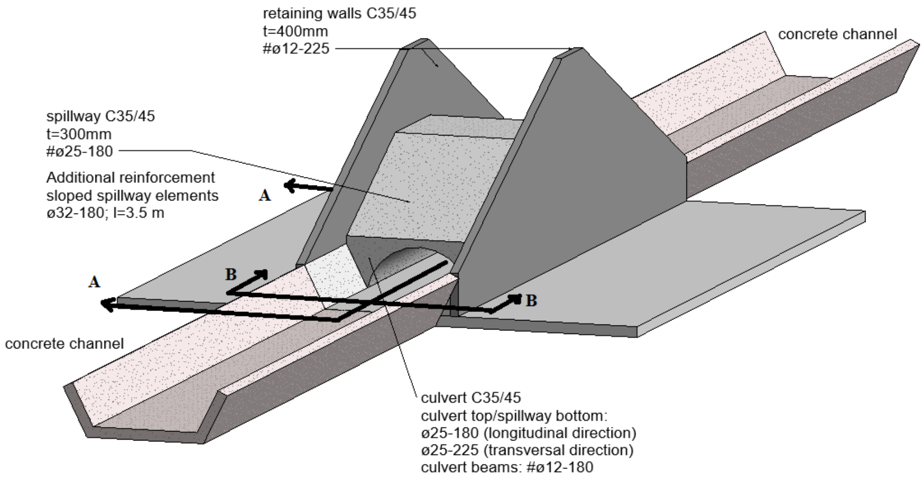


Figure 8.7: 3D summary of structural requirements of the elements and the necessary reinforcement steel for dike 5. These elements are set the same for the structure in dike 6.

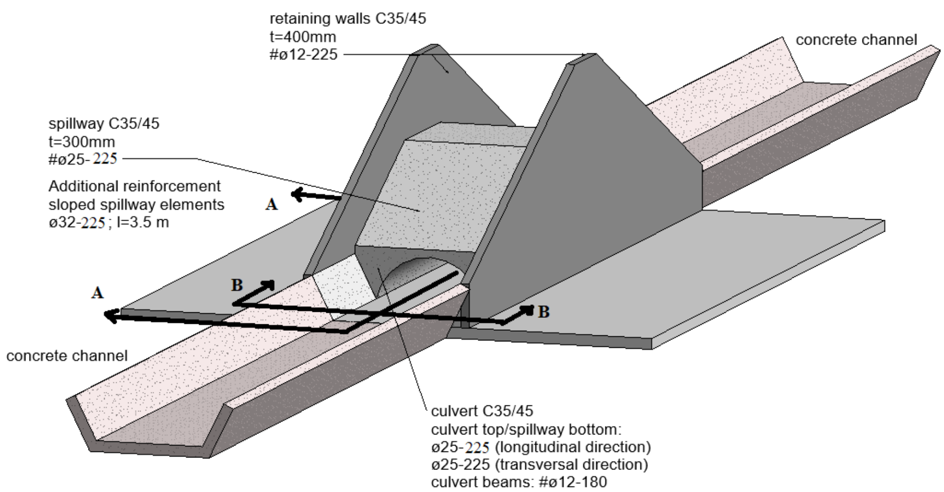
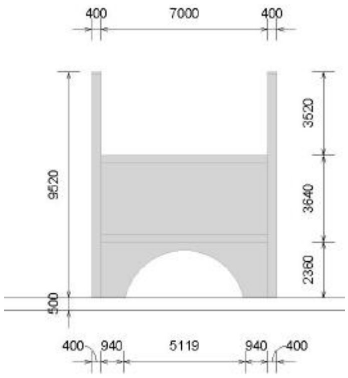
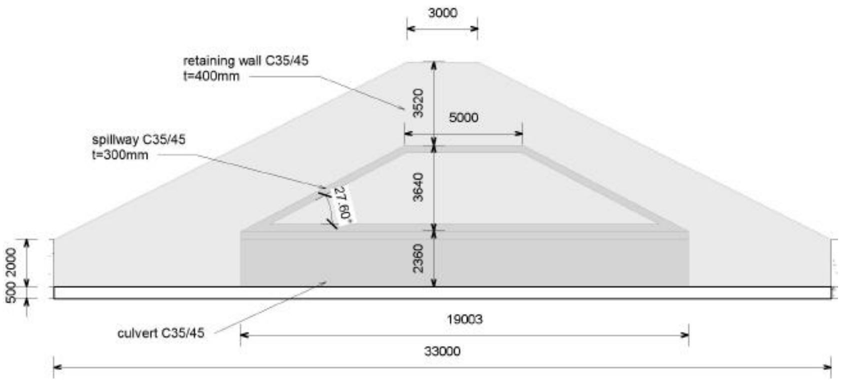


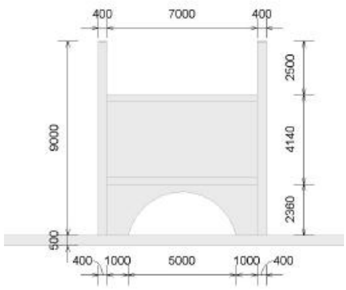
Figure 8.8: 3D summary of structural requirements of the elements and the necessary reinforcement steel for dike 2. These elements are set the same for the structures in dike 3 and 4.



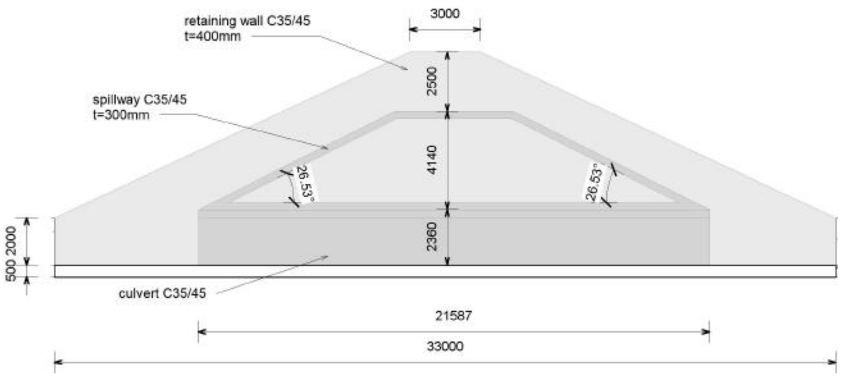
(a) Front view (B-B) of the culvert-spillway structure of located in dike 2,3 & 4.



(b) Longitudinal cross-section (A-A) of the culvert-spillway structure of located in dike 2,3 & 4.



(c) Front view of the culvert-spillway (B-B) structure of located in dike 5 & 6.



(d) Longitudinal cross-section (A-A) of the culvert-spillway structure of located in dike 5 & 6.

Figure 8.9: Cross-sections A-A and B-B of the culvert-spillway structure at dike 2, 3, 4, 5 & 6.

8.2 Sources of uncertainty

During the design, many assumptions have been made to serve as support for design choices. Many of the assumptions are rational and straight forward, but they do not offer a 100% guarantee that the designed structure will fulfill the design criteria. There are multiple sources of uncertainty that could cause the ZAC to have different dimensions than the current calculated dimensions. Therefore, the list below identifies which sources of uncertainty are present in the project. Furthermore, the impact of the uncertainty is mentioned, followed by how to quantify the uncertainty and what measures could be taken to minimize the uncertainty. It is stressed that the methods are discussed, but the actual numbers have not been investigated.

Acceptable probability of failure on ZAC structure
In Section 3.1.2 the design life time and the accompanying probability of failure is given, supported by the derivation of the values in Appendix 8.4. The table that is used is normally applied to breakwaters and sea dikes. Since no information could be found, the acceptable probability of failure has been assumed to be the same for this area too. This may differ for inland reservoir guidelines. It could also be that the governmental decision-makers impose a different acceptable probability of failure during the lifetime, leading to a design based on a rainfall event with a larger or smaller return period. This would affect the required storage volume, the required downstream discharge capacity and the dimensions of the complete ZAC and accompanying structures.

To minimize this uncertainty, one could check with governmental agencies what the range of acceptable probability of failure is, or find more accurate guidelines on probability of failure of inland flood defences.

Soil type
In Section 7.7, dikes are designed using an assumed composition of soil. Based on scientific sources, a range of soil percentages has been determined, but the exact composition of the soil at the project location is not known. This affects the unit-weight γ, the friction angle and therefore the dimensions of the required inner and outer dikes. Besides, in reality the soil is never completely homogeneous, especially in a project area of approximately $0.38\ km^2$. This uncertainty was accounted for by taking conservative values for the soil parameters and larger load conditions. The size of the uncertainty remains unknown until soil samples are taken and/or CPTs are performed on multiple locations on site. This allows to minimize the uncertainty at the same time. However, the heterogeneous properties will always cause a base level of uncertainty.

Scaling the hydrograph to different return periods
In addition to the acceptable probability of failure, the validity of the size of the maximum discharge given by the design rainfall event is uncertain. This is because the data set for the discharge values during rainfall events provided by Bermejo (2022) in Appendix Figure A.3 only contains 3 rainfall events. When logarithmic interpolation was used to get the maximum discharge value of the design rainfall event, it was not known how accurate the interpolation was. The size of uncertainty could be identified if more maximum discharge values of different return periods were given.

2D flow model outcomes
Generally, in modelling it is known that the outcomes of a model are only representation of reality, but

they are still useful. In the 2D flow model, the major sources of uncertainty are:

- The model has not been calibrated by other data sets or other locations. this induces an error in the outcomes of the flood map depths. The only validation was done using images of an other flood map of the same area. This is not a quantification.
- The velocity calculated in the model is depth-averaged everywhere. This can give skewed results regarding the outflow velocity of a culvert inside the ZAC. The water depth of the basin downstream of this culvert can be around 6 meters, While the maximum outflow velocity of the culvert only takes place in the bottom 1.5 meters. The velocity plot of the HEC-RAS model can then give much lower results than in reality.
- There can always be inconsistencies in the computational grid. This can mean that the grid might be too course on various locations, or that a high ground is not perfectly modelled.
- The applied Manning values on areas of the terrain can be not detailed enough. One can make a Manning land cover layer as detailed as the resolution of the original LIDAR data set. For this project, distinctions were made on the level accuracy of cultivated areas, orchards and urban areas.
- Differences with the used local terrain with the actual current terrain. New constructions might have been build in the time between the LIDAR data was obtained and the time the data was used, thus missing out on possible important details.

Some of these errors could be mitigated if the set-up of the model was done by a more experienced modeller.

Presence of rare species
Since this is an initial design of application of the ZAC, where the actual construction is not yet a realistic proposal, the involvement of rare species is neglected. However, when design is more detailed and actual construction is more realistic, contractors are advised to get in contact with nature organisations in an early stage. Then, a research of the location could be conducted to see if endangered species live on either location A or B. This could be financed by the contractor.

8.3 Conclusion

The measures that are advised based on this report are a ZAC structure with five reservoirs, accommodating a total storage volume of $2.06 * 10^6\ m^3$, and an increased inlet structure at the entrance of the already existing downstream channel, as stated in Section 8.1.1.

Figure 8.10 shows the result the ZAC has on the peak discharge reduction in the form of a hydrograph. The original hydrograph (dark blue) has a peak discharge of 207 m^3/s and by constructing the proposed desing of the ZAC, the peak is reduced to 106 m^3/s (green). Furthermore, the duration of the hydrograph is increased from 13 hours into 20 hours.

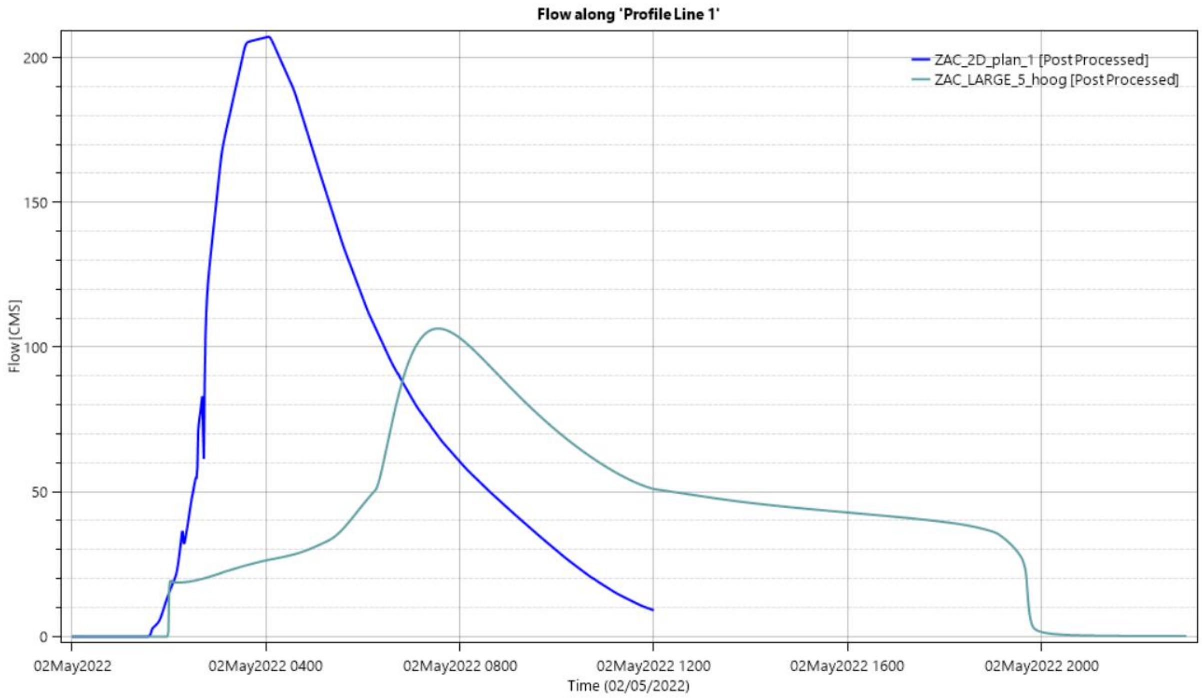
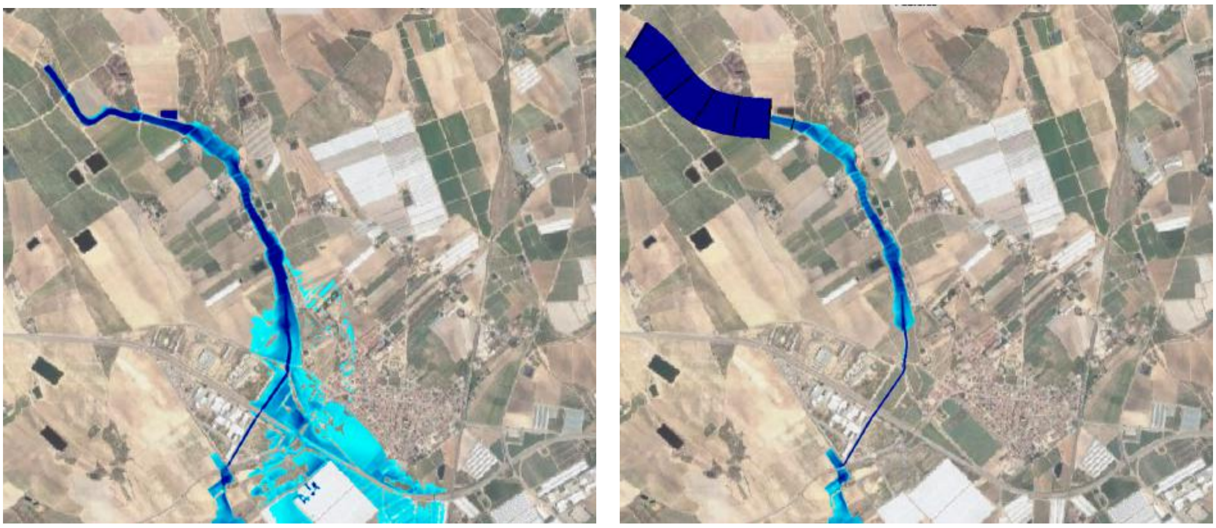


Figure 8.10: Resulting hydrograph of the situation without a ZAC (dark blue) and with a ZAC consisting out of five reservoirs (green). The duration of the new graph is increased and the peak is decreased.

Figure 8.11 shows the result as a flood map for both the situation without any measures and with the ZAC and inlet structure modelled. The situation without the ZAC shows a flooded area in the town Balsicas. By constructing the ZAC and increasing the inlet structure of the existing channel,future floods are prevented for rainfall event up to once every 474 years.



(a) Situation without any measures

(b) Situation with a ZAC structure and an increased inlet structure at the existing downstream channel

Figure 8.11: Comparison with and without measures.

8.4 Recommendations

The information about this design is aimed to be as extensive as possible. However, due to time constraints and lack of expertise in areas such as economic optimization and species richness in the environment, the design could have been optimized better. Therefore, the following list of recommendations discusses what improvements could be made or what things could be investigated more thoroughly.

Construction plan and cost analysis
Due to time constraints, it was not possible to create a detailed construction planning. This planning would indicate the duration of the construction phase. The planning could be subdivided into project planning and design, site preparation and execution. Furthermore, the duration of the execution, the required number of workers, the duration and amount of rented equipment and the material costs must be analysed to gain insight in the total costs of the construction of the ZAC. These two analyses help to add detail to the entirety of the project.

Economic optimization of number of reservoirs
In this design the costs of the construction of a dike and a spillway is not considered. To economically optimize the costs-benefits of the structure, an extensive risk analysis could be made to investigate the different levels of safety for rainfall events with different (large to small) return periods. Subsequently, the risks of those rainfall events happening could be plotted against the damage costs that are at risk for the specific rainfall event. To this end, an extensive analysis on the economic value of the town of Balsicas that are at risk to get damaged by the rainfall event. Then, the total expected costs per year can be calculated by the investment costs and the risk per year times the value of the properties at risk. This investigation could result in the optimal number of reservoirs in the ZAC.

Larger project area
The area upstream of Balsicas has been investigated. The problem of flood areas extends to the whole Murcia region, so a next step in the series of safety measures would be to investigate the effect of the ZAC further downstream. In the downstream area there are more confluences, tributaries and bifurcations from other local catchments. It is important to investigate the functionality of the ZAC in the network of ZACs as explained in Section 1.2. In addition, an investigation could be started about implementing a network of channels in combination with ZAC structures tom improve the drainage of the area towards the coast.

Bed protection downstream of the ZAC
The bed protection downstream is determined using the method Breusers & Raudkivi. This method determines the equilibrium depth of a scour hole in constant clear water flow, while the application of the ZAC takes place during rainfall events. This means that the used method is not completely accurate, and a better suitable method could be applied for this parameter. Therefore, this could be optimized.

Location of the ZAC
The client ordered that the ZAC is being placed on location A or B. Therefore, it might be beneficial to look at more locations than only these 2 and investigate whether the design of this ZAC could be fit better into the existing terrain.

The 2D flow computational method
The computational method that is used in HEC-RAS software to calculate is the 2D diffusive wave approach. However, in order to get more accurate results, it is possible to use a more detailed computational method. The 2D Saint Venant Full Momentum computational method is an example. This computational method is applicable for:

- Dynamic flood waves with rapid rise and rapid fall
- Flows with sudden expansion or contraction and accompanying high velocity changes
- Detailed flow solutions around hydraulic structures and obstacles

All three of these applications are present in the ZAC project. The trade-off of higher accuracy of the Saint-Venant equation is that this method requires a smaller timestep to yield a stable solution. This would increase the time per model run, and this could be of interest when one requires a solution for the flood depth map with more precision.

Validation and calibration of the 2D flow-model
Common modelling practise is the calibration and validation of model set-ups. Calibration is not performed, as there is only 1 more ZAC structure in existence and this has been constructed by a private company. This company has not publicly shared the data on this ZAC. In this design, the model has only been validated qualitatively. A more extensive research could be performed on the accuracy of the flow model, by comparing the results of the 2D flow model with the model data from the Centro de Descargas - the Spanish national institute of geographic information. Then, the difference in the results could be quantified.

Optimization of structural elements' dimensions
The structure can be designed in a more slender way, such that the structural efficiency of the material is higher. This leads to higher stresses in the elements and thus higher unity checks. This can be achieved

by choosing a smaller thickness for some elements and adjusting the reinforcement in such a way that some parts have more reinforcement and some parts have less reinforcement. This In practice this is labour-intensive, so structures are usually designed in such an exact way. This project is a preliminary design of the ZAC structure. When applying the DIANA model, the number of iterations to redesign elements in the spillway and culvert could become very high while not yielding much extra result. The dimensions of the materials can therefore be optimized even further than they are right now. However, the function of the ZAC is to offer safety, so a slight over-dimensioning of elements only strengthens the ZAC.

Sediment transport
The negligence of sediment transport is a significant simplification of reality. The sedimentation of sediment in the ZAC would reduce its storage capacity of water. This might be a disadvantage of the excavation of the ZAC. The ZAC might start to act like a giant sediment trap and regular maintenance might be necessary. A more detailed study including sediment transport could give insight on the functionality of the ZAC in the future and the accompanying floods near Balsicas. This could be important for knowing about what areas are prone to erosion and which flood areas could be closed off by the deposition of sediment. Especially as sediment transport is high during flood events, it is important to take it into account. Therefore, the next step in the design of the ZAC structure would be to include sediment transport in the flow model.

In case sediment transport is included, the ZAC will act as a giant sediment trap, because the ZAC is partially dug into the ground. It is not attractive to construct the ZAC on top of the existing soil, because then the ZAC would stick more out of the surroundings. Therefore, it could be investigated how much sediment is being deposited during extreme rainfall events. Subsequently, it can be calculated how much the ZAC storage volume would be reduced. This sediment volume must then be deposited in an extra reservoir. This extra reservoir could be constructed upstream to act as a sand trap, so the ZAC keeps its water storage capacity throughout its lifetime. The other option of constructing the ZAC on the existing ground level requires its own model in HEC-RAS so this could also be evaluated.

References

Anejo, J. A. (2014). *Impact of Concrete, Steel and Timber on the Environment: A Review.* International Journal of Technology Enhancements and emerging engineering research.

Ariens, E.E. (1993) *Relatie tussen Ontgrondingen en Steenstabiliteit van de Toplaag (in Dutch).* M.Sc. Thesis, Delft University of Technology

Arvanaghi, H., & Oskuei, N. N. (2013). *Sharp-crested Weir Discharge Coefficient.* Journal of civil engineering and Urbanism.

Avigueras Rodríguez, A. (2022). *Zones of flood-abatement control (ZAC) project*. Universidad Politéchnica de Cartagena, Cartagena, Department of Civil Engineering. [PowerPoint slides].

Azimi, A. H., Rajaratnam, N., & Zhu, D. Z. (2013). *Discharge Characteristics of Weirs of finite crest length with upstream and downstream ramps*. Journal of irrigation and drainage engineering.

Bishop, A. (1995). *The Use of Slip Circle in the Stability Analysis of Earth Slopes*. Geotechnique 5: 7-17

Breusers, H.N.C., Raudkivi, A.J. (1991) *Scouring.* Balkema, Rotterdam

CivilGEO Inc. (2022, January 21). *HEC-RAS 2D Flow Area Modeling | CivilGEO. Knowledge Base on CivilGEO.* Retrieved June 1, 2022, from https://knowledge.civilgeo.com/knowledge-base/hec-ras-2d-flow-area-modeling/

Deltares. (2019, June). *D-GEO Suite Stability.* Delft. Retrieved June 1, 2022, from https://www.deltares.nl/app/uploads/2019/06/UserManual.pdf

EN 13383-2, 2013. *Armourstone - Part 1: Specification.* 1st ed. Brussels:BSi

García Bermejo, J. (2022). *Return period and design discharges of ZAC*. Universidad Politéchnica de Cartagena, Cartagena, Department of Civil Engineering.[Excel]

Instituto Geológico y Minero de España. (n.d.) *MAPA GEOLÓGICO DE LA COMUNIDAD AUTÓNOMA DE LA REGIÓN DE MURCIA.* [map].

Jiménez-Martínez, J. et al (2012, March). *A 3D geological model of Campo de Cartagena, SE Spain: Hydrogeological implications*. Departament d'Enginyeria del Terreny, Cartogràfica i Geofísica, Universitat Politècnica de Catalunya. Instituto Geológico y Minero de España, Murcia Office. DOI:10.1344/105.000001703

Kramer, R. (2014, October 9). *Piping under transient conditions - Investigation of time-dependent erosion under dikes.* University of Twente. Department of Water Engineering and Management. Twente.

Materials & Environment-Sustainability group. (n.d.). CIE4100: *Integrated sustainability the Triple-Green design and build concept.* Delft.

Martínez, J.G. (2020, January 29). *Las consecuencias de las precipitaciones asociadas a la DANA en el Campo Cartagena.* La Oficina de Planificación Hidrológica. Ministerio para la Transición Ecológica y el reto Demográfico. Gobierno de España. Confederación Hydrográfica del Segura, O.A. Sociedad Española SEPREM de Presas y Embalses. Retrieved from: https://www.seprem.es/ponencias/JESUS%20-%20SEPREM%20Inundaciones%20Campo%20de%20Cartagena.pdf

Ministerio de medi ambiente, (2015). *Caracterizatión adicional de las masas de agua subterránea en riesgo de no cumplir los objetivos medioambientales en 2015.* Spain. Ministerio de medio ambiente. https://www.chsegura.es/export/sites/chs/descargas/servicios/informacionpublica/decriesgoccartagena/docsdescarga/EstudioDeclaracionRiesgo.pdf

Molenaar & Voorendt (2020). *General lecture notes hydraulic engineering*. (Edition 2020 ed.). Delft University of Technology. Delft. Faculty of Civil Engineering and Geosciences. [Lecture notes Delft University of Technology]

Murcia Today. (2019). *What is a Gota Friá; Never underestimate the destructive power of the rain in Spain.* Murcia Today News. [News article]. Retrieved March 10, 2022, from https://murciatoday.com/what-is-a-gota-fr%C3%ADa_120983-a.html

Norton Rose Fulbright. (2021, April). The EU Green Deal explained. Retrieved from Norton Rose Fulbright: https://www.nortonrosefulbright.com/en/knowledge/publications/c50c4cd9/the-eu-green-deal-explained

PIANC. (2016). *Criteria for the selection of breakwater types and their related optimum safety levels*. PIANC REPORT N° 196. Boulevard du Roi Albert II 20, B 3. B-1000 Bruxelles. Belgique. ISBN 978-2-87223-239-0

Reardon, K. (2020, March 24). *Heavy rains once again cause flooding in Los Alcázares and San Javier.* [Web article]. Retrieved from: https://theleader.info/2020/03/24/heavy-rains-once-again-cause-flooding-in-los-alcazares-and-san-javier/

Sargison, J., & Percy, A. (2009). *Discussion of "Hydraulics of Broad-crested Weirs with Varying Side Slopes"*. Australia: University of Queensland.

Servicio Geográfico del Ejercito, n.d. *Mapa de suelos de la Región de Murcia.* Región de Murcia. [map].

Sierra, C. et al. (1992). *LUCDEME PROJECT SOIL MAP*. Icon University of Granada, pharmacy department of edaphology. M-23001-1992 NIPO: 254.86-007-0

SkyCiv. (2019, February 13). *A comparison: Steel, Wood and Concrete.* Retrieved from Skyciv.com:

https://skyciv.com/technical/steel-vs-timber-vs-concrete/

Theresa. (2022, January 3). *How To Keep the Courant Number Below 1.* SimScale. Retrieved May 30, 2022, from https://www.simscale.com/knowledge-base/what-is-a-courant-number/#:%7E:text=The%20Courant%20number%20is%20a,it%20%E2%80%9Cskips%E2%80%9D%20the%20cell.

Uijttewaal, W. (2020). *Friction and Turbulence - Applications to rivers*. Delft University of Technology, CIE4345 River Dynamics 1, Faculty of Civil Engineering and Geosciences. [PowerPoint slides].

US Coastal Change Analysis Program (C-CAP). (2016). *C-CAP Regional Land Cover and Change.* NOAA Office for Coastal Management. Retrieved from: https://coast.noaa.gov/digitalcoast/data/ccapregional.html

Baars, S. van, & Voorendt, M. Z., & et al. (2022). *Manual Hydraulic Structures, edition 2022.* Delft University of technology.

Breuge, R.H. van, Ten Hove, T.D. (1995) *Steenstabiliteit bij horizontale Vernauwingen.* M.Sc. Thesis, Delft University of Technology

Vuistregels voor het ontwerpen van een draagconstructie. (2013, 11 20). Retrieved from wiki.bk.tudelft.nl: http://wiki.bk.tudelft.nl/mw_bk-wiki/images/1/12/Vuistregels.pdf

Wade, S. (2019, December 3). *Los Alcazares mayor says 'mud-filled drains' from September's gota fria to blame for fresh flash flooding.* Olive Press News Spain. Retrieved March 10, 2022, from https://www.theolivepress.es/spain-news/2019/12/03/los-alcazares-mayor-says-mud-filled-drains-from-septembers-gota-fria-to-blame-for-fresh-flash-flooding/

Wright, R. L., and Wilson, S. R. (1979). *On the analysis of soil variability, with an example from Spain.* Geoderma 22, 297-313.

Appendices

A Determination of design life and design hydrograph

This Appendix calculates the design life time and the corresponding design hydrograph. These calculations are used to support Section 3.1.2

To determine the value Q_{ext} of the extreme river discharge, multiple different inputs are needed:

- Design life time of the ZAC
- Acceptable probability of failure during design life of the ZAC
- River discharges for different return periods. These values are given by research of García Bermejo (2022).

The design life time of the ZAC has been derived from Figure A.1 (PIANC, 2016). This figure displays the different safety classes and the type of use that are applicable on the ZAC. Since the ZAC is part of the flood protection in the Murcía region, it falls in safety class 3. The direct surroundings as depicted in Figure 1.4 show that only rural roads lie in the vicinity, so hindrance is low. It is fulfilling only 1 infrastructural function, so the category Infrastructure of specific use is selected, leading to a design life time of 50 years.

Type of work	Safety classes		
	Safety class 1 Installations of local interest, minimum risk of loss of human life and environmental damages in case of collapse (coastal defences, minor or sea port works, sea waste-pipes, coastal roads, etc.)	Safety class 2 Installations of general interest, moderate risk of loss of human life or environmental damages in case of structural collapse (large port works, sea discharge exits of big cities, etc.)	Safety class 3 Installations for flood protection, works of supernational interest, high risk of loss of human life and environmental damages in case of structural failure (protection of urban or industrial centres, etc.)
	Minimum design working life (years)		
Infrastructure of general use	25	50	100
Infrastructure of specific use	15	25	50

Figure A.1: Design life time of structure classes. The circled value is the chosen value for the design life of the ZAC. (PIANC, 2016)

Subsequently, the acceptable probability of failure during the design life of the ZAC is determined. For this, Figure A.2 is used (PIANC, 2016). This figure is normally used for breakwaters and sea dikes. Since no documentation could be found on inland reservoirs and acceptable probabilities of failure, it is assumed that these values are applicable on this project as well. The ZAC is a structure that is used only if floods occur. Under normal conditions, there is a dry river. This means that there is no serviceability limit state (SLS), just the ultimate limit state (ULS), corresponding to total destruction in the figure.

When the structure fails, the risk of loss to human life is high in the downstream area in Los Alcázares, as this indicates a floods. Since the ZAC is part of a larger system consisting of multiple ZAC structures in the complete region, the economic repercussions are considered to be medium. Considering these assumptions, the acceptable probability of failure following from the table is $0.10[-]$ during its 50 year life time.

Incipient Damage	Risk to Human life	
Economic Repercussion	Limited	High
Low	0.50	0.30
Medium	0.30	0.20
High	0.25	0.15
Total Destruction	Risk to Human life	
Economic Repercussion	Limited	High
Low	0.20	0.15
Medium	0.15	0.10
High	0.10	0.05

Figure A.2: Acceptable probability of failure p_f during the life time of the structure. The circled value is the chosen value for p_f. (PIANC, 2016)

The occurrence of meteorological events like rainfall events are Poisson distributed and can be calculated with:

$$p_f = 1 - e^{\frac{T_L}{R}} \tag{A.1}$$

From this it follows that:

$$R = \frac{T_L}{-ln(1-p_f)} = \frac{50}{-ln(1-0.1)} = 474years \tag{A.2}$$

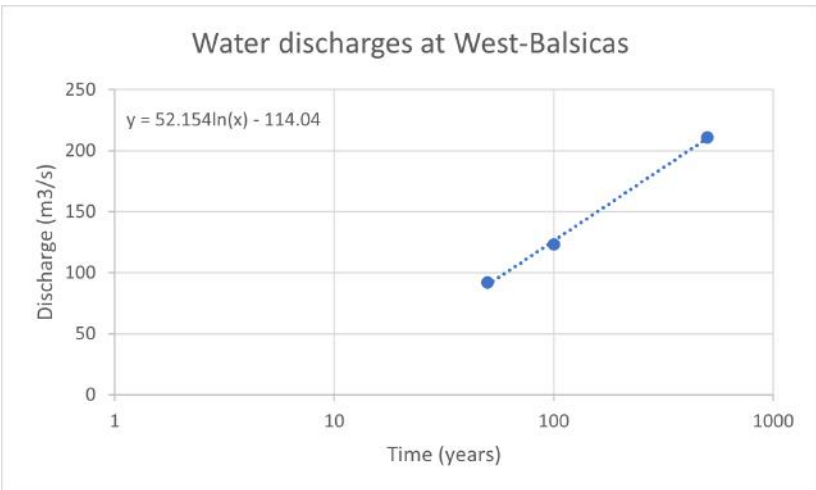


Figure A.3: Water discharges at West Balsicas. (García Bermejo, 2022)

The river discharge values for the return periods are given by García Bermejo (2022). Since there are only 3 data points, as depicted in Figure A.3, the estimate of the river discharge is crude. Nonetheless, it is a trend that can be used for the first analysis. The 3 return periods are plotted versus the extreme water discharge on a log-scale and a logarithmic trend line has been fitted to the data points. Finally, the Q_{ext} is calculated by using $R = 474$, leading to a value of $Q_{ext} = 207.4m^3/s$.

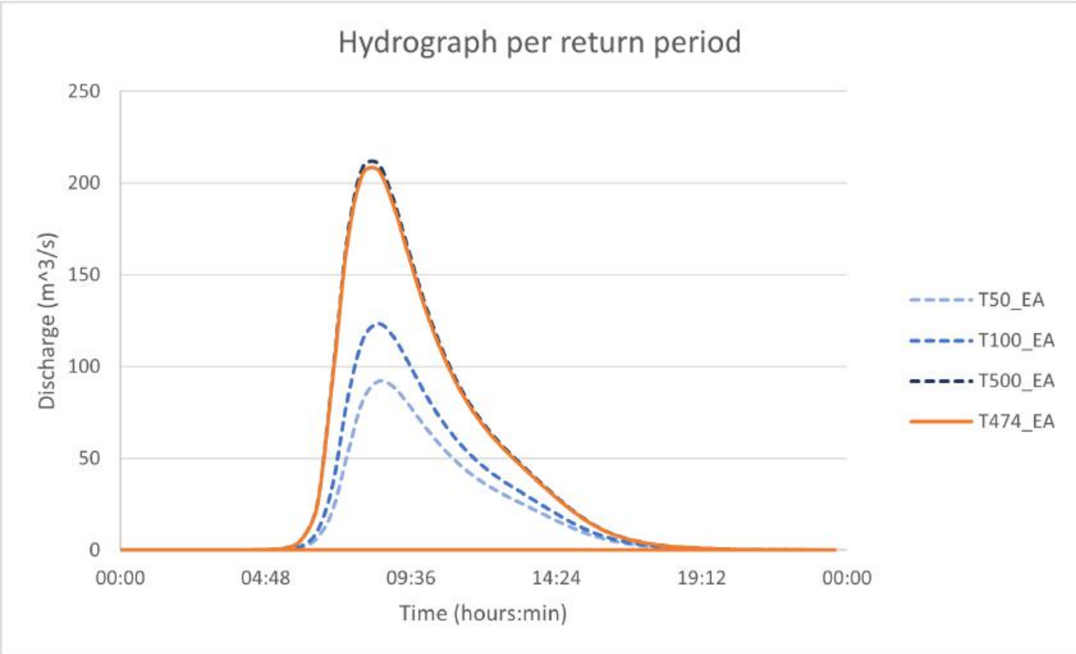


Figure A.4: Hydrographs for the return periods of 50, 100, 500 year and design return period 474 year.

B Determination of maximum flow through downstream channel

The main objective of this project is to implement a ZAC structure to reduce the maximum discharge during a once in 474-year discharge event. During rainfall events that do not flood the downstream area of the dry river, the ZAC should have no function and the flow must go freely through the ZAC structure. This enables the maximum flow towards the coast without flood. This maximum discharge capacity of the dry river reach determines the maximum allowed discharge capacity of the ZAC if no changes are made to the current downstream river reach. Therefore it is important to find the location in the river with the lowest discharge capacity and therefore the bottle neck location.

If rainfall event discharges occur that exceed the maximum discharge capacity of the current dry river reach, the ZAC structure should start storing water in its reservoirs to dampen the maximum discharge through the downstream channel.

The maximum discharge through the downstream river is determined via the HEC-RAS 1D model in the current situation without ZAC structure or other changes to the system. The 474-year hydrograph is scaled down to lower discharges. This means the maximum discharge of 207 m^3/s is lowered to 70 m^3/s as initial choice and the other values of the hydrograph which have been reduced with the same scale. The downscaled graph then has a smaller return period R too. These flow hydrographs are implemented in the 1D model to determine a potential flood downstream.

The scaled hydrographs are made by multiplying the hydrograph by the ratio of the wanted maximum discharge divided by the maximum discharge of one in 474 years: $Q_{newmax}/Q_{max474y}$. The first guess was a maximum flow discharge of $70 m^3/s$. Upon iteration, the discharge with no flood in the surrounding is found at Q_{newmax} = $51 m^3/s$ and gives a minimal flood of 51 m^3/s at the downstream part of the dry river. Figure B.1 shows the complete overview of the dry river reach with a maximum discharge of 70 m^3/s, and Figure B.2 is zoomed in on the downstream area. The downstream area is the only relevant area where the water that exceeds the banks causes disturbing flood areas. The bottle neck is indicated in the figure with a yellow circle. This is the area of a orange-farmer. The terrain abruptly transitions into a channel, as shown on the satellite image on the right. Not all water can contract into this stream, so this is a part where water can exceed the banks and flow downstream in direction of the tow of Balsicas, as indicated by the yellow arrow. Furthermore, the results are shown for no overflow during a maximum discharge of 51 m^3/s in Figure B.3. The return period of a rainfall event with a peak discharge of $51m^3/s$ is found by scaling the hydrograph from Appendix A down. The return period for such a rainfall event is $R = 24\, years$.

The result of the flow hydrograph with a maximum discharge of 51 m^3/s is that there is no overflow, meaning that the maximum flow through the downstream channel is equally large. If this bottle neck would be adapted to increase the capacity, it allows the downstream channel to have a larger discharge capacity to avoid flood in Balsicas.

To conclude, the maximum discharge without floods in the downstream region is 51 m^3/s and is set as the upper limit for rainfall events without flooding of the downstream area. Therefore, the ZAC should provide a maximum discharge of 51 m^3/s, if no additional measures are taken. Discharges larger than the maximum downstream discharge should increase the water level in the ZAC reservoirs. The return period of a rainfall event with a peak discharge of $Q = 51\, m^3/s$ is $R = 24\, years$.

Figure B.1: Depth map resulting from a scaled flow hydrograph with a maximum of $70\ m^3/s$. This is a complete overview of the dry river reach.

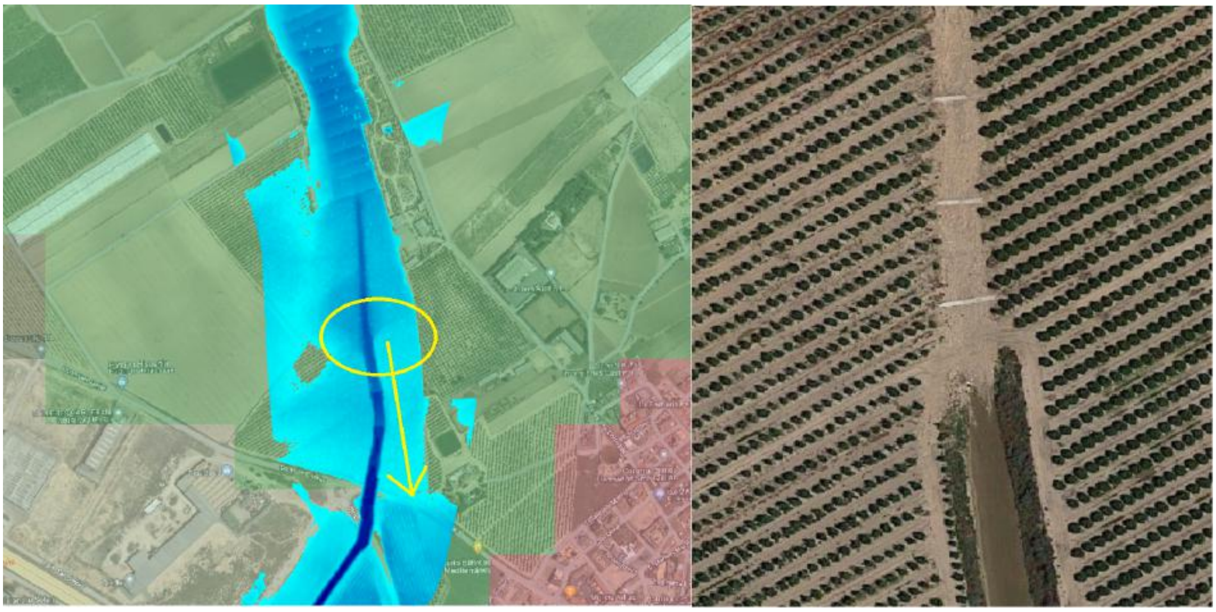


Figure B.2: Depth map resulting from a scaled flow hydrograph with a maximum of $55\ m^3/s$. This is an overview zoomed on the downstream end of the dry river reach.

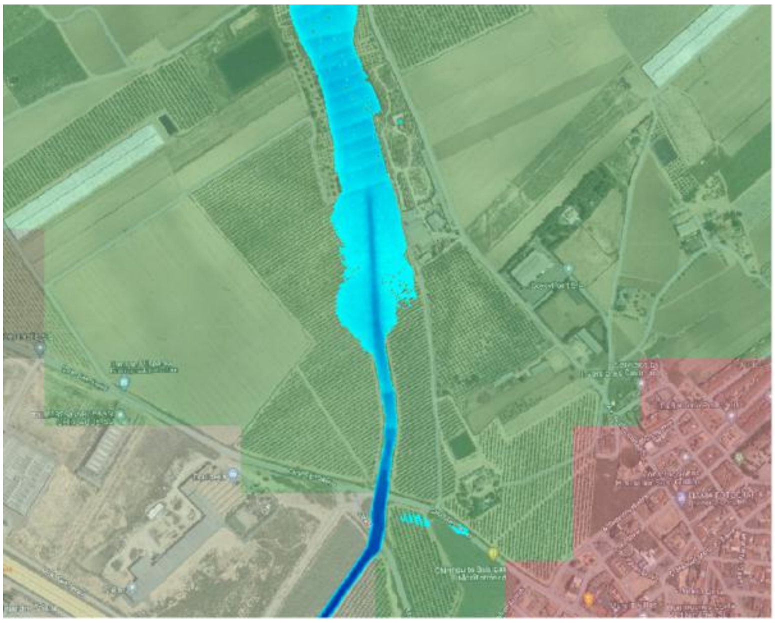


Figure B.3: Depth map resulting from a scaled flow hydrograph with a maximum of $41\ m^3/s$. No overflow occurs, which means it determines the maximum capacity of the dry river.

C Filling-process steps of ZAC

This Appendix shows an illustrated representation on how the ZAC responds to a flow hydrograph that represents a rainfall event.

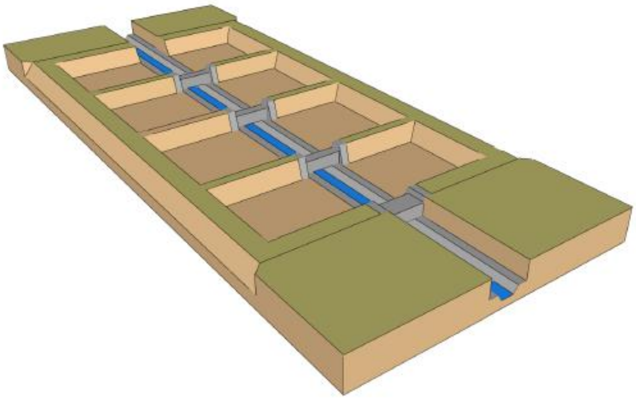


Figure C.1: A 3D-view of the ZAC-structure is shown. The water level flows through the channel and is lower than the spillway structure. Therefore, the water can flow freely through the ZAC without interaction.

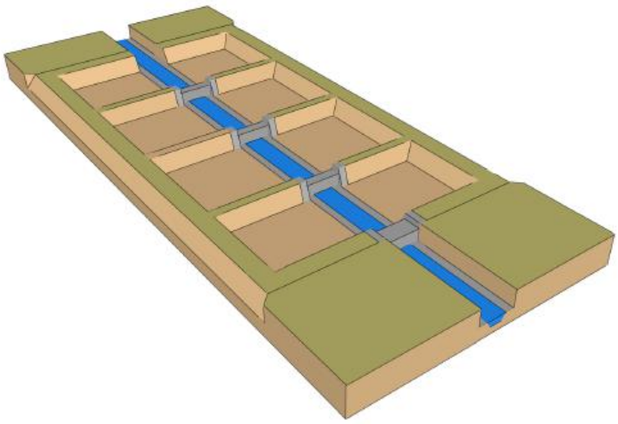


Figure C.2: The water level in the inlet increases but is at the same level as the top of the culvert. So, the water is still able to flow through the culverts.

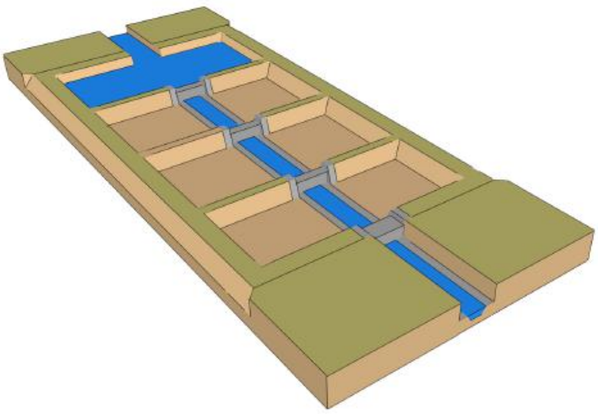


Figure C.3: The discharge at the inlet of the ZAC is larger than the maximum discharge through the culvert, which result in increasing water level of the first basin.

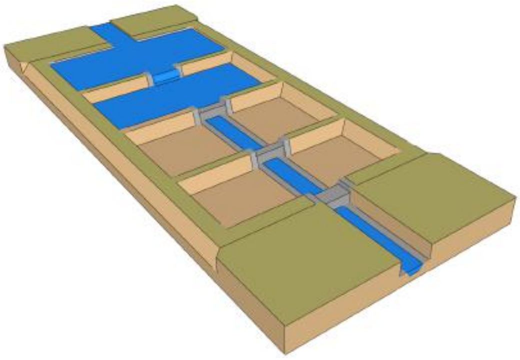


Figure C.4: After the first basin is going to be filled, the water will overflow through the spillway when the water level is at the same level as the spillway. Furthermore, the culvert has a larger discharge due to the increasing hydraulic pressure. This two processes results in filling of the second basin.

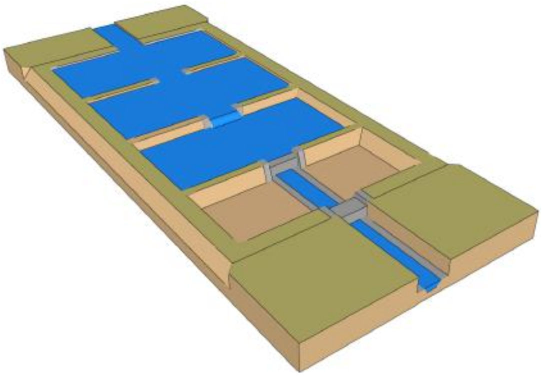


Figure C.5: When the large discharge event is of significant duration, the consecutive basins are filling up. The structure operates still within capacity.

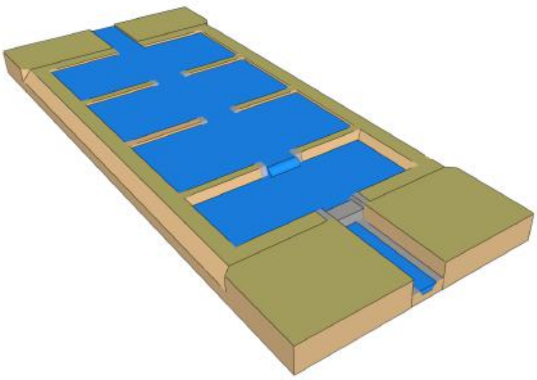


Figure C.6: The last basin is filled. The ZAC-structure reaches its maximum capacity.

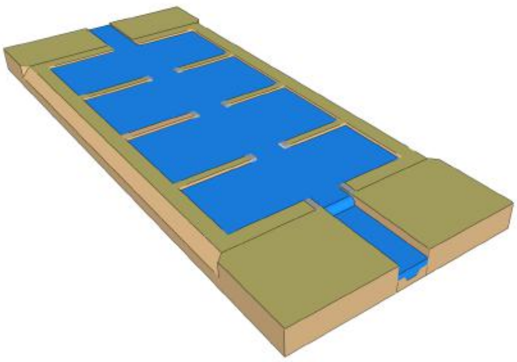


Figure C.7: When the extreme discharge event exceeds the design values, the ZAC can no longer fulfill its function. This results in overflow into the river downstream.

D Location description

This Appendix gives an elaborate description of the two project locations. This Appendix is used to support Chapter 4.

D.1 Location A

Surrounding buildings
There are no direct surrounding buildings bordering the area and also no buildings inside the area. At 180 meters of the southernmost boundary there is a small corporate building, and a horticultural building is located at 110 meters of the downstream boundary. Due to its size, it is counted as 4 buildings. Behind the downstream bordering reservoir, at 170m of the downstream boundary, there is a small corporate building.

Surrounding houses
There are no directly bordering houses. The closest home is 180 meter downstream of location A, behind the downstream boundary, with neighbours on both sides.

Storage facilities
There are no storage facilities like silos, warehouses or water reservoirs in the area. Next to the east side of the area is a water reservoir. The other closest water reservoir is at approximately 70 meters from the southernmost point of the area, right next to the corporate building. At 100 meters from the downstream end of the area, there is a warehouse.

Farmland in and around the area
Farmland in the area of location A is approximately $113.000\ m^2$. Further, the area is bordered by one farm on the west side. If the ZAC were to be expanded in the future, these patches need to make space for the ZAC entirely, because the patches would get too narrow if only a part of the patch is used.

Infrastructure
The area cuts through a private road connecting the home in the northeast direction. The home is connected via another access road too, which means it is still accessible. It is rerouted by 2.5 km. The area does not cut through electricity power lines, but in the south near the corporate building there are multiple power line poles installed. These would limit future expansion, but do not limit the current area. The rest of the companies and houses can still access the road the same way.

Vegetation
The area cuts (partially) through 5 fields that are in use, the current dry river plains, and 1 unused field. This project area cuts through a total area of 113.000 m^2 that is in use. The total area is approximately 250.000 m^2.

Water supply
The only water supply in this area is the upstream part of the location.

D.2 Location B

Surrounding buildings
Along the east side of the project location, across the road, there is a relatively large horticultural company with accompanying production buildings. At the west side of the project location, there is an agricultural machinery manufacturer. Around 50 meters north of the project location, a farm is located.

Surrounding houses
Around 60 meters south of the project location, there is a house. Right outside the project location, and just below the horticultural company, another house is located.

Storage facilities
Just above the farm, a long storage facility is located. Inside the project location, there are two existing water reservoirs.

Surrounding farmland
The project location has approximately 95,000 m^2 of farmland in use. The total project area is approximately 180,000 m^2. The area that must be bought from local population is smaller than the area required for location A.

Surrounding infrastructure
Since the project location does contain quite a lot of buildings, and also a large agricultural machinery manufacturer, there will be electricity cables and sewers running in and around the project location. A road forms a natural boundary around the project location. Furthermore, there is a smaller road crossing the narrowest part of the project location. 5 houses are cut off from their main route to the doorstep. Their access to the road has been rerouted by 1.4km.

E Multi criteria analysis tables

Table 1 and table 2 form the multi criteria analysis used in chapter 4 to decide which location is best suitable for the construction of the ZAC.

Main Criterion (A)	Weight Factor A	Sub-criterion (B)	WF B	Relative weight (C=A*B)	Score Vc	Relative Score (D=C*Vc)
Environmental	0.1	a. Farmland	0.8	0.08	45.67	3.65
		b. Trees	0.15	0.015	75.22	1.12
		c. Plants	0.05	0.005	52.46	0.26
Societal	0.3	a. Close Work buildings (200m reach)	0.3	0.09	82.85	7.45
		b. Close Houses (200 m reach)	0.45	0.135	81.25	10.96
		c. Storages	0.15	0.045	44.54	2.00
		d. Roads	0.1	0.03	26.31	0.78
Water	0.6	a. Inflow	0.6	0.36	47.61	17.14
		b. Potential storage area	0.4	0.24	58.13	13.95
					Total	**57.36**

Table 1: Multi criteria analysis of location A. The total score of A and B summed is 100 for every value in the Vc column. The total score of 57.4 means location A is preferable over location B.

Main Criterion (A)	Weight Factor A	Sub-criterion (B)	WF B	Relative weight (C=A*B)	Score Vc	Relative Score (D=C*Vc)
Environmental	0.1	a. Farmland	0.8	0.08	54.32	4.34
		b. Trees	0.15	0.015	24.77	0.37
		c. Plants	0.05	0.005	47.53	0.23
Societal	0.3	a. Work buildings (200m reach)	0.3	0.09	17.14	1.54
		b. Houses (200m reach)	0.45	0.135	18.75	2.53
		c. Storages	0.15	0.045	55.45	2.49
		d. Roads	0.1	0.03	73.68	2.21
Water	0.6	a. Inflow	0.6	0.36	52.38	18.85
		b. Potential storage area	0.4	0.24	41.86	10.04
					Total	**42.63**

Table 2: Multi criteria analysis of location B. The total score of A and B summed is 100 for every value in the Vc column. The total score of 42.6 means location B is less preferable than location B.

The explanation of the criteria in the above tables is shown below:

- *Loss of farmland area:* The area of used farmland is measured for both locations using Google Earth. Then, the relative contribution of each location is calculated, giving the lower score to the largest in-use farmland location. This is because more land needs to be bought, which negatively affects the MCA-score. This makes location A less favourable, scoring 45.7, whereas B scores 54.3.
- *Number of removed trees:* The number of trees in each location is estimated using satellite imagery from Google Earth. When more trees need to be removed, it leads to a lower score for the location.
- *Area percentage of plants:* From the same imagery, the percentage of area that is filled with plants on the unused land is estimated for both locations.
- *Number of company buildings (reach 200m):* The number of company buildings within a reach of 200 meter of the project area are counted for both locations, and calculated the same way as farmland.
- *Number of houses (reach 200m):* See above.
- *Existing water storage area:* The cumulative area of all water storages is measured, which indicates how much of this type of property needs to be removed to make space for the ZAC. Calculation: see above.
- *Road-household-kilometers* The number of houses that are affected by either location A or B are counted. Then, the number of kilometers that they need to drive extra to get home, assuming they come from the main roads, are measured. These values are multiplied and summed for each house, so the total house-reroute-kilometers can be compared.
- *Inflow volume of water:* The inflow in the locations can be different, as location B can also experience inflow from the eastern branch. Since no data is available on the potential discharge of this branch, an assumption is made on the maximum value. From Avigueras Rodríguez (2022), shown in Figure 4.2, the red dashed circle includes ZAC 7 and ZAC 8, which are locations A and B respectively. Since this map does not even show a branch on the east between ZAC 7 and 8, it is assumed that the branch contributes to maximum 10% of the inflow at the main branch. This leads to a favourable score at location B, because the total inflow there could be larger. Thus, increasing the effectiveness of the ZAC.
- *Potential storage area:* The project location of A has a larger area ($250,000m^2$) than location B ($180,000m^2$). In the future, this could be a bottleneck for possible expansion, which gives location A an advantage over location B. The same calculation as farmland is applied.

F Determination of the groundwater level at the project location

This Appendix aims to calculate the groundwater level at the project location in support of Chapter 3.

Project location A and B are situated in a remote area, which results in less available data of the local conditions and therefore the groundwater level has to be approximated. The groundwater level in the entire Murcia region is hard to estimate due to the low number of measuring stations. Data of these few stations are available from 1973 to 2011 and are provided by the Spanish Ministry of Environment. By using this data and interpolating towards the project location, an estimation is made for the groundwater level. The water levels are expressed in metros sobre el nivel del mar, with abbreviation SNM + m, which is the reference to measure the local elevations compared to the mean sea level.

Figure F.1 shows the measuring stations in the Murcia region. The measuring station surrounding the project location are used for the estimation. All these stations are within a range of 7 kilometers and show a large difference in groundwater levels. At the northern side of the ZAC, the water levels are between SNM $+11\,m$ and SNM $-31\,m$ at the latest known data in 2011. At the southern side of the ZAC, the measuring stations are in a range of SNM $+59\ m$ and SNM $+104\ m$ at the latest known data in 2011 (Ministerio de medi ambiente, 2015).

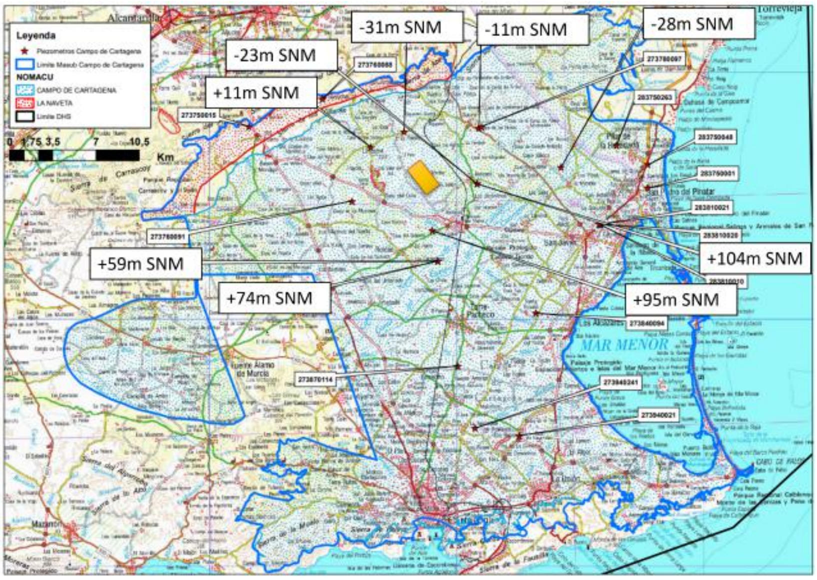


Figure F.1: Groundwater level at measuring stations in the region of Murcia and the last known groundwater levels surrounding the project location in the yellow rectangle (Ministerio de medi ambiente, 2015).

The maximum and minimum levels at the northern and southern stations are used to estimate the range of the groundwater level at the project location. By interpolating the minimum levels from North to South (SNM $-31\ m$ to SNM $+59\ m$) results in an estimated level of SNM $+14\ m$ at the project location. The maximum levels from North to South (SNM $+11\ m$ to SNM $+104\ m$). results in SNM $+57.5\ m$ for the groundwater level. This leads to an approximated range of SNM $+14\,m$ to SNM $+57.5\,m$.

The project location A has an elevation of SNM $+134.5m$ to SNM $+126.3m$ from inlet to outlet of the ZAC structure when the total available area is used. Comparing the maximum approximated groundwater level at SNM $+57.5\ m$ and the lowest elevation of the ZAC at location A, it can be concluded that the groundwater level is approximate $68.8\ m$ below the ZAC.

G HEC-RAS 1D and 2D model set-up and limitations

This Appendix gives a more elaborate explanation of the HEC-RAS model described in Chapter 5.

G.1 1D model set-up

Step 1

The terrain is loaded into HEC-RAS. This is done by applying terrain data retrieved from Centro de Descargas – DTM02 which has a spatial grid with cells of 2 x 2 m. The model requires a spatial reference system, which for this area is EPSG:32630, WGS84 UTM zone 30N: the time zone for Spain. Then, satellite imagery from Google Earth is loaded into the model. The spatial reference system allows the satellite imagery to overlay the terrain without any errors.

Step 2

A land cover layer has been loaded into the model, assigning different types of land cover to each area. This data has been retrieved from CORINE land cover 2018 from Copernicus Land Monitoring Service. Each land cover type means a different type of soil where the water is flowing over. Simultaneously, these different soil types represent different Manning coefficients. The resolution of the layer is 100 x 100m, which is significantly coarser than the 2 x 2m of the terrain profile. For a first estimation of the river flow this resolution is considered acceptable. Manning values are taken from the US Coastal Change Analysis Program (C-CAP) (2016). The relevant land covers in the project area and their and governing Manning values are shown in Table 3. An overview of the terrain and the land cover layer on top is displayed in Figure G.1.

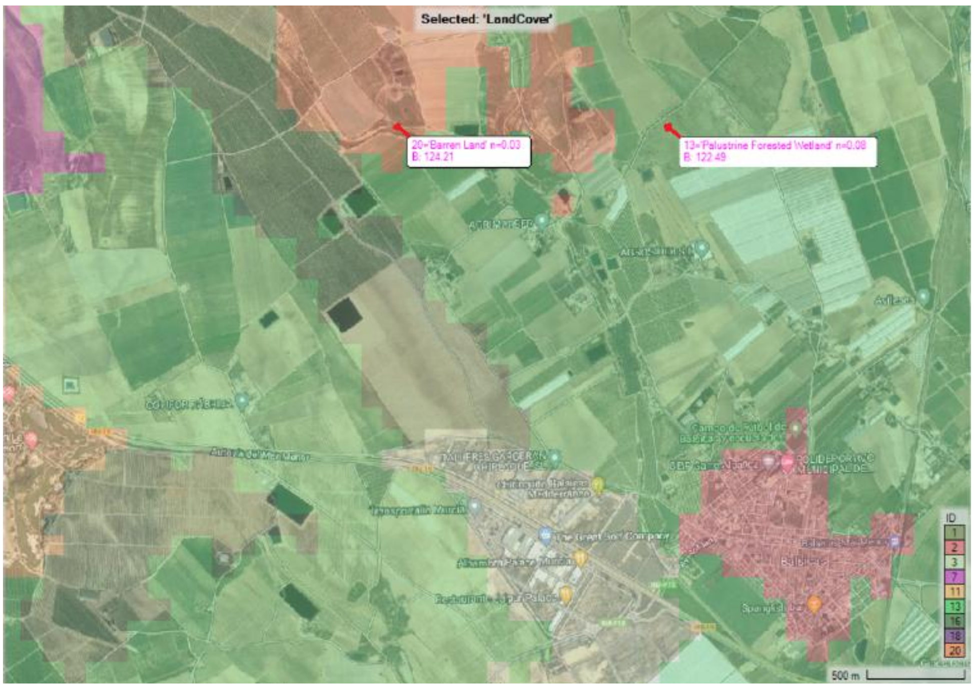


Figure G.1: Terrain and accompanying land cover. The land cover layer is on top of the satellite image and each color indicates its own Manning value. It is the pixel-like layer. The 2 red arrows indicate respectively the land types 'Barren Land, n = 0.03' and 'Palustrine Forested Wetland, n = 0.08'.

Table 3: Manning values for different land cover types from C-CAP (2016).

Land Cover Type	Initial Manning Value [-]
Estuarine Emergent Wetland	0.06
Palustrine Forested Wetland	0.08
Estuarine Forested Wetland	0.08
Barren Land	0.03
Developed - High Intensity	0.15
Mixed Forest	0.12
Developed - Medium Intensity	0.12
Pasture/Hay	0.045

Step 3
The flow path of the river, the banks and a global conveyance area are set by using the satellite images and the digital terrain profile.

Step 4
Cross-sections are drawn in the river. The 1D-model interpolates the varying bed level values between cross-sections. It is therefore important to draw cross-sections without too many vertical deviations. In the downstream area, near the highway, there are a couple of bridges in the terrain data. The model does not know that water flows underneath this surface. Therefore, the terrain has been lowered manually at these locations. Upon model iterations and visual investigation, it is chosen to take spatial steps of 80-100m between cross-sections. This is a trial and error process in which the analysis must be ran several times for spacing varying from every 10 to every 200m between the cross sections to compare the accuracy of the results. Having a too high spatial step impacted water flow between cross sections, as it went out of the banks and did not follow the river reach accurately. A smaller spatial step did not yield much differences in the details of the final overflow depth-maps A smaller spatial step requires longer computation time, so the spatial step of 80-100m is accepted. For a more elaborate explanation with figures, the same iterative process as for the time step has been used, see *step 6*.

HEC-RAS calculates a water level, then fills the complete area underneath that water level with water. This means that areas that are not part of the flow are also filled with water. This is the case for cross-section 3018, see Figure G.2. This requires the application of levees in the model. In Figure G.3a the unchanged cross-section is shown. The left 'channel' is filled, and the right is the main channel. However, upon investigation of the local terrain the left channel turns out to be a farmland water reservoir. The bump of land in the middle separates these flows, and therefore the left part only gets filled if the water level exceeds the bump level. For this cross-section this is not the case and it remains empty, as shown in Figure G.3b. The complete river reach needs to be examined on this phenomenon to avoid incorrect flow areas in the 1D model.

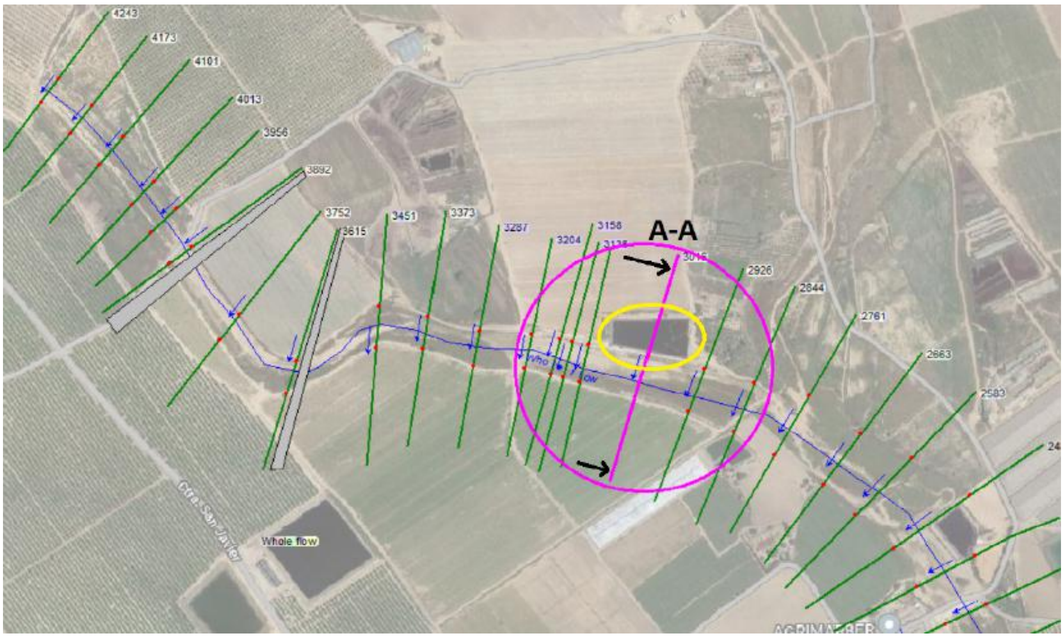


Figure G.2: Location cross-section 3018 in the river reach. A-A depicts the cross-section as given in Figure G.3. The yellow circle depicts the reservoir, which is the left 'channel' in Figure G.3.

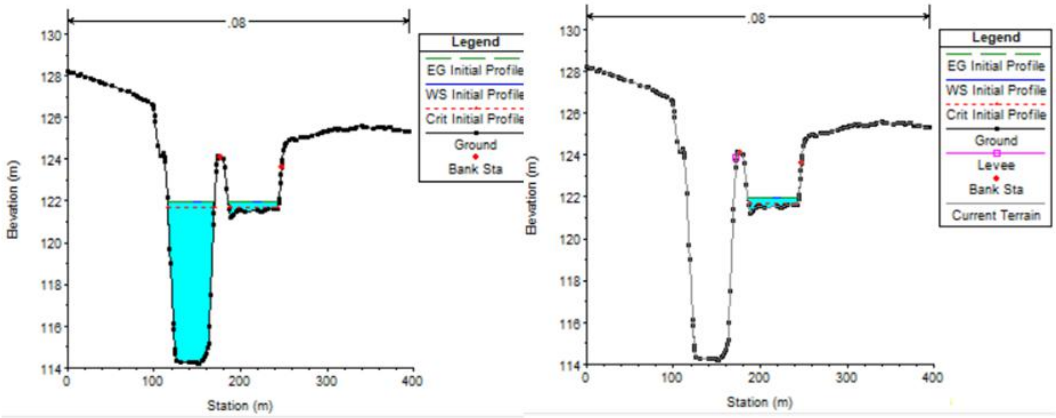


Figure G.3: A) Unchanged cross-section 3018. This is how the model fills up the storage area under the calculated water level. B) After putting a levee on the bump, only the right channel is filled, as the water can not flow over the bump into the reservoir.

Step 5

The model represents a dry river with a flood event, meaning at the upstream boundary an unsteady flow is imposed. This is the flow hydrograph as provided by Bermejo (2022), see Figure G.4. The data for the flow hydrograph are flow rates over time based on statistical data with a certain return period. As input for the model, these values have been interpolated for a return period of 474 years, see Appendix 8.4, with the extreme discharge event having a total duration of 18 hours. When using a base flow of $Q = 0\, m^3/s$, numerical errors occur. In this case, the Overall Volume Accounting Error (OVAE) is 99.92 %. As a result, the minimum flow rate at each time step in the flow hydrograph has been set to $10\, m^3/s$, which calculates the OVAE with an error of 0.03 %. Otherwise the model result is that the river remains empty at different locations and the flood wave does not propagate smoothly towards the downstream area, see Figure G.5. A larger base flow is not desirable, as it affects more strongly the resulting water levels contributing to overflow. Applying a base flow of $10\, m^3/s$ means that all values in the 474-year flow hydrograph below $10\, m^3/s$ are rounded up to $10\, m^3/s$. This value is small enough to avoid a significant impact on the flood event, as with this discharge the flow remains within the banks in the entire river reach, and simultaneously it lets the numerical inconsistencies disappear, see Figure G.6.

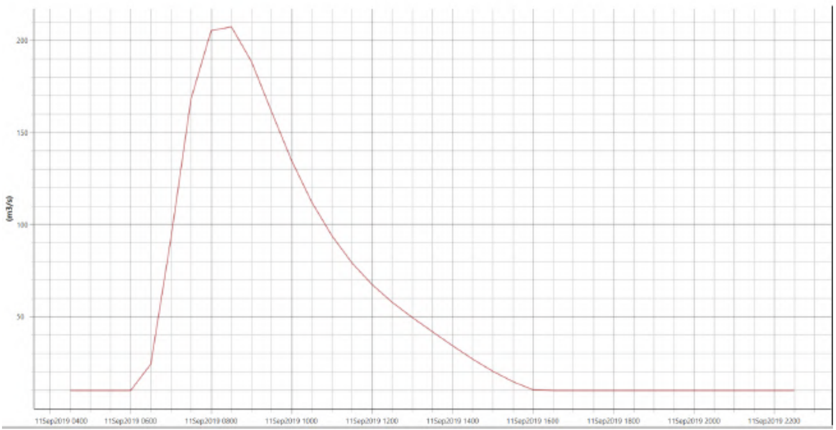


Figure G.4: Flow hydrograph input at the upstream boundary. The base flow has been modified to $Q = 10m^3/s$.

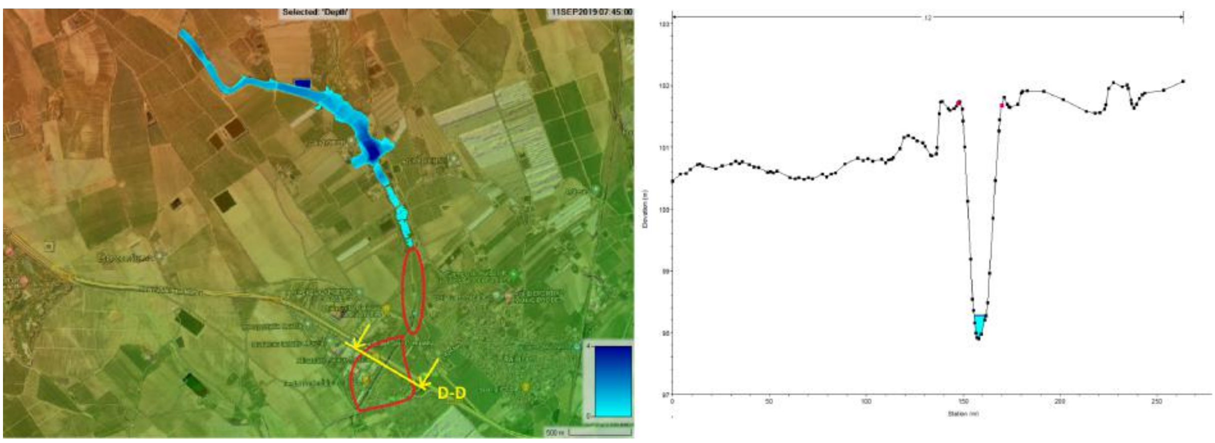


Figure G.5: Left: Flow depth map for an unchanged discharge base flow. The red shapes indicate that downstream the flow disappears, which is unrealistic. Right: Cross-section D-D and its water height displayed in the cross-section.

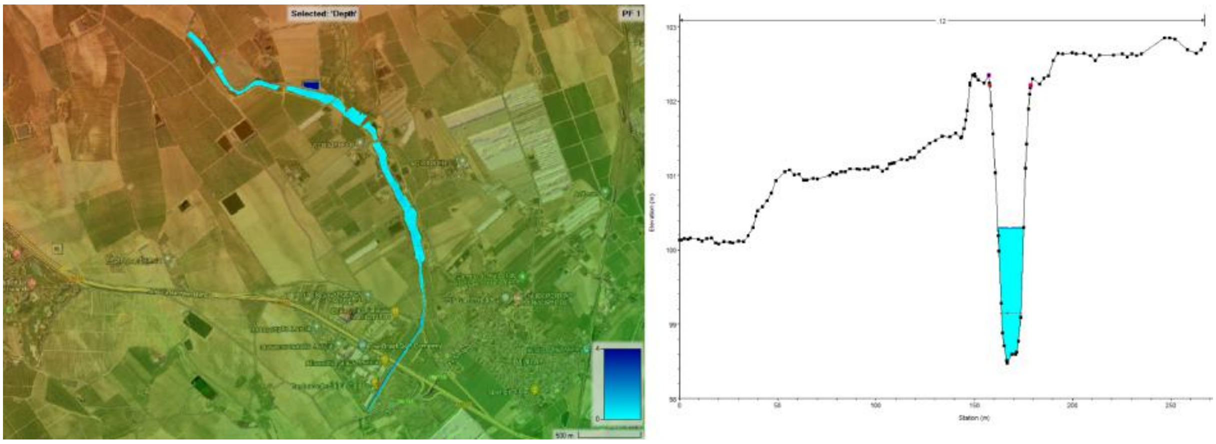


Figure G.6: Left: Flow depth map for a steady base flow of $10\ m^3/s$. The entire river reach is covered with water. Right: Cross-section at downstream area: the base flow does stays well between the river banks, so applying this base flow is a solution to the OVAE.

Subcritical flow is assumed downstream, even though there is no normal flow. Subcritical flow is defined as $Fr << 1$ where

$$Fr = \frac{u}{\sqrt{gd}} \tag{G.1}$$

When modelling the channel downstream with an assumed downstream boundary condition normal depth, the calculated depth in the design rainfall event turns out to be approximately 3.6m. For the flow to be supercritical, this requires a flow velocity of $u > \sqrt{3.6 * 9.81}$, so $u > 6.0\ m/s$. This is an unrealistically high value for flow velocity in the entire cross-section, which justifies the assumption that subcritical flow can be imposed on the downstream boundary.

The required value of the bed slope i_b is the only unknown in the determination of the normal depth.

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

This is because g is a constant $[m/s^2]$, q is given by the upstream boundary, and c_f follows from rewriting the channel roughness given by the Manning coefficient (Uijttewaal, 2020). At the downstream boundary a normal depth boundary condition is imposed by entering the bottom slope at the downstream end. This value is measured in the terrain and is equal to 0.011 $[-]$.

Step 6

The next step was to determine the time interval. The computational time step interval has been set at 1 minute. The results of computations with different time steps are shown in Figure G.7. The flow areas of the models with a time step of 1 s, 10 s, and 1 min are identical. This means there is no extra visual accuracy on this scale to choose for a time step smaller than 1 minute. In the model with a time step of 10 minutes, the results are drastically skewed: the river is partially dry at the maximum flow, and does not propagate entirely towards downstream, which represents an unrealistic scenario. Further optimization was attempted by trying a time step of 5 minutes, but this also did not suffice. Therefore, the time step of 1 minute has been applied in the 1D model. It is stressed that this flood event is hypothetical, as the once in 474-year rainfall event is discussed. This type of repetitive approach was also used to determine the spatial step in *Step 4*.

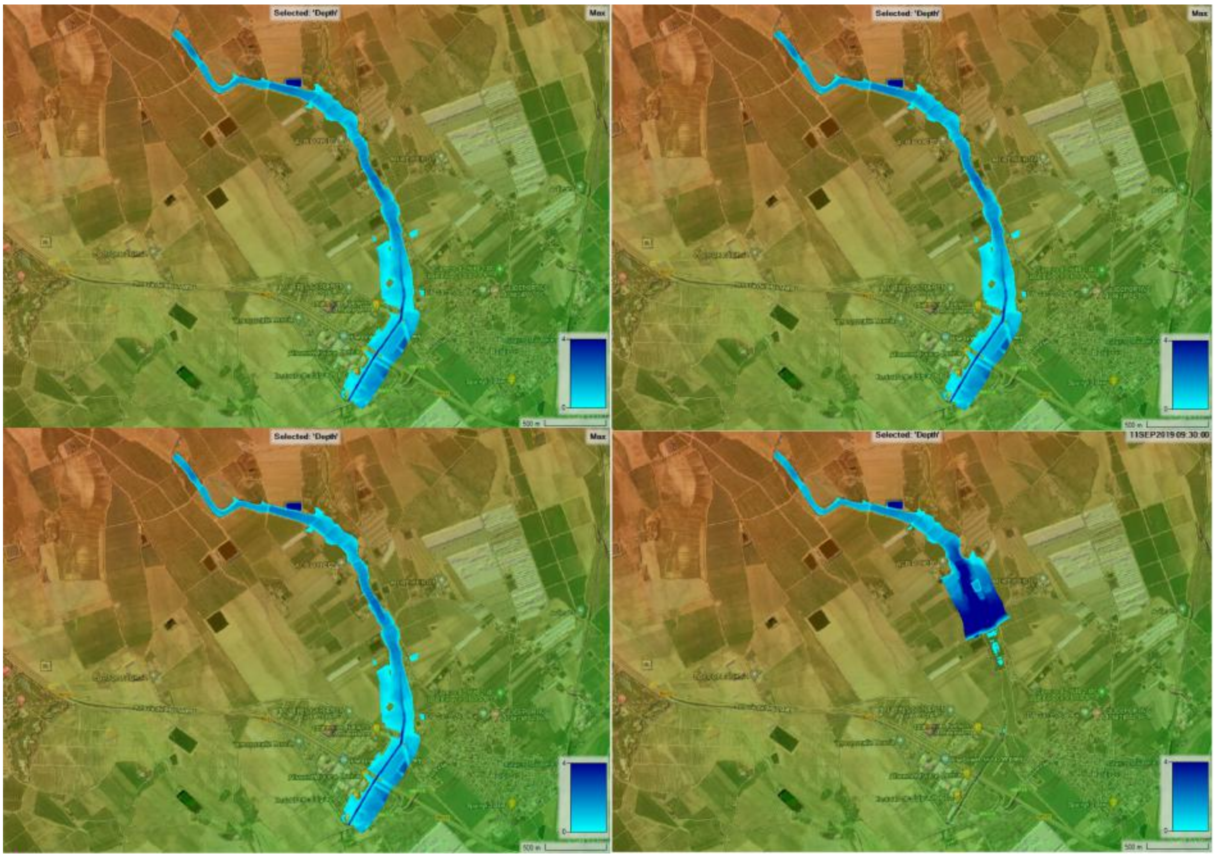
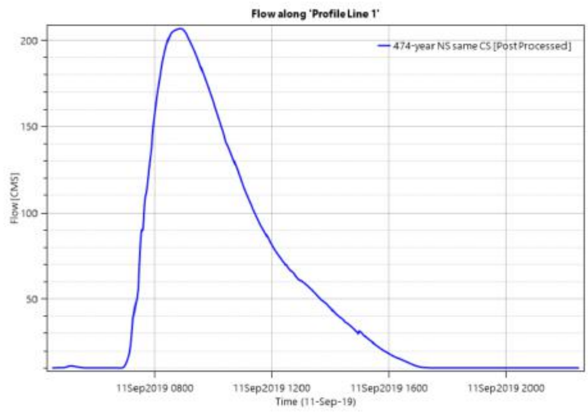


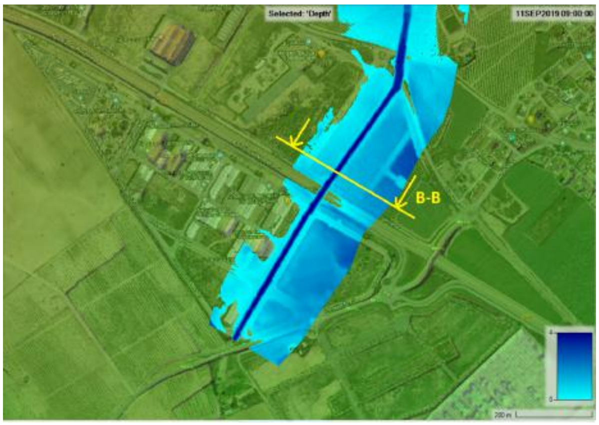
Figure G.7: Top left: $dt = 1s$. Top right: $dt = 10s$. Bottom left: $dt = 1min$. Bottom right: $dt = 10min$. The resulting flow maps are identical for the first 3 time intervals, which means the time interval of 1 min is the optimal choice in terms of computation duration. The time interval of 10 min is too large, resulting in an incorrect flow depth map.

Step 7

Now the model can be computed. After the model is finished, the output is a flow-depth map for each time step, see Figure G.8a. The flow depth is taken in the downstream region of the flow, as illustrated in Figure G.8b. This method is used to calculate the current conditions if nothing is done about the flood-risk in the area.



(a) Flow hydrograph at cross-section B-B during a 474-year rainfall event in current (unchanged) project area. The flow area is maximum when the peak of the flood wave passes. That is the moment visible in the picture.



(b) Flow depth map near downstream end of river reach.

Figure G.8

G.2 Limitations of the 1D model

The application of the model has some limitations. Some can be fixed and some must be considered acceptable or unacceptable. The first one is the model calculating the water level, then filling the water in the entire cross-section underneath that level. As explained in *Step 4*, this is not accurate if there is a reservoir which is not part of the flow.

The above problem cannot be fixed if there is no land bump that keeps the water contained in a channel. In Figure G.9 the water level is slightly higher than the banks. The red arrows indicate that the small amount of water that overflows from the bank floods the area behind the bank instantaneously. This is unrealistic, as it is a volume that is distributed of the cross section over time. This model problem cannot be fixed by simply putting a levee on the left bank (which is done in Figure G.3), because the complete area left of the left bank is lower. The model thinks this area is flooded, not taking into account the total volume that has been released at the upstream boundary by the flow hydrograph. As a result, the flow map has created non-existent water and therefore is incorrect, see Figure G.10. This limitation is because the model is 1D. A 2D model is able to perform calculations in the lateral direction.

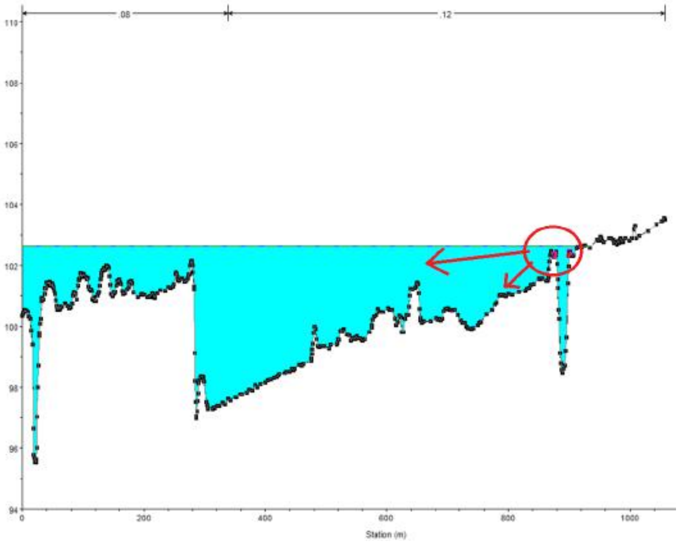


Figure G.9: Flooding is instantaneous. HEC-RAS does not take into account how much volume can fill up during a time interval, making the flow depth maps in this case incorrect.

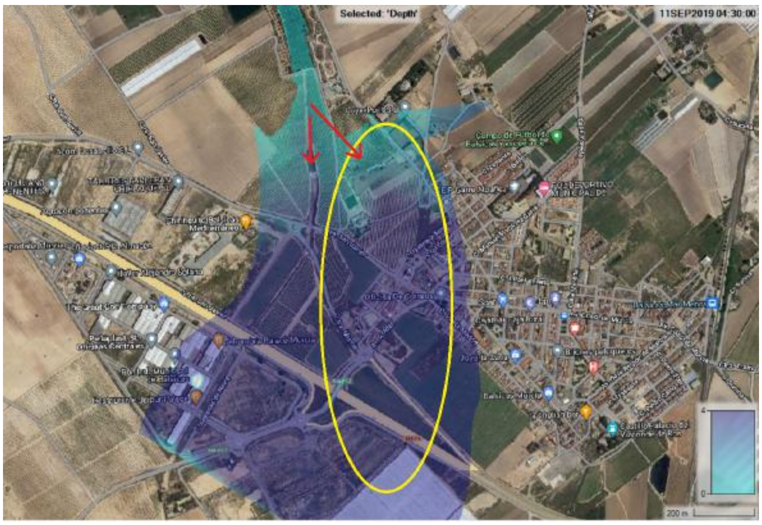


Figure G.10: Flood map if there is no land bump in the cross-section enclosing the water in a channel.

Another way to try fix this problem is to draw a bifurcation upstream, at the location of the red arrows in Figure G.10. Then, 2 river reaches exist, which could model the overflow area downstream. The reason that the water level starts to get out of the bank at this location is because there is a very small channel, and the bed slope is reduced, which leads to a larger flow cross-section and therefore flooding. There are 2 drawbacks of trying to model a bifurcation at the indicated area:

- It is unclear what the flow path of the downstream area would be. This flow path should be somewhere in the yellow circle in Figure G.10, but there are no visual river marks to draw it accurately.
- The bifurcation requires a split factor of the flow into the 2 downstream reaches. There is no information available on this.

These 2 reasons are enough to conclude that applying the 1D model for the 474-year rainfall event is not adequate in the case where no storage areas are applied upstream. This means that even if it is possible to model a ZAC-structure at location A, reducing the peak flow and therefore the flood area downstream, it is not possible to compare the ZAC-situation with the no-ZAC-situation.

Yet, still an attempt was made to model a ZAC at location A by adding dikes orthogonal to the flow, lateral dikes in line with the flow and even a storage area to the right of the ZAC. Unfortunately, the goal to create the ZAC-situation in the 1D model was too complex to get the model to function. Due to time constraints and the above mentioned issues it was decided to give up on the 1D model application further in this project.

G.3 2D model set-up

Step 1

For the 2D model, the same LIDAR terrain data is used as for the 1D model. This is a DTM layer with a resolution of 2 x 2 m.

Step 2

The terrain layer is accompanied by a land cover layer. A rough land cover layer is retrieved from CORINE land cover 2018. Subsequently, this layer is then further detailed by manually adding sections with different Manning values and permeability values. The reason behind this is to realistically estimate the resistance the water experiences while flowing downstream. The Manning values are taken from the US Coastal Change Analysis Program (C-CAP) (2016). Figure G.11 is an overview of the of terrain and the land cover layer.

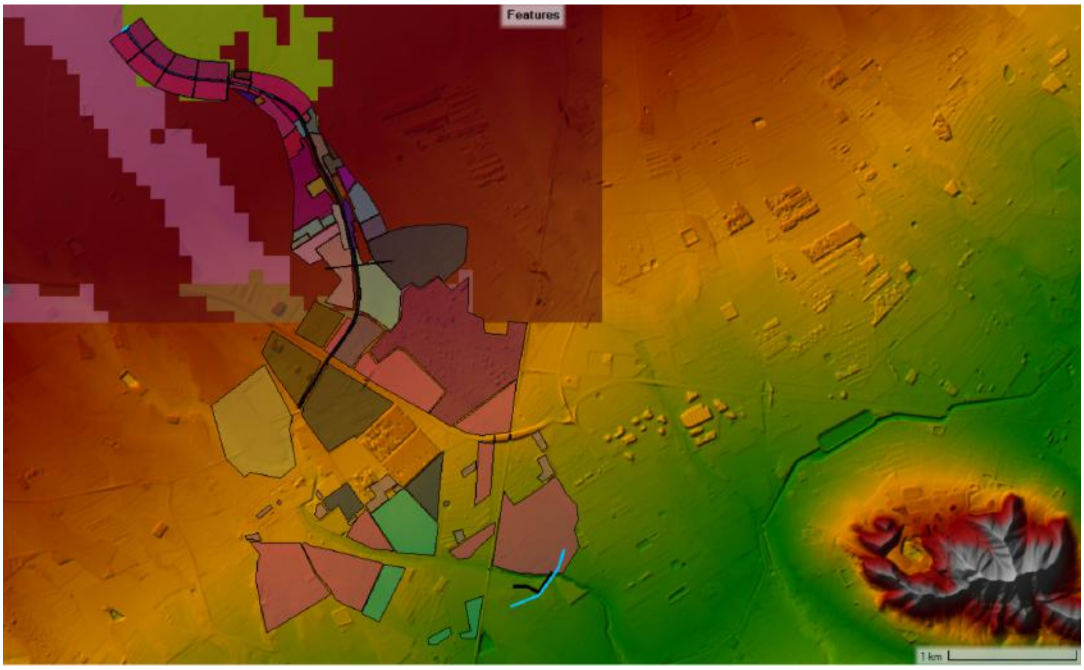


Figure G.11: The terrain map and accompanying land cover layer used in the 2D model.

Step 3

The next step is to model the situation without a structure. This is done by first defining a computation area with a refined region in the main channel. The computation area has a grid size of 20 x 20 m and the refined region has a grid size of 2 x 2 m. These resolutions are used so that the area of the main flow has the same resolution as the LIDAR data and the rest of the computation area has a coarser resolution to reduce the simulation time. With the computation area defined, the boundary conditions are imposed. This model uses two boundary conditions: a flow hydrograph for the upstream boundary, which corresponds to the design conditions calculated in Appendix A, and a normal depth for the downstream boundary, which is set at a sufficiently large distance from the structure to reduce computational errors regarding the appearance of normal depth. Figure G.12 shows a part of the computational grid.

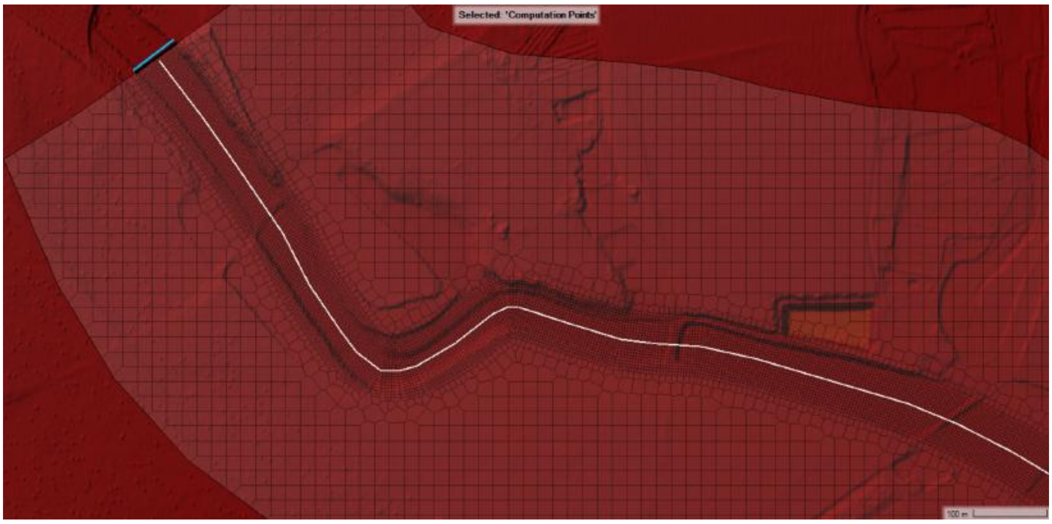
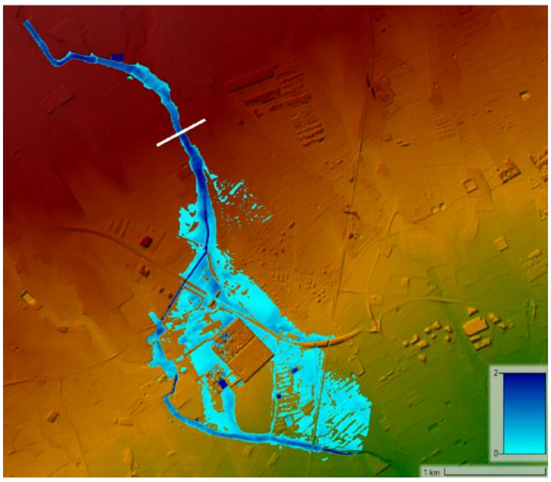


Figure G.12: Detailed view of the computational grid for the situation without a structure. The white line corresponds to the breakline of the refined region and the blue line is the location of the upstream boundary condition. The grid is displayed on the terrain layer.

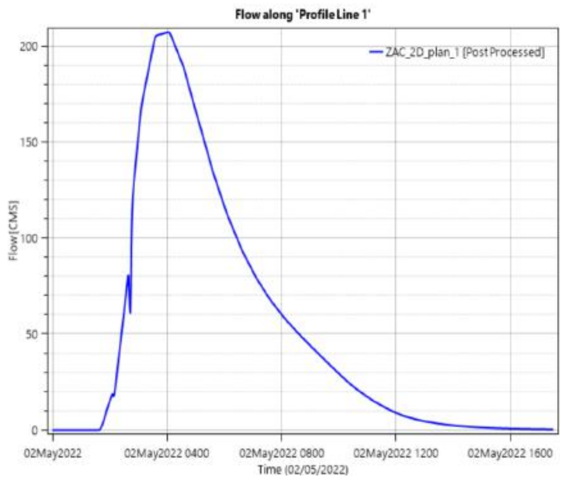
Next, the computational time step is determined. This is done using the Courant number principle. A simulation is stable if the Courant number is at least smaller than 1, and preferably smaller than 0.7 (Theresa, 2022). This is to reduce the chance of skipping cells during the computation. Assuming a Courant number of 0.7, the computational time step is calculated using formula G.3 below.

$$\Delta T = 0.7 * \frac{\Delta X}{u} \tag{G.3}$$

After running a simulation with a coarse time step, the maximum velocity u was around 4 m/s. This in combination with a minimum grid size of 2 x 2 m results in a computational time step of around 0.3 seconds. Figure G.13 shows the result of this simulation.



(a) Result of a simulation without a ZAC structure. After the white line the river starts to overflow. This is also the location from which the hydrograph is taken.



(b) Flow hydrograph for the situation without a ZAC structure.

Figure G.13

Step 4

After the initial situation is modelled, the ZAC structure is modelled. The first step was to determine the size of the project location, which was set to be the complete area of project location A. As stated in Chapter 6, the most important parameters to test in the model are: the culvert spillway structure and the number of reservoirs. Within the culvert spillway structure there are a number of smaller parameters that are optimized. For example: the height, width and shape of the culvert and spillway, whether the structure will have flared walls for flow direction and various energy loss parameters. Also, the maximum water depth in the basins are determined.

All the testing for these parameters is done in a structured manner starting with the parameter with the largest influence on the peak discharge reduction: the number of reservoirs. In order to determine the optimal number of reservoirs, arbitrary dimensions for the culvert spillway structure are used, as this is only to determine the optimal number of reservoirs. The dimensions of each reservoir is roughly equal to 250 m in length and 300 m in width. With every consecutive simulation, a reservoir is added to the ZAC. The results of these simulations are shown in Figure G.14.

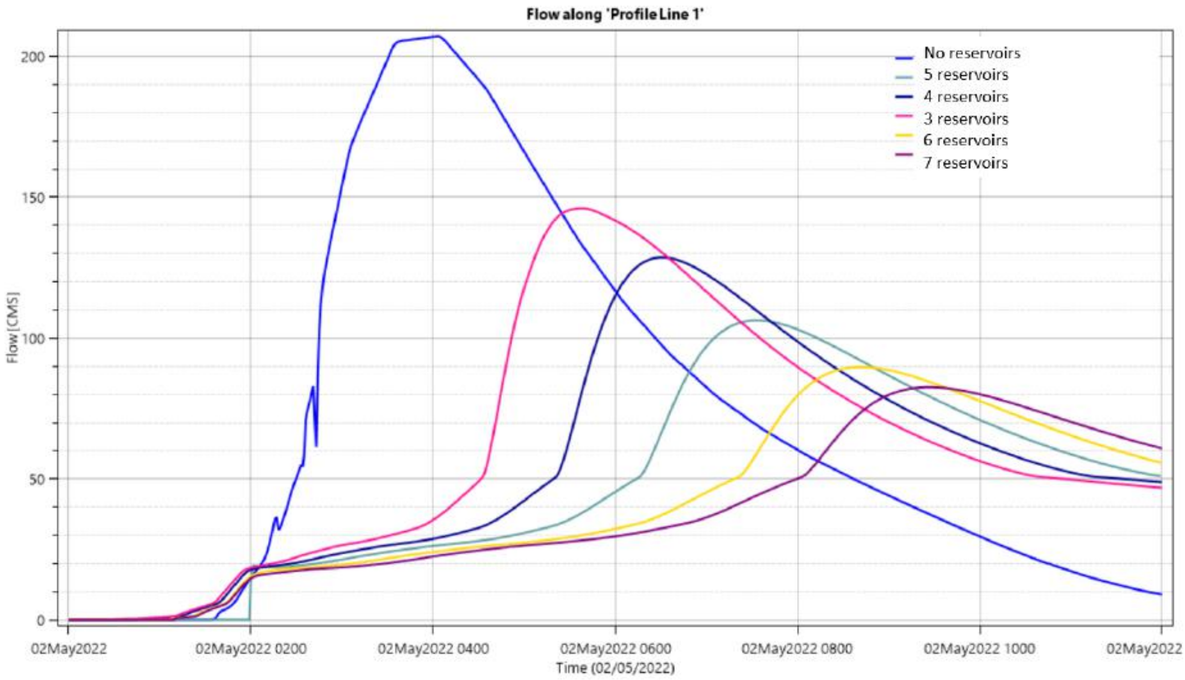
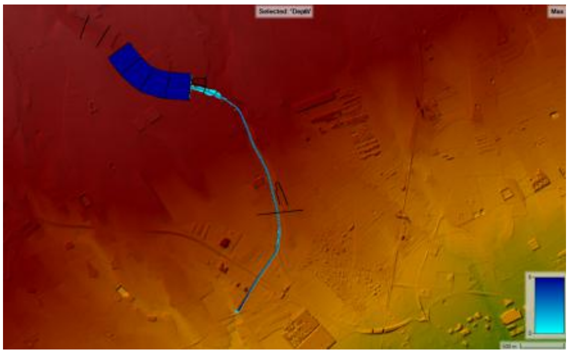
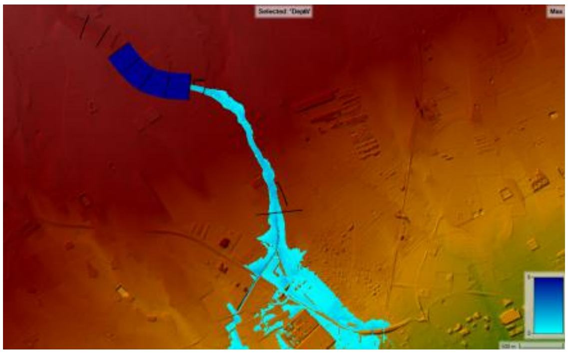


Figure G.14: This figure shows the outcome of the simulations for determining the optimal number of reservoirs. The effect a certain number of reservoirs has on the peak discharge reduction is displayed in the hydrographs. Around 50 m^3/s, every graph shows a sudden change. This is when the spillways start to work.

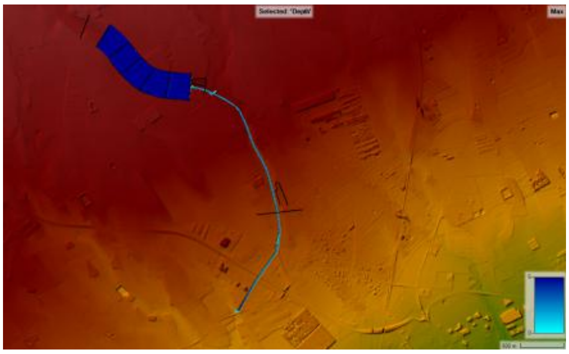
Figure G.14 shows a reduction in peak discharge and an extension of the outflow duration of the ZAC every time an extra reservoir is added. However, the reducing effect of adding an extra reservoir decreases when the ZAC increases in size. To clarify, the effect from three to four reservoirs results in a peak discharge reduction of 20 m^3/s and the effect from six to seven reservoirs results in a peak discharge reduction of only 7 m^3/s. This is an important aspect in choosing the optimal number of reservoirs for the ZAC. A different way to show the effect an extra reservoir has on the downstream area is to get a flood map of the area downstream of the ZAC. Figure G.15 shows the flood map for a ZAC with four, five and six reservoirs, all of which are modelled with and without a downstream channel.



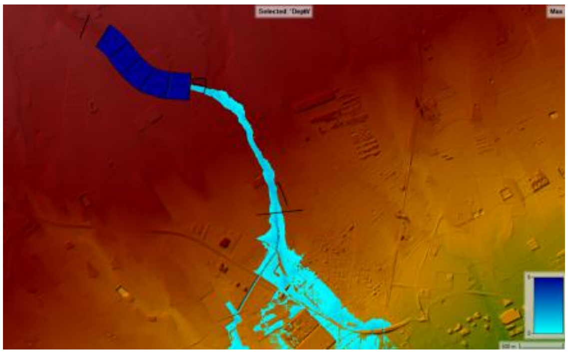
(a) Situation: four reservoirs with a downstream channel. There is no flood in the area directly downstream.



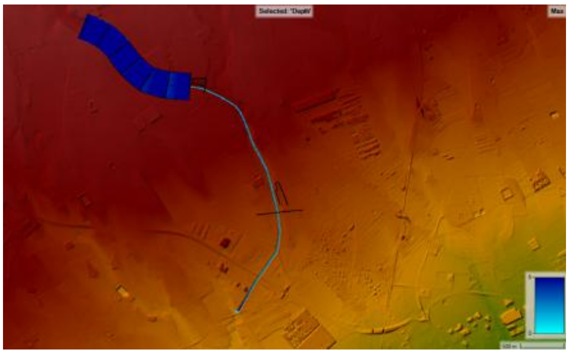
(b) Situation: four reservoirs without a downstream channel. The flooded area is covered by roughly 70 cm of water.



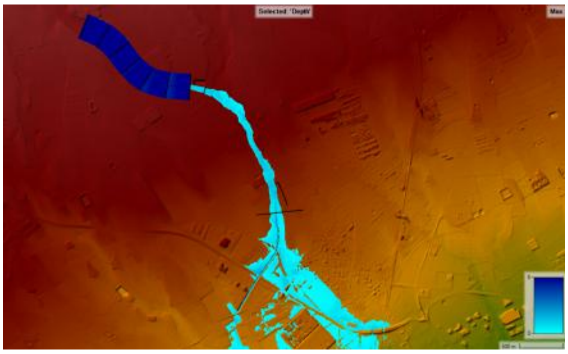
(c) Situation: five reservoirs with a downstream channel. There is no flood in the area directly downstream.



(d) Situation: five reservoirs without a downstream channel. The flooded area is covered by roughly 50 cm of water.



(e) Situation: six reservoirs with a downstream channel. There is no flood in the area directly downstream.



(f) Situation: six reservoirs without a downstream channel. The flooded area is covered by roughly 40 cm of water.

Figure G.15: Flood map of the area downstream of the ZAC at the time the flood is maximum for each situation.

Based on this information, five reservoirs are optimal for this design. This also confirms the need for a downstream channel, as the maximum discharge capacity of the current downstream channel ($51\,m^3/s$) is exceeded. Based on Figure G.15, four reservoirs will not result in a flood of the downstream area. However, the outflow duration is also an important parameter. This value cannot be too large, because downstream of the model the river confluences with another reach, which would cause a flood even further downstream. Based on Figure G.14 the outflow duration increases when adding extra reservoirs. The reason not more than five reservoirs are used is because the advantage gained from adding an extra reservoir will probably not justify the extra costs. There is no exact cost benefit analysis at hand, but assuming that adding one extra reservoir has a constant price and that in combination with the fact that from Figure G.14 it follows that the effect of an extra reservoir reduces, it is said that more than five reservoirs is not beneficial.

In a case of less reservoirs, the peak discharge reduction effect of the ZAC is considered not large enough, which eventually would have to be compensated by a larger downstream channel cross-section. This is also undesirable as it diminishes the aim of the ZAC.

Now that the number of reservoirs is known, the dimensions of the culvert spillway structure is optimized. As mentioned in Section 3.3, the maximum allowable discharge for the culvert is equal to 51 m^3/s. For the culvert to be modelled, HEC-RAS requires multiple input parameters, like the shape, size, energy loss coefficients and manning values. A detailed explanations for the exact values of these parameters and the reasoning behind them is given in Chapter 6: functional design. The spillway is modelled just above the culvert as a local reduction in the dike elevation. The only other parameter is the weir coefficient C.

In order to determine the dimensions and shape of the culvert, the spillway was initially left out as to not disturb the downstream hydrograph. The dike elevation was set at a height at which the spillway was expected to be. This way, the maximum discharge through the culvert can be determined by simply looking at the downstream hydrograph and verifying the location of the sudden change of the hydrograph. This indicates the time at which the dike overflows and the thus when the culvert is at its maximum capacity. Then, the dike level was increased and the spillway was added. This spillway was designed to be sufficiently large (by changing its dimensions) and effective (by changing the weir coefficient). The spillway has the correct dimensions when the dike does not overflow and all the excess water is discharged by the spillway.

The application of the 2D model regarding this project has some limitations. The main problem might be the computational method. HEC-RAS is able to use two computational methods: the 2D diffusion wave method and the 2D Saint Venant full momentum method. The computational method used for this project is the 2D diffusion wave method. The reason this method is used is because it is much faster and robust than the Saint Venant method. In other words, the computational power to solve this problem with the Saint Venant method is not at hand for this project. The model of the ZAC involves sudden expansion or contraction of flow with high velocity changes, dynamic flood waves and detailed flow around hydraulic structures (i.e., the culvert and spillway). These are situation where the 2D Saint Venant method would be more accurate than the 2D diffusion wave method (CivilGEO Inc., 2022).

H Risk of 2 large rainfall events

In Chapter 3 it is stated that the occurrence of two extreme discharge events within a few days is negligibly small. The calculation is given in this Appendix.

The structure is being designed for a design life time of 50 years, with a probability of failure of 10 % during its life time. The ZAC is being designed to have a capacity of $3.3 * 10^6\ m^3$. In this section it is disproved that the occurrence of 2 large rainfall events is the determining factor of the ZAC.

If 2 rainfall events occur in very fast succession, the storage volume of the ZAC can be exceeded. The volume of a once in 474-year rainfall event leads to a storage volume of $3.3 * 10^6\ m^3$. When 2 rainfall events of equal volume happen that have the same volume as the total storage volume, the return period is 66 years. So the volume of 2 66-year rainfall events is the same as a once in 474-year rainfall event. This has been concluded from scaling the hydrograph and its peak discharge when the return period was reduced from 474 years to 50 years. The volume of 2 66-year rainfall events is $3.3 * 10^6\ m^3$, meaning $1.65 * 10^6\ m^3$ per rainfall event. Even if 2 of these rainfall events occur in 2 days, the storage capacity of the ZAC would still be sufficient. In this case, the time until the second rainfall event is not even considered. That means in reality, during the time the ZAC awaits the second rainfall event while the first has passed, the outflow during the pause is not even considered.

Thus, the probability of 2 rainfall events with a return period of 66 years in 2 days must be examined.In the rest of this chapter, a rainfall event is meant as the 66-year rainfall event. If the occurrence of a rainfall event is considered a success p, the years with no rainfall event are q. For a 1 in 66-year rainfall event:

- $p = 1/66$
- $q = 65/66$

The chance of a rainfall event happening in 50 years follows the binomial distribution:

$$P(x\, rainfall:\ events) = \binom{N}{x} \cdot p^x q^{N-x} \tag{H.1}$$

Where:
- N = number of years in design life
- x = number of years with a design rainfall event

For 0 rainfall events in 50 years this means:

$$P(0\, rainfall:\ event) = \binom{50}{0} \cdot p^0 q^{50} = (\frac{65}{66})^{50} = 0.466[-] \tag{H.2}$$

For 1 rainfall event in 50 years this means:

$$P(1\, rainfall:\ event) = \binom{50}{1} \cdot (\frac{1}{65})^1 (\frac{65}{66})^{50-1} = 50 * 0.0072 = 0.359[-] \tag{H.3}$$

The probability of a combination of 1 rainfall event in a particular year can be 50 different solutions: $[p, q, ..., q]$, or $[q, p, q, ..., q]$, ..., or $[q, ..., q, p]$. Therefore, the probability of 1 rainfall event in those 50 years is multiplied by a factor 50.

For the chance of 2 rainfall events in the given period of 50 years, any combination of rainfall event years is given by:

$$P(2\,rainfall:\ events) = \binom{50}{2} \cdot p^2 q^{48} = \frac{50 * 49}{2!} * 1.1 * 10^{-4} = 0.135[-] \tag{H.4}$$

Here the first rainfall event can be in any year, 50 spots, then the second rainfall event can be in 49 remaining years. The factor is divided over 2!, which represents the number of combinations the p can have. The occurrence of 3 rainfall events in 50 years has an even smaller probability, namely:

$$P(3\,rainfall:\ events) = \binom{50}{3} \cdot p^3 q^{47} = \frac{50 * 49 * 48}{3!} * 1.6972 * 10^{-6} = 0.033[-] \tag{H.5}$$

Smaller values by 4 or more rainfall events are neglected because of their even smaller probability. The chance of 2 or more rainfall events happening in the life time period of 50 years is:

$$P(X \geq\ 2) = 1 - P(0\,rainfall:\ events) - P(1\,rainfall:\ event) = 1 - 0.466 - 0.359 = 0.175[-] \tag{H.6}$$

From the above, only $0.175 - 0.135 - 0.033 = 0.007[-]$ is missing, so this is neglected for the sake of the calculation. It is important to note that not all combinations of years that have rainfall events are the right combinations. In fact, when there are 2 rainfall events, only 49 combinations of $[p, p, q, ..., q]$ exist so that there are 2 consecutive years with the 66 year design rainfall event. For 3 rainfall events, there are more combinations that lead to 2 consecutive rainfall events (49*46), but the probability of any combination of 3 rainfall events is smaller than 2 rainfall events. This means that the total probability of 2 consecutive rainfall events is given by:

$$P(2\,consecutive\ years) = 49 * 1.1 * 10^{-4} + \frac{49 * 48}{2} * 1.6972 * 10^{-6} = 7.47 * 10^{-3}[-] \tag{H.7}$$

However, this is for 2 consecutive years and not for 2 consecutive days. The ZAC is empty after flowing out for 18 hours from the start of the rainfall event. This means that 2 66-year rainfall events need to occur in 2 days in order to interfere with each other. Given that you are in 2 consecutive years that rainfall events occur, the first rainfall event has a probability of 1.0, as any day is correct. Then, the second rainfall event must be either the day before or the day after, meaning there is a probability of 2/729 that 2 66-year rainfall events occur in 2 days. To make the step from years to days, the total probability is:

$$P(2\,consecutive\ days \mid 2\,consecutive\ years) = \frac{2}{729} * 7.47 * 10^{-3} = 2.05 * 10^{-5} \tag{H.8}$$

This is 1 in 50,000 which is astronomically low. This is logical, as the regular interval between 66-year rainfall events is 66 years (more than 24,000 days). And even with this return period between the design rainfall events, the ZAC could still store the volume of the rainfall events. This risk is considered acceptable for the design.

It should be noted that in this written case the rainfall event events occur independently. In reality, large meteorological events can have some degree of dependency, especially in the order of days. However, this is out of the scope of this design, and the dependency has therefore been neglected. It is assumed that the resulting chance of 2 66-year rainfall events hitting in 2 days will then still be acceptably low.

In conclusion, the ZAC is able to store 2 consecutive rainfall events with a return period of 66 years. This is considered acceptable due to the low probability of occurrence. Therefore, the occurrence of multiple rainfall events in this project is not further investigated.

I Culvert and spillway alternatives and optimal design

Section 6.2.2 gives the functional design of the culvert spillway structure. This appendix elaborates and supports this design loop.

I.1 Multi-criteria analysis building materials

Durability
This criterion evaluates the resistance of the building materials to the surrounding environment, without them experiencing damage or regular maintenance.

Due to the corrosive nature of steel, it requires regular maintenance, which is not ideal for a remote location. Like steel, timber is susceptible to the surrounding environment. The (partly) submerged nature of the structure is a source of abiotic deterioration of the timber components (Skyciv, 2019). This also leads to regular maintenance. If designed correctly, concrete is a very durable building material. Due to this high durability, it has low maintenance costs compared to steel and timber.

Environmental impact
This criterion evaluates the environmental impacts of the building materials, taking into account the energy consumption and the expelled emissions.

The production of steel is highly energy-intensive. This results from the fact that high temperatures are needed to oxidize the unwanted elements in the steel composite (Anejo,2014). This high energy consumption leads to a severe environmental impact.

During the production of concrete, with emphasis on cement, large amounts of greenhouse gases are emitted. These large emissions lead to a high environmental impact. Furthermore, energy consumption is highly correlated to CO2 emissions (Anejo,2014), since the CO2 emission during cement production is large. It can be concluded that the production of concrete is highly energy and emission-intensive, which leads to an overall high environmental impact compared to steel and timber.

Compared to steel and concrete, the production of timber has the lowest CO2 emissions. Since the energy consumption is highly correlated to CO2 emissions, it can also be concluded that the production of timber has the lowest energy consumption (Anejo, 2014). This leads to timber having the overall lowest environmental impact, compared to steel and concrete.

Execution
The construction of the structural components differ for the three materials. For the timber and steel elements, extra attention has to be paid to the connections. Steel connections can be either welded or bolted. Welded connections have a longer execution time due to the specialistic work. Bolted connections, either applied to steel or timber, require regular maintenance since corrosion of the bolts can occur. Concrete components can be connected through anchors, if for example the elements are prefabricated. Otherwise, the concrete elements are cast together or connected with a grout layer.

MCA building materials results
The structure has a service life of 50 years and lies in a remote location. Based on that, durability and maintainability are taken as the governing criteria for the structure. Staying in line with the European green deal, to become the first climate-neutral continent by 2050 (Norton Rose Fullbright, 2021), the environmental impact criterion has a significant influence on the selection of the building material.

Table I.1: MCA for building materials

	Weight factor	Steel	Timber	Timber
Durability	5	1	1	5
Maintainability	5	2	2	5
Environmental	4	2	5	1
Execution	3	2	3	4
Score		29	44	66

I.2 Spillway structure alternatives

Shapes spillway structures

Rectangular spillway
The spillway is designed with a hollow rectangular cross-section, essentially a box girder. The spillway is designed as hollow to reduce the amount of building materials. This leads to a lower environmental impact and lower overall costs since fewer materials are needed. On the upstream side of the spillway, the flow increases gradually, until it flows over the spillway and experiences a free fall on the downstream side. Due to the free fall, the structural component below experiences an impact load. Which component experiences this extra loading depends on the chosen final design.

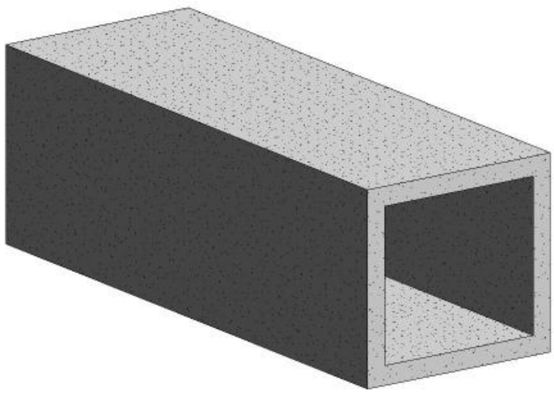


Figure I.1: Hollow rectangular spillway

Right-angled trapezoidal spillway
The spillway is designed with a hollow right-angled trapezoidal cross-section. As with the rectangular cross-section, the hollow design reduces the amount of building materials, which lowers the overall costs and environmental impact. The right-angled slope is located on the upstream side of the structure, and the sloped side is on the downstream side of the structure. The slope at the downstream side of the spillway allows for a more gradual flow over the spillway, and not a sudden drop compared to the rectangular spillway.

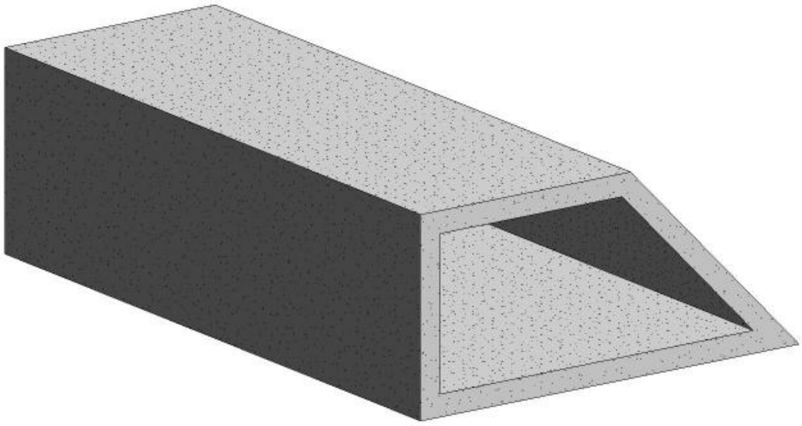


Figure I.2: Hollow right-angled trapezoidal spillway

Trapezoidal spillway
The spillway is designed with a hollow trapezoidal cross-section. As with the rectangular cross-section, the hollow design reduces the amount of building materials, which lowers the overall costs and environmental impact. The slope is located on the upstream and downstream sides of the structure. The slope at the downstream side of the spillway allows for a more gradual flow over the spillway, and not a sudden drop compared to the rectangular spillway. According to the paper of Azimi, Rajaratnam, & Zhu (2013) , the slopes, primarily the upstream slope, increase the flow efficiency of the spillway.

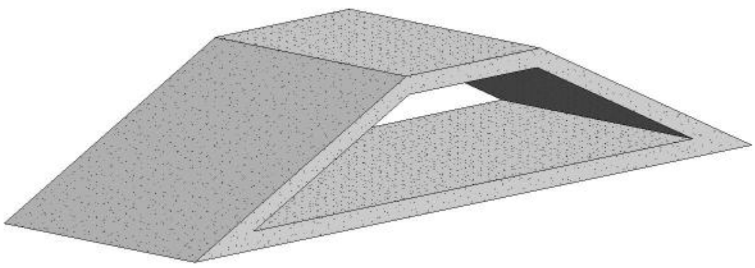


Figure I.3: Hollow trapezoidal spillway

Dimensions spillway alternatives

Rectangular broad crested spillway

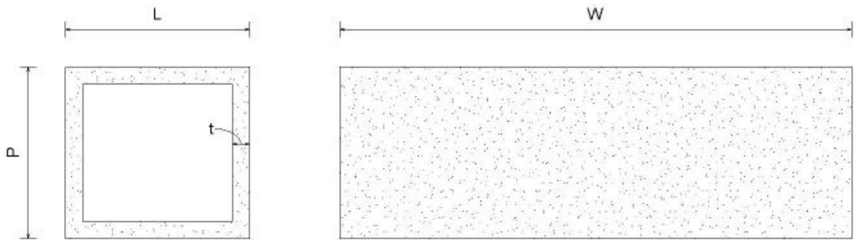


Figure I.4: Assumed dimensions rectangular broad crested spillway

- W = 7 meters
- P = 4.5 meters
- t = 0.5

The minimum length for the broad-crested spillway is selected in order to achieve the minimum amount of building materials.

- L = 5.0 meters
- Area spillway = 8.5 m^2
- Volume spillway = 8.5 m^2 * 7m = 59.5 m^3

According to the paper of Sargison & Percy (2009), the discharge coefficient for a broad-crested weir with squared-edge upstream corners ranges from 0.76-0.88. For a ratio of h/L = 0.4, a discharge coefficient of 0.85 is assumed.

Rectangular sharp-crested spillway

Figure I.5: Assumed dimensions rectangular sharp-crested spillway

- W = 7 meters
- P = 4.5 meters
- t = 0.5

To stay on the conservative side for the loads acting on the spillway, the maximum length for a sharp-crested spillway is selected.

- L = 1.0 meters

- Area spillway = 4.5 m^2
- Volume spillway = 4.5 m^2 * 7m = 31.5 m^3

The discharge coefficient for a sharp-crested spillway is determined based on the paper of Arvanaghi & Oskuei (2013). This gives a discharge coefficient of 0.65.

$$Cd = 0.611 + 0.08 * (h/P) = 0.611 + 0.08 * (0.444) = 0.65$$

Right-angled trapezoidal broad crested spillway with a downstream ramp

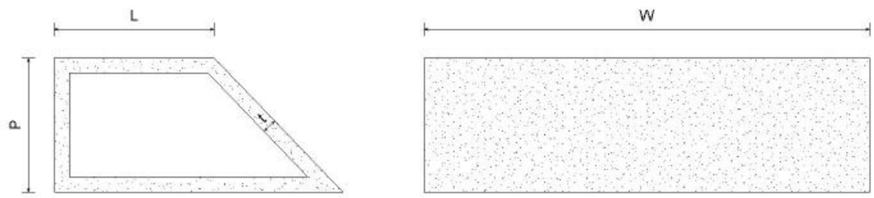


Figure I.6: Assumed dimensions of right-angled trapezoidal broad crested spillway with a downstream ramp

As previously indicated, the water depth over the spillway crest is 2 meters. So, to be on the conservative side, it is assumed that the water depth over the entire spillway is 2 meters. Taking this into account and seeing that according to the paper of Azimi, Rajaratnam, & Zhu (2013), the downstream slope has no significant effect on the flow efficiency of the spillway, it is assumed that the downstream ramp has the same slope as the surrounding dikes. For the horizontal part of the spillway, the minimum broad-crested length is selected in order to reduce the amount of building materials.

- P = 4.5 meters
- t = 0.5
- Lw = 5 meters
- Ld = 14.28 meters
- alpha = 25.02
- Area spillway = 15.24 m^2
- Volme spillway = 15.24 m^2 * 7 m = 106.7 m^3

Azimi, Rajaratnam & Zhu (2013), produced a paper where an overview of experimentally determined discharge coefficients for broad-crested weirs is given. Based on these experimental investigations and taken into account the spillway dimensions, the discharge coefficient for the broad-crested trapezoidal spillways are determined.

$$h/(h + Lw) = 2/(2 + 5) = 0.285- > Cd = 0.84$$

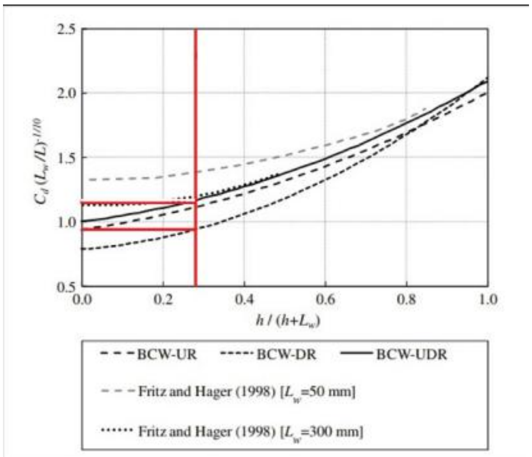


Figure I.7: Discharge coefficient for broad-crested spillway with downstream ramp (right-angled trapezoidal spillway) and discharge coefficient for broad-crested spillway with downstream and upstream ramp (trapezoidal spillway)

Trapezoidal broad crested spillway

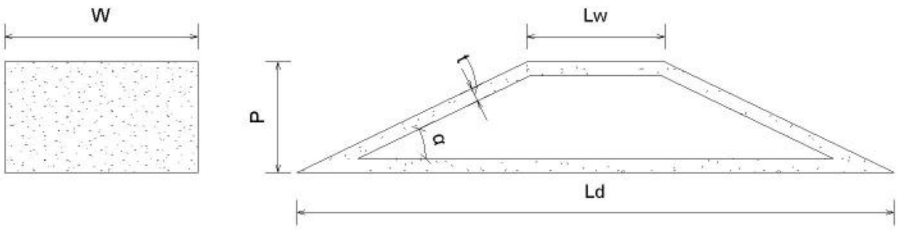


Figure I.8: Assumed dimensions for broad crested trapezoidal spillway

As with the spillway with a downstream ramp, the water depth above the entire spillway is assumed to be 2 meters. The dimensions of the spillway are selected to maximize the flow efficiency of the spillway. These are based on the paper of Azimi, Rajaratnam & Zhu (2013), meaning both ramps have a slope of m = 2, and the length of the horizontal part of the spillway should be minimal. In this case, it is equal to the minimum length for a broad-crested spillway.

- P = 4.5 meters
- t = 0.5
- Lw = 5 meters
- Ld = 14.28 meters
- Lu = 9 meters
- alpha = beta = 25.02
- Area spillway = 19.31 m^2
- Volme spillway = 19.31 m^2 * 7 m = 135.2 m^3

Similar to the previous spillway, the discharge coefficient is also determined based on the paper of Azimi, Rajaratnam & Zhu (2013).

$$h/(h + Lw) = 2/(2 + 5) = 0.285- > cd = 1.1$$

MCA spillway

Maintainability
Due to the slender nature of the sharp-crested spillway, it is expected that abrasion of the structure will occur. Due to the abrasion, the sharp-crested spillway would require regular maintenance compared to the broad crested spillway. This is also an issue for the broad-crested spillway, only in a lower degree, since the spillway length is larger. It is also expected that abrasion will occur on the slopes of the trapezoidal spillways. This effect is smaller compared to the rectangular spillway, since the slopes are not sharp-cornered. This leads to the trapezoidal spillway to require less maintenance.

Environmental impact
For the environmental impact, only the amount of building materials are considered. After all, as indicated in Section 7.3, fewer building materials lead to an overall lower environmental impact. In the same section, it is determined that the most optimal building material is concrete. Table I.2 gives an overview of the approximate amount of building materials per spillway type, assuming the spillways are constructed with concrete. For a rough estimate of the amount of reinforcement, it is approximated to be 1.6 % of the concrete weight (Materials & Environment-Sustainability group).

Table I.2: MCA for spillway types

Spillway type	Volume $[m^3]$	Density concrete $[kg/m^3]$	Concrete weight [kg]	Approximation reinforcement weight [kg]	Amount of materials [kg]
Trapezoidal spillway	135.2	2400	324.46*10^3	5191.4	329652
Right-angled trapezoidal Broad crested spillway with downstream ramp	106.7	2400	256.06*10^3	4096.9	260155
Rectangular Broad crested spillway	59.5	2400	142.80*10^3	2284.8	145085
Rectangular Broad crested spillway	31.5	2400	75.6*10^3	1209.6	76809.6

Difficulty of constructability
This criterion depends on the selected building material of the spillway. It is determined that the most optimal building material is concrete. Assuming that concrete is applied, the various types of spillways can all be constructed either in-situ or as pre-cast.

It is expected that the rectangular spillways are easier to construct and are less labour-intensive compared to the trapezoidal spillways since it is essentially a box-girder. This is assumed, since the box girder is a standard structure, with standard reinforcement and standard design methods. For the trapezoidal spillways, slope accuracy needs to be guaranteed, and reinforcement placement needs to be thought of in the sloped area. This leads to the trapezoidal spillways being more difficult to construct, hence being more labour-intensive.

Effectiveness of the structure on its function
This criterion evaluates the hydraulic performance of the spillway. The hydraulic performance is based on the flow efficiency of the spillway, this efficiency is based on the discharge coefficient, Cd. The higher the coefficient, the more efficient the flow is. Table I.3 gives an overview of the previously calculated discharge coefficients.

Table I.3: Discharge coefficient per spillway type

Spillway type	Discharge coefficient (Cd)
Broad crested trapezoidal spillway	1.1
Right-angled trapezoidal Broad crested spillway with downstream ramp	0.84
Rectangular Broad crested spillway	0.85
Rectangular Broad crested spillway	0.65

Based on the discharge coefficient, the broad-crested trapezoidal spillway is the most efficient. This is understandable since the upstream slope reduces the separation zone at the entrance of the spillway.

MCA Shape Spillway
The spillway is constructed for its hydraulic performance. Thus, functionality is the governing criterion. The structure has a service life of 50 years and lies in a remote location. Based on that, maintainability is also taken as a governing criterion for the structure. Nevertheless, constructability is an important aspect for the design of the structure. Staying in line with the European green deal, to become the first climate-neutral continent by 2050 (Norton Rose Fullbright, 2021), the environmental impact should also be taken into account for the selection of the spillway.

Table I.4: MCA of shape of the spillway

Spillway type	Weight factor	Rectangular sharp-crested	Rectangular broad-crested	Trapezoidal	Right-angled trapezoidal
Effectiveness of the structure on its function	5	2	3	5	3
Maintainability	4	1	2	4	3
Difficulty of constructability	3	5	4	2	3
Environmental impact	3	5	4	1	2

I.3 Culvert structure alternatives

Culvert shape alternatives

Rectangular culvert
The culvert is designed as a rectangular box girder without the bottom slab. The bottom slab in this project is not needed, since the culvert is constructed on top of the foundation. The absence of the bottom slab leads to fewer building materials, which leads to a lower environmental impact and overall costs. A rectangular box shape is structurally less efficient compared to other culvert shapes, due to the flat sides and top. The standard range of span size lies between 1 to 4 meters. Otherwise, more spans are required, which means that the culvert becomes of a multiple barrel type.

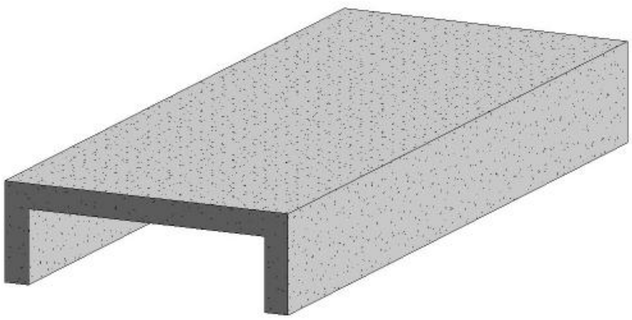


Figure I.9: Sketch Rectangular culvert

Arched culvert
This culvert opening has an arched top and a flat bottom. This culvert shape requires more material compared to the rectangular box shape in order to obtain the same cross-sectional area. Arched shapes are more structurally efficient than the rectangular box shape, due to the load distribution, this is explained in the following section. Because of this, it is expected that spans of 7 to 12 meters can be obtained.

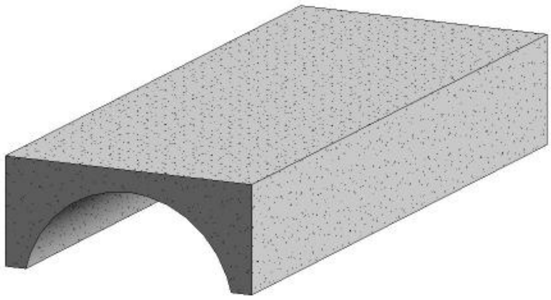


Figure I.10: Sketch Arched culvert

Multiple circular culverts
Usually multiple culvert openings are applied for wide channels because a single culvert opening requires a too large span or in most cases, multiple openings are necessary to obtain a sufficient flow capacity because of a low embankment. In some locations, this particular type of culvert is more prone to clogging, due to the accumulation of sediment and debris between the culvert openings. This leads to a higher need for maintenance. An advantage of this type of culvert is that a larger span can be obtained, this is explained in the following section.

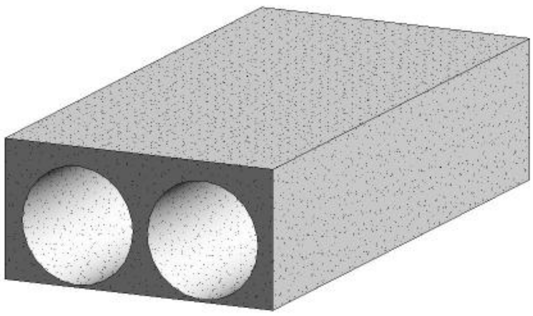


Figure I.11: Sketch multiple circular culverts

Dimensions culvert

Based on the requirements of the culvert and the evaluated culvert shapes, the following dimensions and technical drawings are constructed. Because of the discharge flowing through the culvert following from the 2D HEC-RAS model, the cross-sectional area of the opening of the culvert should be 7.5m^2. As a guideline, the width of the bottom of the culvert opening is set to the same width of the channel (the horizontal part), which is 5.0 meters.

Rectangular culvert
In Figure I.9 the dimensions of the rectangular culvert are shown. Since a span of 5 meters is too large for a rectangular culvert box with one opening, the choice is made to apply two culvert openings that are separated by an intermediate wall. The two openings have a larger height in order to achieve the same required cross-sectional area of the culvert opening. For an initial design, the concrete thickness is set to 500 mm. This is evaluated further in the structural design phase.

Arched culvert
In Figure I.10 the dimensions of the arched culvert are shown. The height of the arch and the radius of the circular arc of the cross-sectional area of the opening is calculated with a base level of 5.0m and the required area of 7.5m^2. From this follows that the height should be 2.01m with a radius of 2.56m. For an initial design, the concrete thickness is set to 500 mm. This is evaluated further in the structural design phase.

Multiple circular culverts
In Figure I.11 the dimensions of the multiple circular culverts are shown. The area of the circular openings are equal and have a total area of 7.5 m^2. From this the radius is calculated, which is 1.1m. The dimensions of the total culvert cross-section are adjusted to the available space of the ZAC structure. For an initial design, the concrete thickness is set to 500 mm. This is evaluated further in the structural design phase.

MCA Shape Culvert

Structural efficiency (Williams et al, 2012)
The structural efficiency of a structural component such as a culvert depends on the efficiency of distributing the loads acting on it. It is assumed that the culverts are constructed with concrete. The concrete design uses strut and tie modelling for determining where compressive and tensile forces will occur. In strut and tie modelling, vertical loads are transferred not vertically but with an angle. Also, loads are distributed along the stiffest and strongest parts of the structural component. The most realistic model is the one that features the fewest and shortest ties.

With this knowledge, the shapes of the culvert cross-sections also have different structural efficiencies regarding the load distribution and thus the placement of reinforcement since the ties represent the steel reinforcement. This is therefore an important criterion for determining beforehand which culvert shape is the most optimal for the ZAC structure.

Based on the hydraulic structures manual, and keeping the strut and tie modelling in mind, a rectangular box cross-section is the least favourable option. An arched or circular-shaped cross-section would be the better solution.

Maintenance
As mentioned in the previous section, a culvert with multiple barrels is more prone to the accumulation of sediment and debris between the different openings. This leads to a higher need for maintenance, which is not desirable. The difference between an arched and rectangular box cross-section with respect to the maintainability is not significant. An arched shape is preferable over a rectangular cross-section because of fewer re-entrant corners in the cross-section, which are prone to concrete cracks. This could lead to a higher need for maintenance.

Material and environment
The preliminary choice of a culvert shape also strongly depends on the span that can be obtained. This is also related to structural efficiency. More material use, for the same function, leads to a larger environmental impact. A multiple barrel culvert can have larger spans. However, it requires more material because of the intermediate walls. A rectangular box cross-section can have spans up to 4 meters. Arched shaped culverts can span up to 12 meters, which is the most efficient, regarding material use. With the same culvert opening area, the arched shaped cross-section is larger than a rectangular box section. However, it is expected that due to the larger spans of the arched shape, it still is more favourable with respect to material use.

Table I.5 gives an overview of the approximate amount of building materials per culvert type, assuming the culverts are constructed with concrete. For a rough estimate of the amount of reinforcement, it is approximated to be 1.6 % of the concrete weight (Course CIE4100, slide 91).

Table I.5: Material properties per culvert alternative

Culvert type	Volume [m3]	Density concrete [kg/m3]	Concrete weight [kg]	Approximation reinforcement weight [kg]	Amount of materials [kg]
Rectangular	38.375	2400	9.21*10^4	1473.6	93573.6
Arched	50.35	2400	1.21*10^5	1933.44	122773.4
Multiple circular	56.83	2400	1.36*10^5	2182.272	138574.3

It can be seen that the multiple circular culvert has the highest amount of building materials and the rectangular culvert the lowest. This is understandable, seeing the volume per culvert.

MCA shape Culvert
The culvert is constructed for its hydraulic performance. Thus, structural efficiency is the governing criterion. The structure has a service life of 50 years and lies in a remote location. Based on that, maintainability is also taken as a governing criterion for the structure. Nevertheless, constructability is an important aspect for the design of the structure. Staying in line with the European green deal, to become the first climate-neutral continent by 2050 (Norton Rose Fulbright, 2021), the material and environment criterion should also be taken into account for the selection of the culvert.

Table I.6: MCA of culvert shape

	Rectanglar culvert	Arched culvert	Multiple circular culvert
Structural efficiency	2	5	4
Maintenance	1	4	2
Material and environment	4	3	2
Score	26	50	34

From the MCA follows that the arched culvert is the overall optimal culvert.

I.4 Flared walls

With the 2D HEC-RAS model, the influence of the flared walls on the ZAC structure is calculated. The pre-determined requirements of the culvert and spillway are kept constant during the simulations. Table I.7 gives an overview of the HEC-RAS inputs and outputs.

Table I.7: Flared walls results

Input		Output	
Structure	Contraction coefficient	Discharge [m^3/s]	Velocity [m/s]
Flared walls (angle between 30-75)	0.3	50.48	5.8
No flared walls	0.7	50.58	5.7

As can be seen, the presence of the flared walls has no significant effect on the discharge or flow velocities of the ZAC structure. Therefore, in the case that flared walls are present, the construction mainly depends on practical reasons, naturally taking the acting loads on the structure into account.

Hydrograph with flared walls (Cd = 2.56)

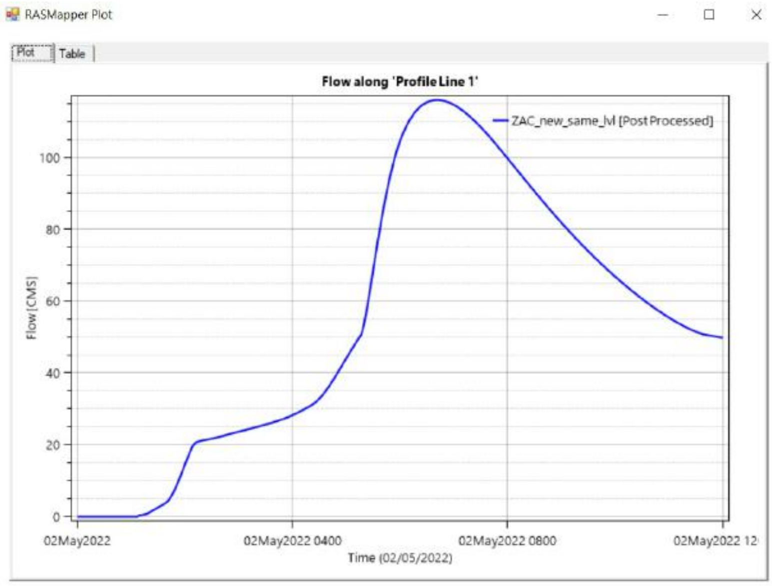


Figure I.12: Flow hydrograph of ZAC structure with flared walls

Hydrograph no flared walls (cd = 2.56)

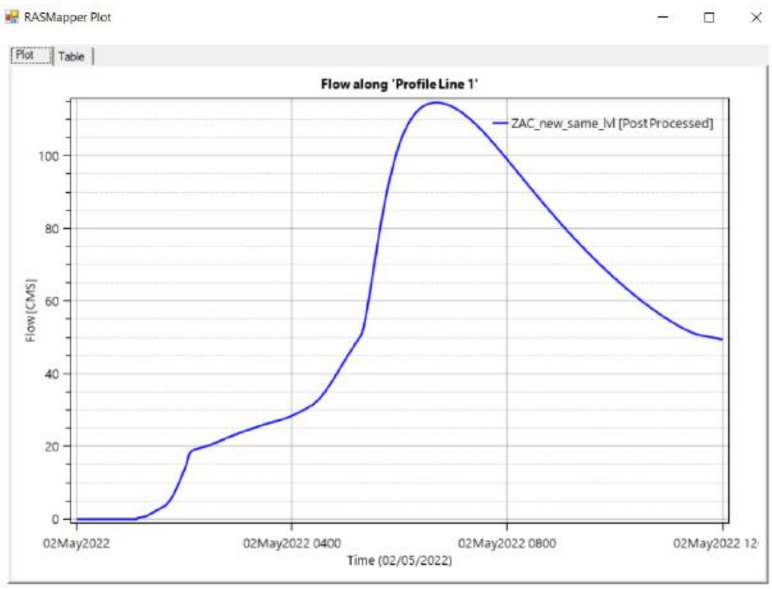


Figure I.13: Flow hydrograph of ZAC structure without flared walls

I.5 Design selection of the culvert-spillway structure

Based on the previously constructed MCA per structural component, three design alternatives are constructed. Each design alternative focuses on a different criterion. The first design alternative focuses on the performance of the structure, which covers the flow efficiency of the spillway and the structural efficiency of the culvert. The second design alternative focuses on the maintainability of the structure. And the third design alternative focuses on the material use and environmental impact of the structure, hence the structural components with the minimal material use.

For all the design alternatives, it is decided not to construct flared walls since it has no added value to the ZAC structure and would only increase the amount of building materials.

Table I.8: MCA results of the structural components for the determining criteria

	Weight factor	Rectangular sharp-crested	Rectangular broad-crested	Trapezoidal	Right-angled trapezoidal
Performance	5	2	3	5	3
Maintainability	4	1	2	4	3
Material use	3	5	4	1	2
Score		24	31	43	31
	weight factor	Rectangular culvert	Arched culverty	Multiplw circular culvert	
Performance	5	2	4	4	
Maintainability	3	1	4	2	
Material use	2	4	3	2	
Score		21	43	30	

As shown in tableI.8, the structural components with the highest rank for performance, are identical to that for maintainability. Thus, two alternatives are considered instead of three. The first is constructed with the highest rank for performance and maintainability. The second is the alternatives which has the lowest value of material use.

Two alternatives are considered. The first is constructed with the highest rank for performance and maintainability. The second is the alternatives which has the lowest value of material use.

Alternative 1

The structural components with the highest rank for performance and maintainability are:

- Trapezoidal spillway
- Arched culvert
- Retaining wall

Alternative 2

The structural components with minimal material use are:

- Rectangular sharp-crested spillway
- Rectangular culvert
- Retaining wall

J Constructability steps

Below the 18 steps to construct the ZAC and the bordering up- and downstream channel are given, in support of Section 7.1. Changes per step are given in the figure descriptions. On the left the plan view of the project area is developed. On the right the cross-section as indicated in the plan view is shown. Step 7 to 11 are changes to the culvert-spillway-structure only, therefore they show just the cross-sections. The cross-sections are not scaled, as the width of the ZAC is $300\ m$, which would make the structure development too unclear.

Figure J.1: The red line indicates the route from the main road towards the project location in the north. The almost straight part is an already existing paved road. The last corner is a dirt road of approximately $150\ m$. The first step is to make the road accessible for all equipment and create a new road towards the ramps to enter the ZAC reservoirs as shown in step 2.

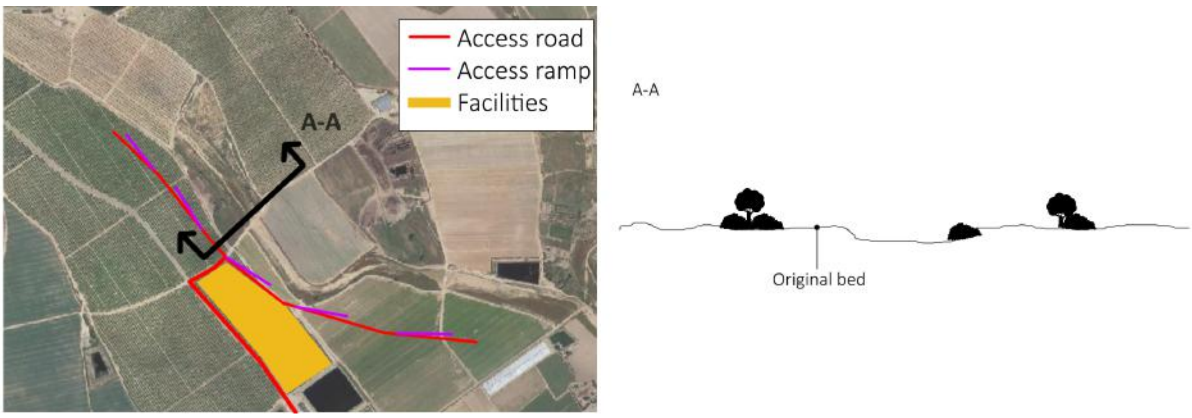


Figure J.2: 5 access ramps are created in total, one for each reservoir. These ramps allow access to the existing soil by excavating equipment. Simultaneously, it is required to set up all the facilities and allocate space for materials at the construction site, such as excavators, construction trailer for construction workers and engineers, water and electricity supply.

Figure J.3: The next step is cutting trees, removing bushes and leveling the bottom to create a nice working area to build the structure in the existing riverbed. Also the existing road in the most downstream reservoir is removed.

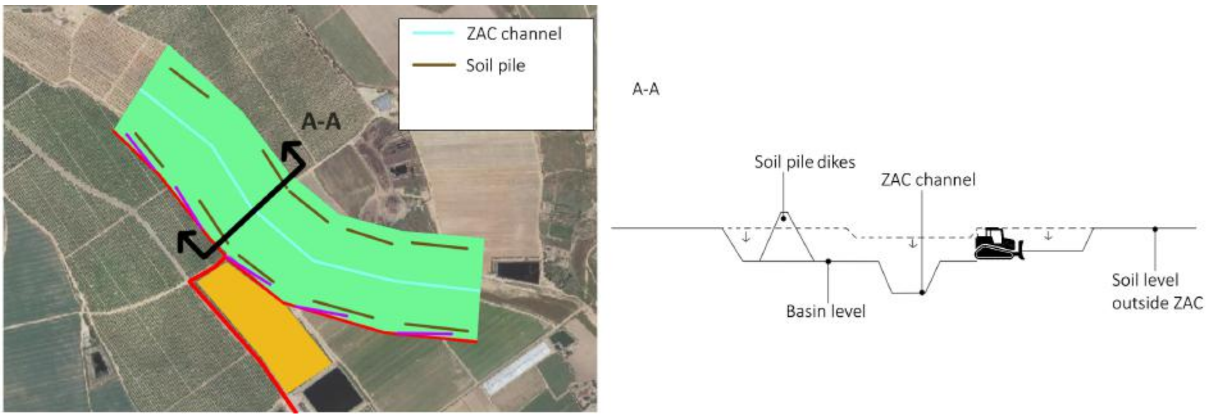


Figure J.4: The soil is excavated and by excavators and displaced by bulldozers. The excavation of the ZAC happens per layer of 50cm, with a slightly deeper ZAC. The ZAC channel is not covered by stones until the final stage of the project. The ZAC channel is excavated wider at the location of the spillways to be able to place the foundation slab (see the wider cross-section B-B in Figure J.6. The excavated soil that needs to be used for the construction of dikes in piles is placed at the sides of the bottom level of the ZAC. This maximizes the working space.

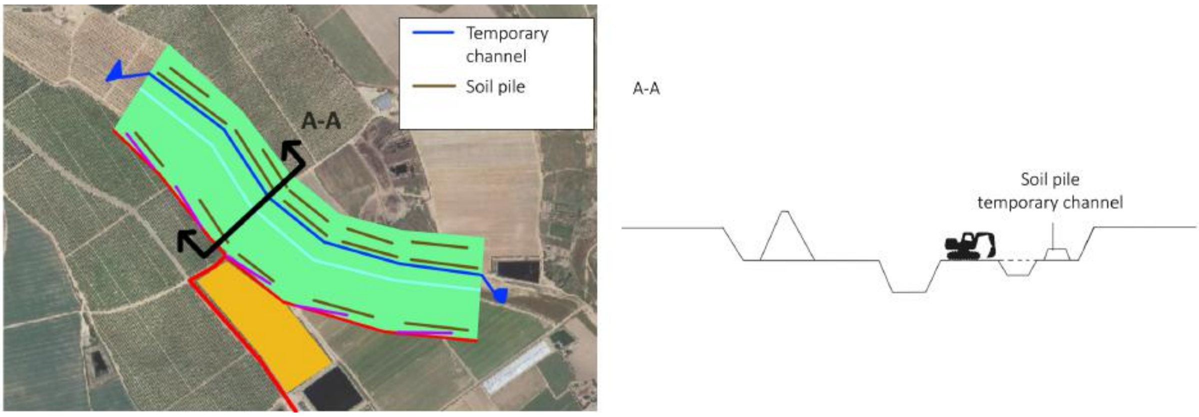


Figure J.5: Excavate a temporary channel at the side which collects the water during the construction of the culvert-spillways in the middle of the ZAC. This temporary channel is placed next to the soil pile from the previous step. The soil from the temporary channel is placed in piles next to the temporary channel. The right soil pile for the side dikes is not shown in the cross-section, as the cross-section is not scaled horizontally.

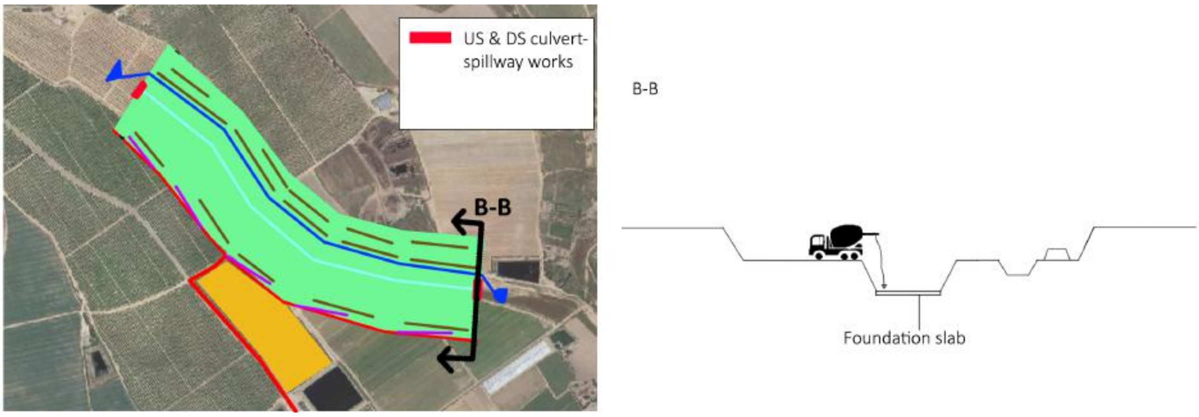


Figure J.6: Foundation works at the up- and downstream end start. Formwork is placed, followed by placing reinforcement and casting concrete for the foundation floor. In the picture the reinforcement that is sticking out of the concrete slab to attach to the culvert and the dike walls is not shown.

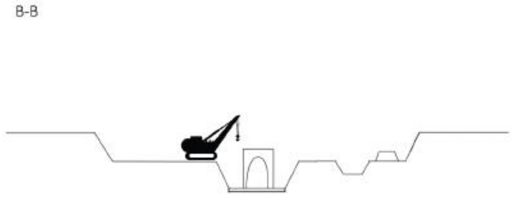


Figure J.7: The prefab culvert boxes up- and downstream are placed by cranes.

Figure J.8: The culvert boxes are connected with a grout layer.

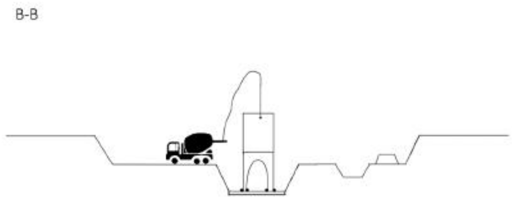


Figure J.9: The formwork and reinforcement of the spillway are placed and the spillway is cast with concrete.

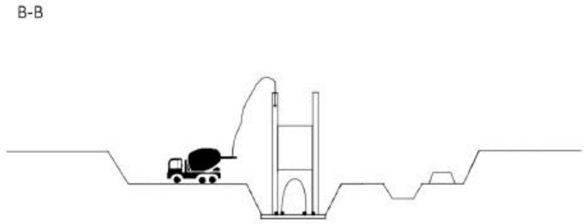


Figure J.10: The formwork and reinforcement for the dike walls are placed and filled by cast concrete. Also, the spillway is connected to the dike walls and the culvert with a grout layer.

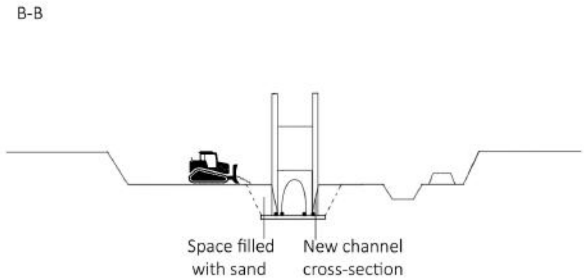


Figure J.11: The spaces between the dike walls and the channel basin level are filled with excavated soil from the area. Finally, the ZAC in- and outflow structures are ready.

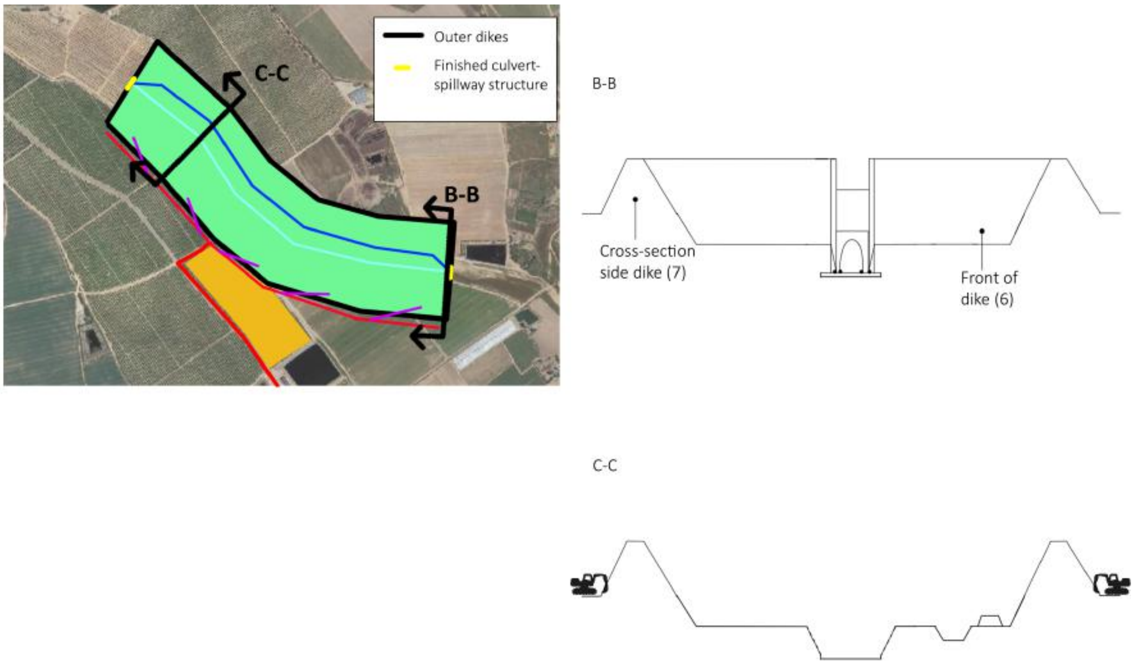


Figure J.12: Use local soil to build up the side dikes, the start and end dike. This is done using excavators and bulldozers. Simultaneously, the up- and downstream structures are connected to the temporary channel.

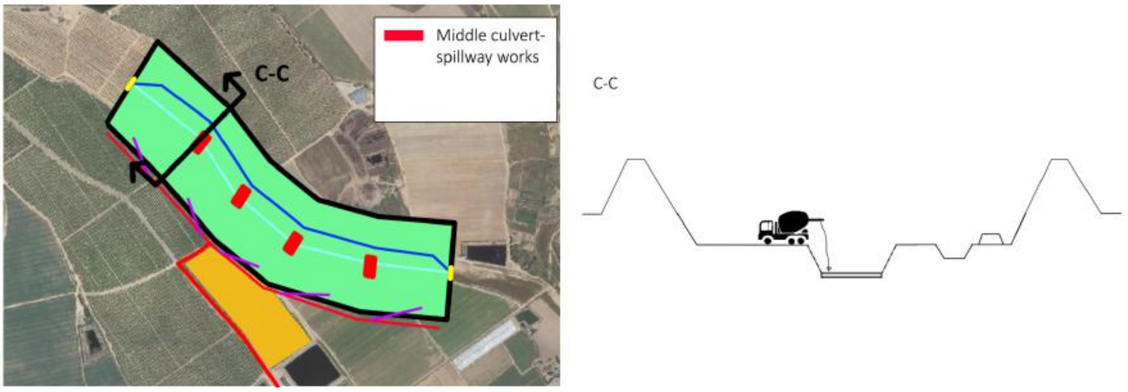


Figure J.13: Foundation works at middle dikes are started: formwork and reinforcement are placed. Concrete is cast for the foundation slab.

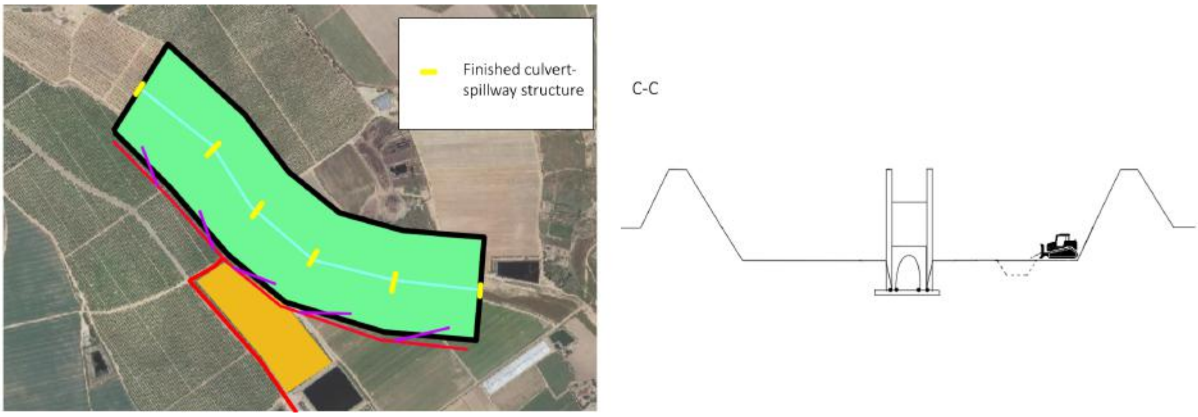


Figure J.14: Repeat step 7 till 11 for the middle structures. After the completion of the middle structures, the temporary channel is closed.

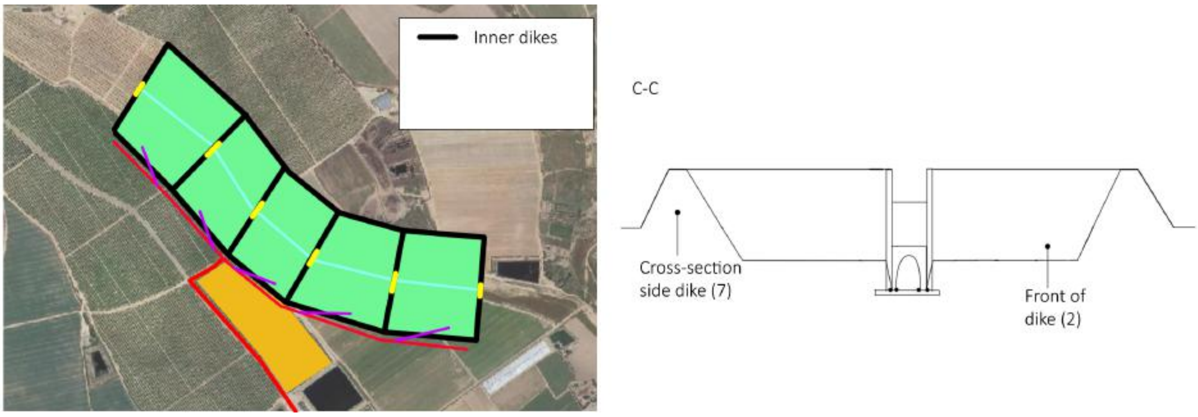


Figure J.15: The middle dikes are constructed and attached to the middle structures. This is done by excavators and bulldozers. The ZAC reservoirs have now been finished.

Figure J.16: The up- and downstream channel are excavated and levelled.

Figure J.17: Stones are placed in the up- and downstream channel by cranes.

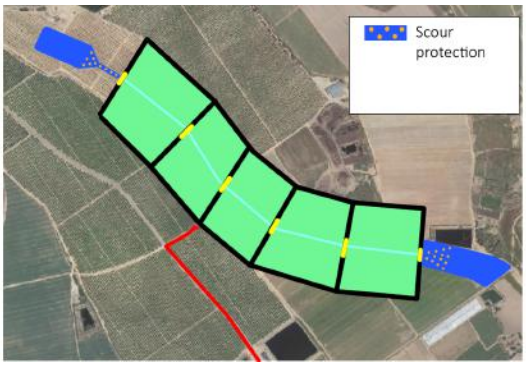


Figure J.18: Fences are placed around the ZAC and all facilities are removed.

K Scour protection

This Appendix gives the calculation for the scour protection in support of Section 7.8

K.1 Armour size

Normative velocity
For constructablilty reasons only one filter is designed and used for the entire upstream channel, main channel and downstream scour protection. From the HEC-RAS model, it followed that the maximum velocity over the entire area is equal to $4.7\ m/s$.

Shields or Isbash
Whether Shield or Isbash is used mostly depends on the ratio between the water depth and the diameter of the armour layer. Shield is applicable when this ratio is equal or larger than 5. When the maximum outflow velocity of $4.7\ m/s$ occurs, the outflow water depth is roughly equal to $2.9\ m$. Assuming a first estimate of the armour diameter of $0.5\ m$, the ratio between the water depth and armour diameter is then equal to 5.8. So, Shields is preferred over Isbash.

Armour layer diameter
For calculation of the armour layer diameter, the following adaptation of the Shields formula is used:

$$d_{n50} = \frac{K_v^2 \overline{u}_c^2}{K_s \psi_c \Delta C^2} \quad with \quad C = 18 \log \frac{12h}{2d_{n50}} \tag{K.0}$$

Where:

- $\overline{u}_c^2$ is the depth averaged critical velocity equal to $4.7\ m/s$.
- K_v is equal to 1 for an abrupt outflow (Ariens, 1993; Van Breugel & Ten Hove, 1995).
- Due to the small slope of the ZAC of 0.0063, the effect of a sloping bed has a negligible influence on the stability of the armour layer. Therefore K_s is also equal to 1.
- ψ_c is the shields parameter and equal to 0.04.
- Δ is the relative density of the rock. This is equal to 1.65.

Filling out this formula with the given parameters results in a d_{n50} of 37 cm. The first larger rock class that fulfills this requirement is LM_A 60 - 300 kg with a d_{n50} of 38 cm.

K.2 Geometrically closed filter design

For the filter layer to be geometrically closed, it must fulfill the three filter rules:

$$1)\ \frac{d_{15,F}}{d_{85,B}} \leq 5 \qquad 2)\ \frac{d_{60}}{d_{10}} \leq 10 \qquad 3)\ \frac{d_{15,F}}{d_{15,B}} \geq 5$$

When choosing a standard grading, it is assumed that the layer always fulfills filter rule 2. The filter layer is optimized to have as few layers as possible. Eurocode EN 13383-1 2013 was used for the properties of the available rock classes (EN 13383-1, 2013). The thickness of the armour layer is equal to $2 * d_{n50} = 80\ cm$. The first filter layer will be of the standard $CP\ 45/125\ mm$ course grading class. This fulfills the filter rules between the armour layer and the first filter layer. In order to fulfill the filter rules between $CP\ 45/125\ mm$ and the base layer with a d_{85} of $300\ \mu m$, a wide graded small gravel layer with a range of 1.5 - $9\ mm$ is used. The thickness of both the filter layers are equal to the minimum required with of $20\ cm$. This is possible because the filter layer is constructed in a dry river during the dry season. This ensures minimum placement errors, which will save construction costs. Figure K.1 below shows an overview of the filter layer.

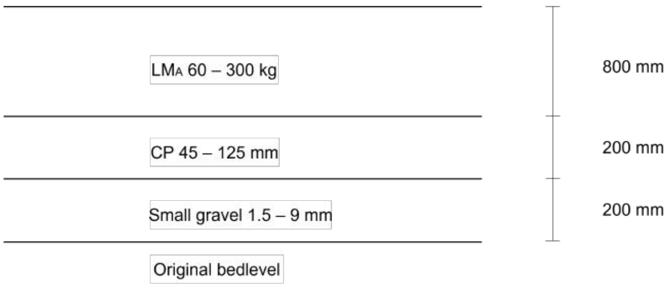


Figure K.1: Overview of the geometrically closed filter used as bottom protection in the ZAC channel, downstream of the ZAC and in the upstream channel.

The total thickness of the filter is $1.2\ m$. This means that the channels have to be excavated $1.2\ m$ deeper in order to maintain the design cross-section. A geotextile is said to be not preferable in this situation. This is because It will probably only replace the small gravel layer. The $CP\ 45/125\ mm$ layer is still necessary because the larger LM_A 60 - 300 kg rock class can damage the geotextile when placed directly on top of it.

K.3 Scour protection length

Downstream of the ZAC, the scour protection will have to extent far enough from the structure to minimize the chance of the structure falling into the scour hole. The water depth downstream of the culvert is important to determine the formula used to calculate the depth of the scour hole in case of no protection. Breusers & Raudkivi (1991) state that if the downstream water depth is larger than the height of the outflow opening, the structure can be assumed to function as a jet. If the downstream water depth is smaller than the outflow opening, the structure functions as a culvert. This is assumed to be useful for a first approach. From the HEC-RAS model, it follows that the downstream water depth is equal to around $1\ m$. Therefore, the following formula for the scour hole depth is used.

$$\frac{h_{se}}{2B} = 0.65\left(\frac{u_0}{u_{*c}}\right)^{0.33} \tag{K.0}$$

Where:

- B is the with of the outflow opening equal to $5\,m$.
- u_0 is the outflow velocity. This is equal to $4.7\,m/s$.
- u_{*c} is the critical velocity. This is calculated using Shields. For a d_{n50} of $150\,\mu m$, the critical velocity is equal to $1.34 * 10^{-4}\,m/s$.

From this formula, it follows that the depth of the scour hole is around $205\,m$ deep. This large depth can be explained by the combination of a large outflow velocity and a small diameter of the bed material, though $205\,m$ might be a little conservative. The also follows from the assumption of clear water scour from upstream. This is probably not the case as it is expected that such rainfall events will bring sediment from upstream. Breusers & Raudkivi (1991) also assume constant flow. In the case of this project, normal conditions represent a dry river. From the research of Breusers & Raudkivi (1991) it follows that the length of the scour hole is around 5 times the depth of the scour hole. This would result in a scour protection length of roughly $1\,km$.

L Dike slip-planes

The results of the stability calculations of the dike are shown below in the figures. This is in support of Section 7.7.5. Each critical cross-section of dike 1, 2, 3, 4, 5, 6, 7 & 8 is checked by the Bishop method and Uplift Van. The critical cross-section are located where the elevation difference of the ground level and the crest level is the largest.

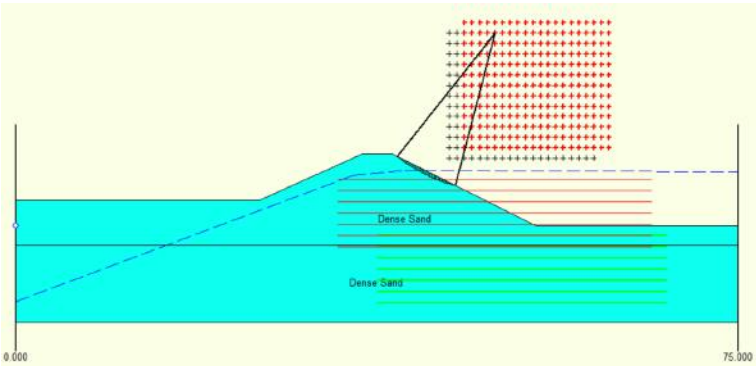


Figure L.1: Dike 1: crest height = 7.0m; maximum water level = 5.4m. Slip plane via the Bishop method. Minimum factor of safety = 1.12

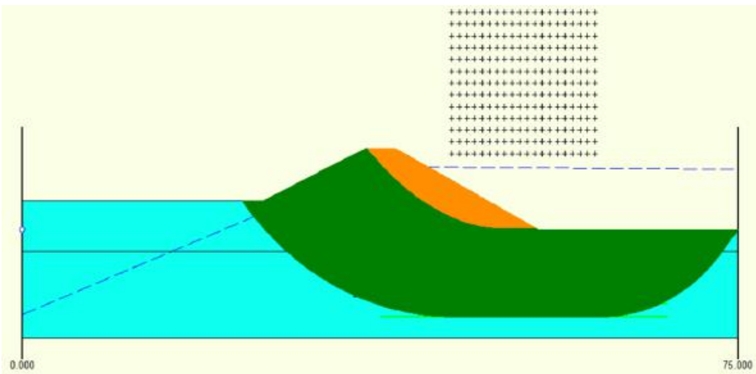


Figure L.2: Dike 1: crest height = 7.0m; maximum water level = 5.4m. Safety overview: orange represent safety factor of 1.00 to 1.15, green represent safety factor of 1.15 or greater. Minimum factor of safety = 1.12

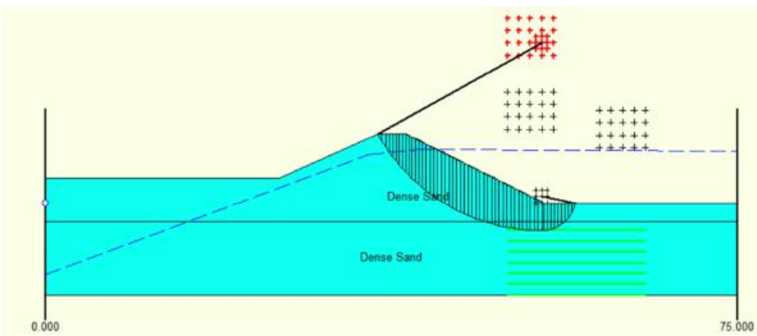


Figure L.3: Dike 1: crest height = 7.0m; maximum water level = 5.4m. Slip plane via the Uplift Van method. Minimum factor of safety = 1.79

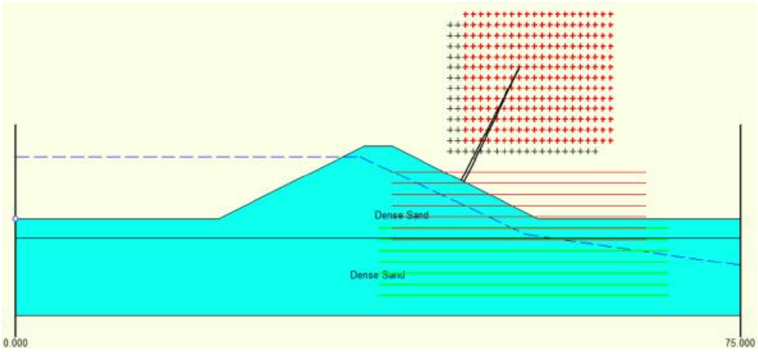


Figure L.4: Dike 2,3,4: crest height = 7.5m; maximum water level = 7.2m. Slip plane via the Bishop method. Minimum factor of safety = 1.12.

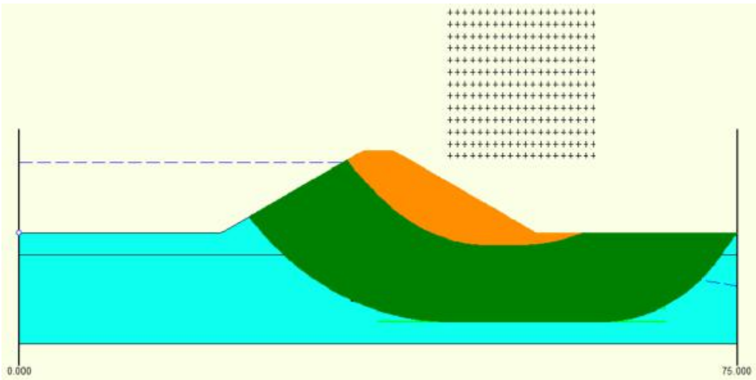


Figure L.5: Dike 2,3,4: crest height = 7.5m; maximum water level = 7.2m. Safety overview: orange represent safety factor of 1.00 to 1.15, green represent safety factor of 1.15 or greater. Minimum factor of safety = 1.12.

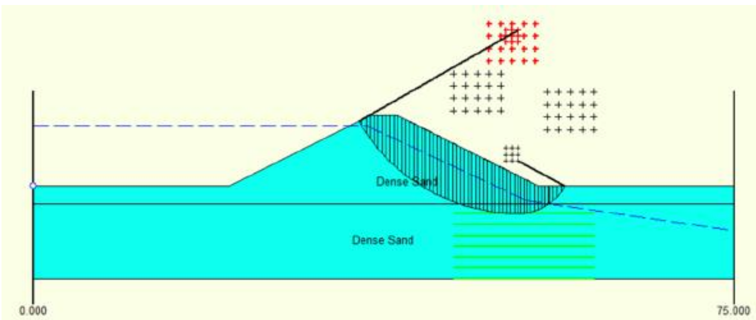


Figure L.6: Dike 2,3,4: crest height = 7.5m; maximum water level = 7.2m. Slip plane via the Uplift Van method. Minimum factor of safety = 1.79

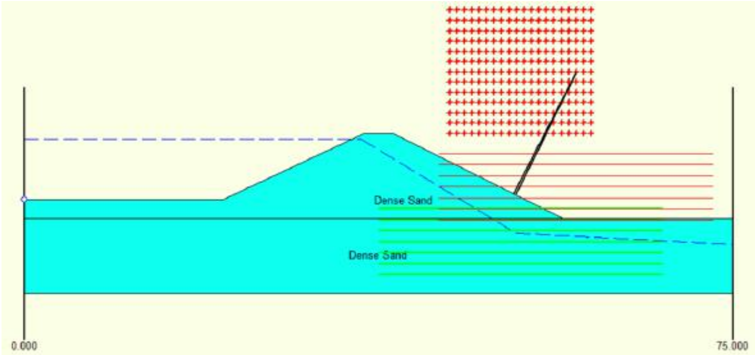


Figure L.7: Dike 6: crest height = 7.0m; maximum water level = 6.8m. Slip plane via the Bishop method. Minimum factor of safety = 1.15.

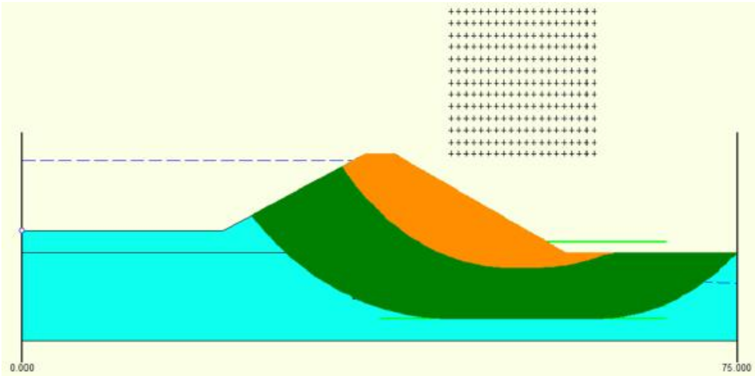


Figure L.8: Dike 6: crest height = 7.0m; maximum water level = 6.8m. Safety overview: orange represent safety factor of 1.00 to 1.15, green represent safety factor of 1.15 or greater. Minimum factor of safety = 1.15.

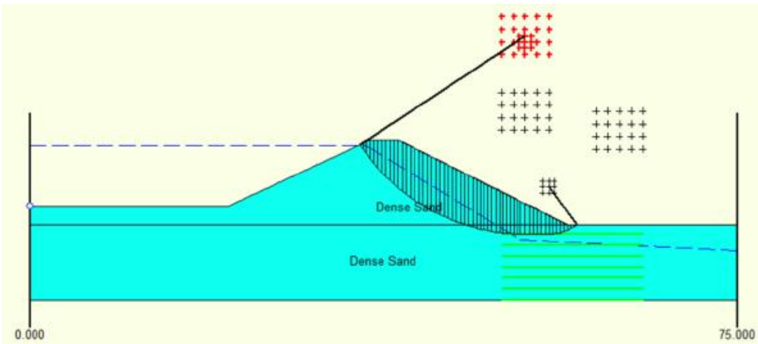


Figure L.9: Dike 6: crest height = 7.0m; maximum water level = 6.8m. Slip plane via the Uplift Van method. Minimum factor of safety = 1.31.

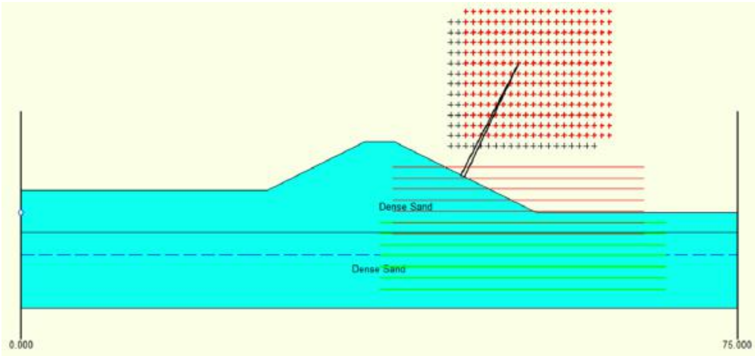


Figure L.10: Dike 7: crest height at critical cross-section = 7.65m; maximum water level = 6.8m. Slip plane via the Bishop method. Minimum factor of safety = 1.12

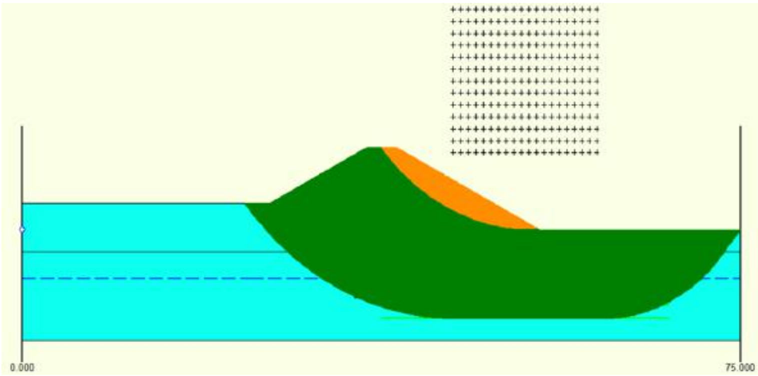


Figure L.11: Dike 7: crest height at critical cross-section = 7.65m; maximum water level = 6.8m. Safety overview: orange represent safety factor of 1.00 to 1.15, green represent safety factor of 1.15 or greater. Minimum factor of safety = 1.12.

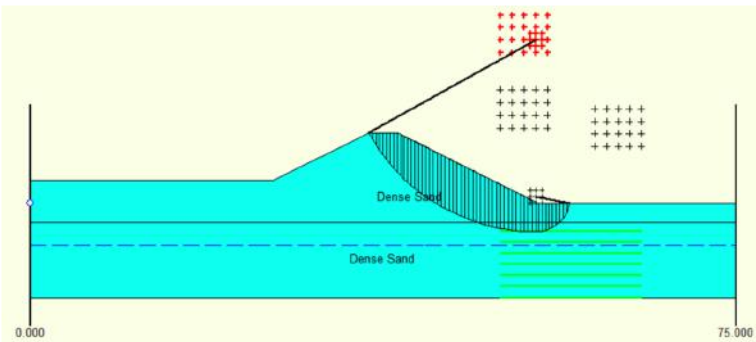


Figure L.12: Dike 7: crest height at critical cross-section = 7.65m; maximum water level = 6.8m. Slip plane via the Uplift Van method. Minimum factor of safety = 1.77.

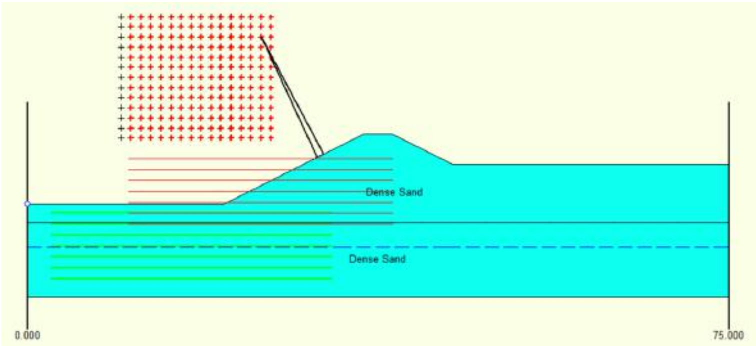


Figure L.13: Dike 8: crest height at critical cross-section = 7.5m; maximum water level = 6.8m. Slip plane via the Bishop method. Minimum factor of safety = 1.16.

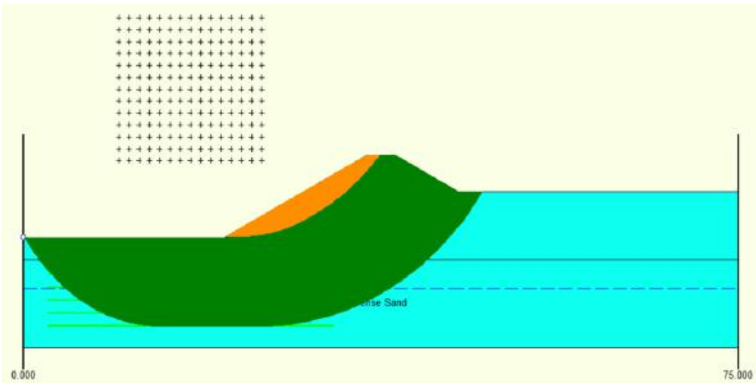


Figure L.14: Dike 8: crest height at critical cross-section = 7.5m; maximum water level = 6.8m. Safety overview: orange represent safety factor of 1.00 to 1.15, green represent safety factor of 1.15 or greater. Minimum factor of safety = 1.16.

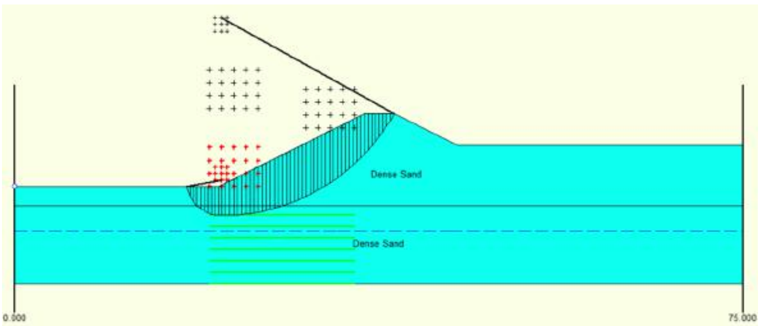


Figure L.15: Dike 8: crest height at critical cross-section = 7.5m; maximum water level = 6.8m. Slip plane via the Uplift Van method. Minimum factor of safety = 1.93.

M Structural Design of the ZAC structure

In this appendix the full elaborations for the sections of chapter 7 of the ZAC structure are given. This includes the concrete element width, the concrete cover, the load quantification, the determination of conceptual reinforcement, and the strength verification in Diana.

M.1 Conceptual design

Table M.1: Design parameters for the chosen concrete class and reinforcement class

Consequence class CC2	
Permanent load safety factor	1.2
Variable load safety factor	1.5
Concrete class C35/35	
Material factor γ_c	1.5
f_{ck}	35 N/mm^2
f_{cd}	23.33 N/mm^2
f_{cm}	43 N/mm^2
f_{ctm}	3.2 N/mm^2
$f_{ctk,0.05}$	2.2 N/mm^2
$f_{ctk,0.95}$	4.2 N/mm^2
E_{cm}	34000 N/mm^2
Steel class B500B	
Material factor γ_s	1.15
f_{yk}	500 N/mm^2
f_{yd}	435 N/mm^2
e_{uk}	2.75 N/mm^2
E_s	$2.0 \cdot 10^5$ N/mm^2

M.2 Concrete element width

For an approximation of the initial concrete element widths, rules of thumb for designing a structure are applied.

Spillway
Based on *Vuistregels voor het ontwerpen van een draagconstructie* (2013), the thickness of an in-situ cast floor element with line supports and a span longer than 7 m, should be 1/32 of the longest span. For simplicity, the slabs of the spillway are assumed to be equivalent to such a description.

For the determination of the concrete width, only the spillway element with the largest span is considered. This is the sloped element since the bottom floor element of the spillway is fully supported by the culvert. The sloped element has a length of 9.20 m. The initial approximation of the concrete thickness is L/32 = 287.59 mm. Thus, for a preliminary design, all the spillway elements are assumed to have a concrete thickness of 300 mm.

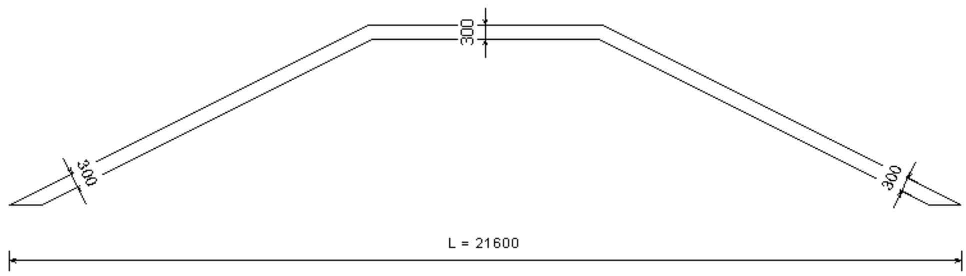


Figure M.1: Sketch of spillway with assumed thickness

Culvert
The determination of the thickness of the culvert element works differently. According to the Eurocode NEN-EN-1992-1-1 (3), the culvert walls are considered beams that are related to the ratio between the dimensions. The width of the beam should be $\frac{1}{2}$ to $\frac{1}{3}$ of the height of the beam. The height of the beam is 2010 mm. This means that the beam width should lie between, 1005-670 mm. For further calculations, the width of the beams is chosen to be 1000 mm, since it matches the geometry of the space for the culvert in the ZAC structure perfectly. The thickness of the culvert roof should be $\frac{1}{22}$ of the span. The span is equal to 7000 mm. Hence, the culvert roof should have a minimum thickness of 318 mm. For a preliminary design of the culvert, a thickness of 350 mm is chosen.

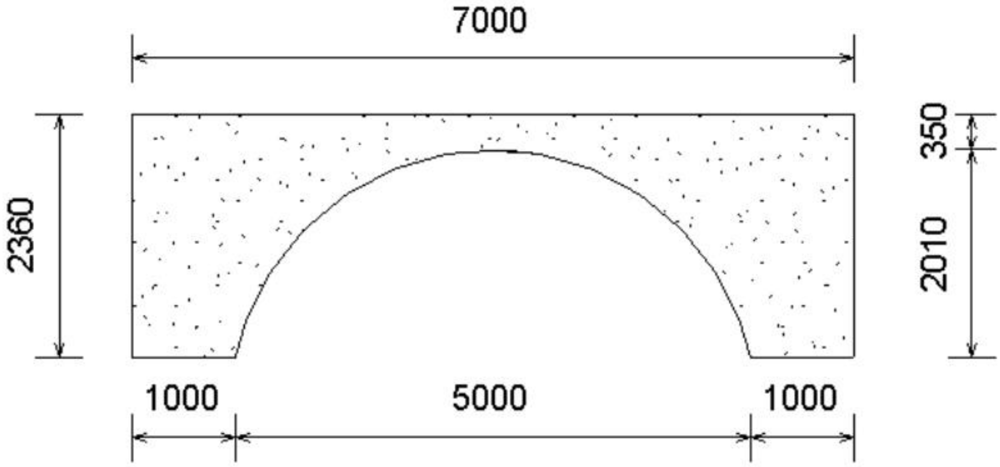


Figure M.2: Sketch of culvert with assumed thickness

Retaining wall
Based on *Vuistregels voor het ontwerpen van een draagconstructie* (2013), the thickness of an in-situ cast wall element with a height up to 4 m, should have a minimum thickness of 150 mm. Since this is a retaining wall with a height of 9 m, a concrete thickness of 400 mm is assumed.

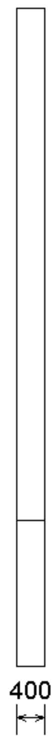


Figure M.3: Sketch of retaining wall with assumed thickness

M.3 Concrete Cover

The minimum required concrete cover is the maximum of either the concrete cover based on the durability of the reinforcement, the concrete cover based on bonding or 10 mm.

Concrete cover, due to the durability of the reinforcement ($c_{min,dur}$)
The minimum required concrete cover, based on the durability of the reinforcement, is determined by the structural class and exposure class per structural component.

Exposure class
The exposure classes are based on table 4.1 of the NEN-EN 1992-1-1+C2:2011. The ZAC structure is affected by the fresh water in the channel. This leads the ZAC structure to be sensitive to corrosion, which is induced by the carbonation of the concrete. This sensitivity to corrosion, due to carbonation, leads to an XC exposure class.

The culvert structure fluctuates between wet and dry, this lead to an exposure class of XC4. Based on the design lifetime, it is determined that the spillways are operational for rainfall events that occur once every 24 years, see Appendix B for the determination of the rainfall event frequency. Thus, it is assumed that the spillway lies usually dry, which leads to an exposure class of XC3. As with the spillway, the retaining wall is in contact with the flow of rainfall events once every 24 years. This also leads to an exposure class of XC3. In Table M.2 the exposure classes are indicated.

In reality, not all the elements of each structural component experience the same environment. For example, the bottom slab of the spillway is only in contact with the culvert and not the flow, this would lead to a lower exposure class. However, for simplicity, the governing exposure class for an element of the structural component is applied to all the elements of that component. Thus, the above-determined exposure classes are applied.

Structural class
The structural classes are based on table 4.3 of the NEN-EN 1992-1-1+C2:2011. The standard structural class is S4 for a design lifetime of 50 years and a low concrete quality. The ZAC structure differs from this, regarding the concrete quality.

The concrete quality of C35/45 reduces the structural class by one. Since the construction environment is unknown, it is assumed that the position of the reinforcement can be affected during construction and that the quality control of the concrete factory is not optimal. For all the structural components, the structural class is reduced by one. In Table M.2 the structural classes are indicated.

Based on the determined exposure and structural classes, the concrete cover per structural component is determined. The values of the concrete covers are based on table 4.4 of the NEN-EN 1992-1-1+C2:2011.

Table M.2: Design parameters for chosen concrete class and reinforcement class

Structural component	Exposure class	Structural class	$C_{min,dur}$ [mm]
Retaining wall	XC4	S(4-1)=S(3)	35
Spillway	XC3	S(4-1)=S(3)	30
Culvert	XC3	S(4-1)=S(3)	30

Concrete cover due to bonding requirements ($c_{min,b}$)
The required minimum concrete cover for the bonding requirements between the concrete and reinforcement, depends on the grain distribution of the surrounding soil. It is assumed that the surrounding soil has no grains larger than 32 mm, since it mostly consists out of sand. Thus, the minimum concrete cover is equal to the diameter of the applied reinforcement plus an additional 5 mm (van Baars & Voorendt, & et al., 2022)

The assumed reinforcement diameter is 32 mm.

$$c_{min,b} > \varnothing_{bar} + 5mm = 32 + 5 = 37mm$$

Minimum concrete cover (c_{min})
As previously indicated, the minimum concrete cover is the maximum of either the concrete cover based on the durability of the reinforcement, the concrete cover based on bonding or 10 mm. Thus,

$$c_{min} = max(c_{min,dur},\ c_{min,b},\ 10mm)$$

Spillway + Culvert

$$c_{min} = max(30,\ 37,\ 10) = 37mm$$

Retaining wall

$$c_{min} = max(35,\ 37,\ 10) = 37mm$$

Construction tolerances, c_{dev}
Based on the hydraulic structures manual (van Baars & Voorendt, & et al., 2022), c_{dev} can be assumed to be 5 mm.

Nominal concrete cover, c_{nom}
The required concrete thickness is: $c_{nom} = c_{min} + c_{dev}$

M.4 Soil pressure on the retaining walls

Vertical soil pressure

The soil pressure is split into water and effective soil stresses. However, since the groundwater is neglected in this project, the soil pressure is equal to the effective soil stresses. The reference level (GL) is taken as the bottom of the channel.

At depth GL+9.0m: $\sigma_{v9} = \sigma'_{v,9} = 0\ kN/m^2$

At depth GL+8.5 m;

The distance between CL+9 m and GL+8.5 m is 0.5 meters. The soil pressure is equal to the distance multiplied by the specific weight of the soil. This method is used for all the upcoming depths.

$$\sigma_{v,8.5} = \sigma'_{v,8.5} = \sigma'_{v,9.0} + d_{9-8.5}\gamma = 0 + 0.5 \cdot 19 = 9.5\ kN/m^2$$

At depth GL+6.5 m:

$$\sigma_{v,6.5} = \sigma'_{v,6.5} = \sigma'_{v,8.5} + d_{8.5-6.5}\gamma = 9.5 + 2.0 \cdot 19 = 47.5\ kN/m^2$$

At depth GL+2.36m:

$$\sigma_{v,2.36} = \sigma'_{v,2.36} = \sigma'_{v,6.5} + d_{6.5-2.36}\gamma = 47.5 + 4.14 \cdot 19 = 126.16\ kN/m^2$$

At depth GL+0m:

$$\sigma_{v,0} = \sigma'_{v,0} = \sigma'_{v,2.63} + d_{2.36-0}\gamma = 126.16 + 2.36 \cdot 19 = 171\ kN/m^2$$

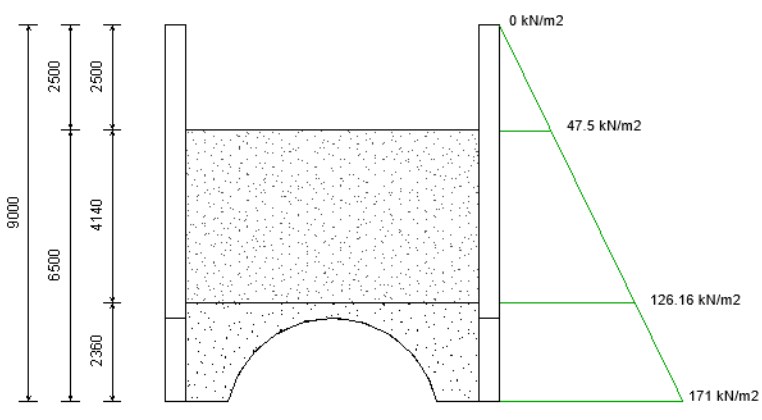


Figure M.4: Sketch of vertical soil stresses on structure

Horizontal soil pressure
For determining the horizontal soil stress acting on the structures, the water pressure and effective soil pressure should be considered separately.

$$\sigma'_{soil} = P + \sigma'_h$$

The horizontal water pressure at a certain depth is equal to the vertical water pressure (Pascals law). The relation between horizontal and vertical effective pressure is usually assumed to be constant.

$$\sigma'_h = K \cdot \sigma'_v$$

For determining the effective horizontal soil pressures, Rankine's theory is applied (Chapter 24.2.2,van Baars & Voorendt, & et al., 2022). The soil under active stress develops less horizontal soil pressure than the soil under passive stress. In this situation, the right-hand side of the retaining wall is the active side and on the left-hand side is the spillway-culvert structure. The lower limit of the active side is indicated below.

$$\sigma'_{h,min} = K_a \cdot \sigma'_v - 2c\sqrt{K_a} \quad with\ K_a = \frac{1 - sin\phi'}{1 + sin\phi'}\ coefficient\ of\ active\ soil\ pressure[-]$$

The effect of the friction at the soil-wall interface is not included in the calculation.

Table 31-6 from the Hydraulic structures gives indicatives values for soil properties. For the assumed uniform soil distribution, an internal friction angle of 30 and cohesion of zero are assumed.

$$K_a = \frac{1 - sin\phi'}{1 + sin\phi'} = \frac{1 - sin30}{1 + sin30} = 0.333$$

The effective horizontal soil pressure is calculated for the specific depths.

At depth GL+9.0m: $\sigma'_h = K \cdot \sigma'_v = 0.33 \cdot 0 = 0\ kN/m^2$

At depth GL+8.5m: $\sigma'_h = 9.6kN/m^2 \cdot 0.333 = 3.17kN/m^2$

At depth GL+6.5 m: $\sigma'_h = 47.5kN/m^2 \cdot 0.33 = 15.83\ kN/m^2$

At depth GL+2.36m: $\sigma'_h = 126.16kN/m^2 \cdot 0.33 = 42.05\ kN/m^2$

At depth GL+0m: $\sigma'_h = 171kN/m^2 \cdot 0.33 = 57\ kN/m^2$

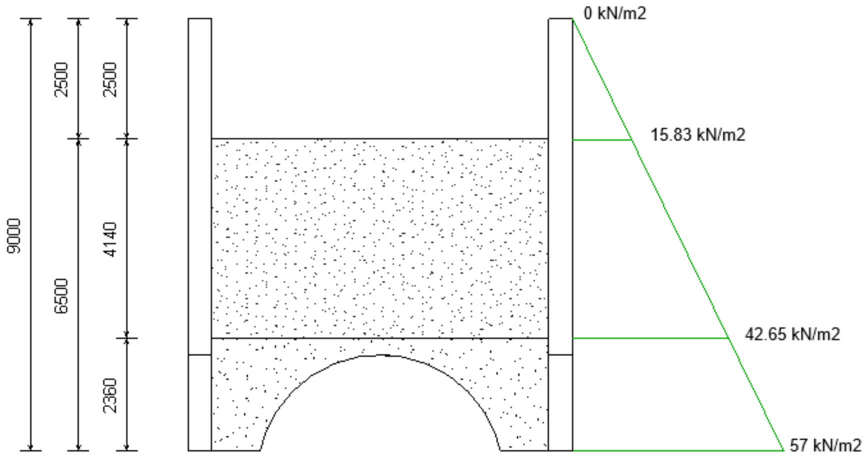


Figure M.5: Sketch of horizontal soil stresses on structure

Dike 2

The surrounding dikes are supported by the retaining walls. Thus, the retaining walls are loaded by soil pressure. The total height of the dike is 9.5 meters. The same method as for dike 5 is applied.

Vertical soil pressure

The soil pressure is split into water and effective soil stresses. However, since the groundwater is neglected in this project, the soil pressure is equal to the effective soil stresses. The reference level (GL) is taken as the bottom of the channel.

At depth GL+9.5m: $\sigma_{v,9.5} = \sigma'_{v,9.5} = 0kN/m^2$

At depth GL+9.1 m: $\sigma_{v,9.0} = \sigma'_{v,9.0} = \sigma'_{v,9.5} + d_{9.5-9.0} \cdot \gamma = 0 + 0.4 \cdot 19kNm^2 = 7.6kN/m^2$

At depth GL+6.0 m: $\sigma_{v,6.0} = \sigma'_{v,6.0} = \sigma'_{v,9.1} + d_{9.1-6.0} \cdot \gamma = 7.6kN/m^2 + 3.1 \cdot 19kNm^2 = 66.5kN/m^2$

At depth GL+2.36m:$\sigma_{v,2.36} = \sigma'_{v,2.36} = \sigma'_{v,6.0} + d_{6.0-2.36} \cdot \gamma = 66.5kN/m^2 + 3.64 \cdot 19kNm^2 = 135.66kN/m^2$

At depth GL+0m:$\sigma_{v,0} = \sigma'_{v,0} = \sigma'_{v,2.36} + d_{2.36-0} \cdot \gamma = 135.66kN/m^2 + 2.36 \cdot 19kNm^2 = 180.50kN/m^2$

Horizontal soil pressure

The effective horizontal soil pressure is calculated for the specific depths.

At depth GL+9.5m: $\sigma'_h = K \cdot \sigma'_v = 0.33 \cdot 0 = 0 \; kN/m^2$

At depth GL+9.1m: $\sigma'_h = 7.6kN/m^2 \cdot 0.333 = \; 2.53kN/m^2$

At depth GL+6.0 m: $\sigma'_h = 66.5kN/m^2 \cdot 0.33 = 21.95 \; kN/m^2$

At depth GL+2.36m: $\sigma'_h = 135.66kN/m^2 \cdot 0.33 = 44.77 \; kN/m^2$

At depth GL+0m: $\sigma'_h = 180.5 kN/m^2 \cdot 0.33 = 59.57\ kN/m^2$

M.5 Determining reinforcement dike 5

Dike 5, situation 2

Spillway
For determining the reinforcement, only the self-weight and hydrostatic pressure are considered. The loads are expressed per unit width in MatrixFrame.

The bottom of the spillway is the same as the roof of the culvert. Therefore, the spillway is modelled as a 2D frame supported by multiple springs, because in reality it is supported by the culvert beams. A beam on 2 supports loaded by a uniform load is considered to determine the spring stiffnesses. In the centre of the beam lies the governing deflection. This deflection is determined by the forget-me-nots.

$$w = \frac{5}{384}\frac{qL^4}{EI}$$

The uniform load has the resulting force of $q \cdot L$ in the centre of the beam. In order to have zero deflection of the beam, the spring force should be equal to qL. This gives a spring stiffness of:

$$k = \frac{F}{w} = \frac{qL}{w}$$

$$k = \frac{384}{5}\frac{EI}{L^3}$$

Table M.3: Overview of the determined parameters of the bottom spillway/culvert roof element. The value of the bending stiffness does not include the reinforcement.

Input		
Width	b	1000 mm
Height	h	350 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9\ mm^4$
Length	L	21600 mm
Output		
spring stiffness (dike 5)	k	583 kN/m

MatrixFrame model

The springs are evenly distributed with a distance of 0.6 meters.

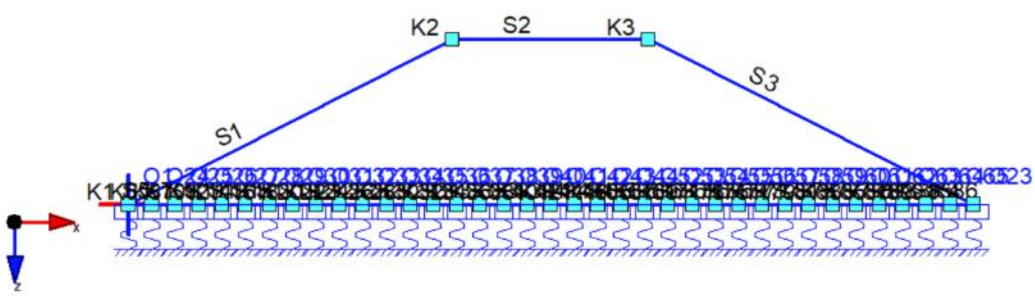


Figure M.6: Matrixframe model of the spillway for dike 5

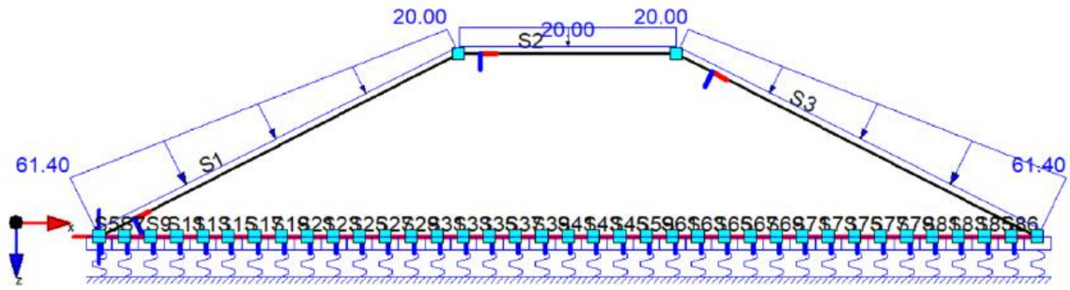


Figure M.7: Matrixframe model of the spillway for dike 5 with the loading of situation 2

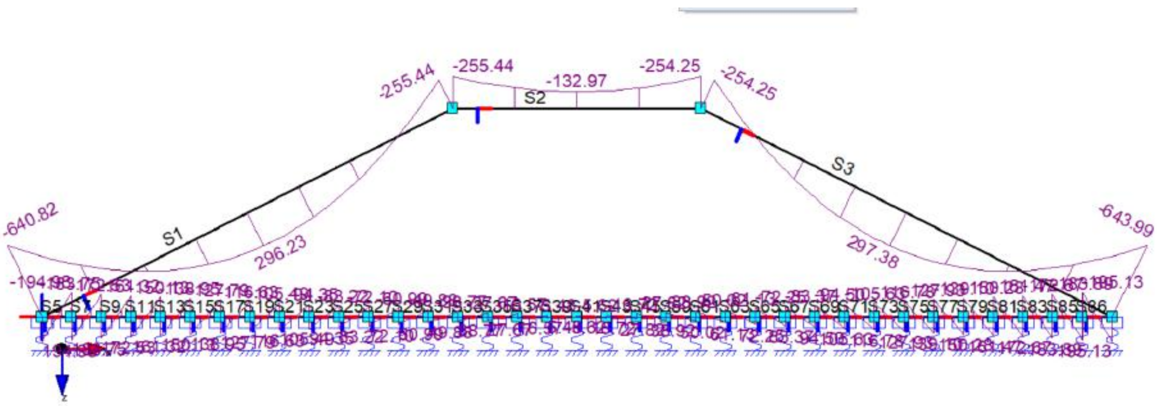


Figure M.8: Moments line of the spillway model for dike 5 with the loading of situation 2

The reinforcement is determined with formula 35.5 of the hydraulic structures Manual (van Baars & Voorendt, & et al., 2022)

$$M_{ed} = M_u = A_s \cdot f_{yd} \cdot (1 - 0.52 \cdot \rho \cdot k$$

$$k = f_{yd}/f_{cd} = 435/23.33 = 18.646$$

$$\rho = \frac{A_s}{bd}$$

Table M.4: Overview of the governing bending moments per spillway element and the corresponding required reinforcement area. The values are per unit width.

Spillway element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm^2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm^2]
Top slab	-255	2097	0	0
Sloped slabs	-644	6163	298	2483
Bottom slab	-195	1330	195	1330

Culvert
The culvert is modelled as a 2D portal frame for the transversal reinforcement. The loads coming from the spillway are taken from the previously constructed MatrixFrame model. The governing characteristic vertical reaction force from the spillway is applied as a uniform load.

Table M.5: Overview of the determined parameters of the culvert elements. The value of the bending stiffness does not include the reinforcement.

Input		
Culvert beams		
width	b	1000 mm
height	h	1000 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9$ mm^4

MatrixFrame model

Governing vertical characteristic reaction force from spillway = 26.11 kN.

Figure M.9: Matrixframe model of the culvert for dike 5

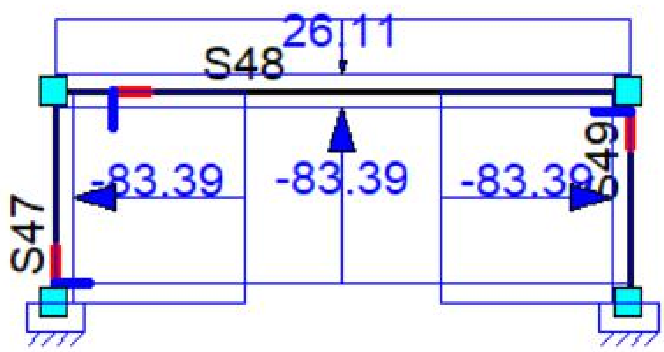


Figure M.10: Matrixframe model of the culvert for dike 5 with the loading of situation 2

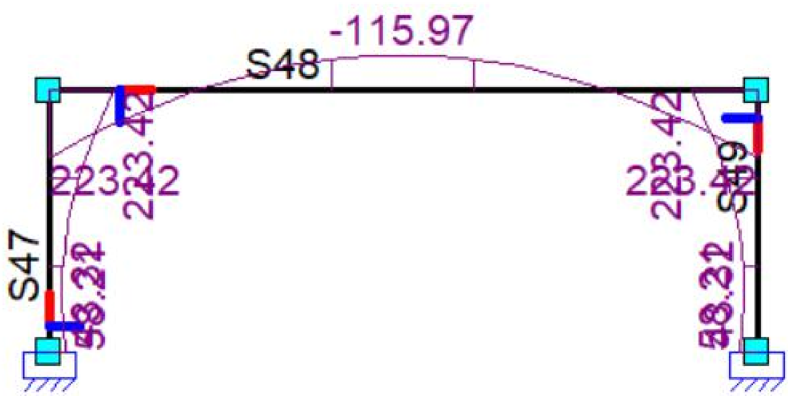


Figure M.11: Moments line of the culvert model for dike 5 with the loading of situation 2

Table M.6: Overview of the governing bending moments per culvert element and the corresponding required reinforcement area. The values are per unit width.

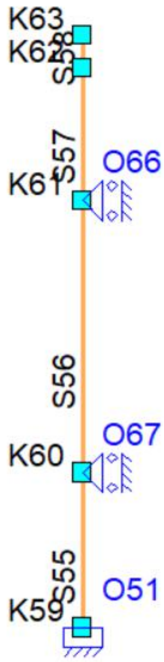
Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm^2]	Sagging moment [kNm]	Required reinforcement area [As,req] [mm^2]
Dike 5 situation 2				
Top slab (trans)	-116	779	224	1537
Beams	0	0	224	518

Retaining wall
The retaining wall is modelled as a 2D beam with rigid support due to the foundation. Also, the retaining wall is supported in the horizontal direction by the spillway, this is modelled as two horizontal supports, present at the top and the bottom of the spillway.

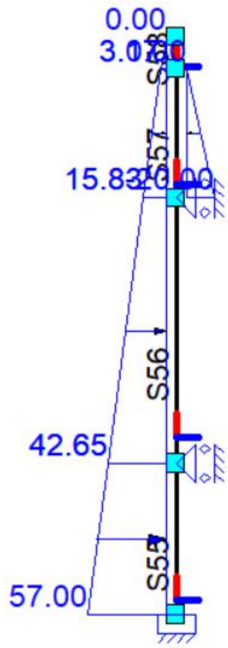
Table M.7: Overview of the determined parameters of the retaining wall. The value of the bending stiffness does not include the reinforcement.

Input		
width	b	1000 mm
height	h	400 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9$ mm^4

MatrixFrame model



(a) Matrixframe model of the retaining wall for dike 5



(b) Matrixframe model of the retaining wall for dike 5 with the loading of situation 2

(c) Moments line of the retaining wall for dike 5 with the loading of situation 2

Table M.8: Overview of the governing bending moments of the retaining wall and the corresponding required reinforcement area. The values are per unit width.

	Hogging moment [kNm]	Required reinforcement area [As,req] [mm^2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm^2] bottom part
Retaining wall	-77	448	55	318

Dike 5, situation 1

The same matrixFrame models of situation 2 are applied for situation 1, only the loading differs.

Spillway

The calculated spring stiffness for situation 2 also applies for situation 1.

MatrixFrame model

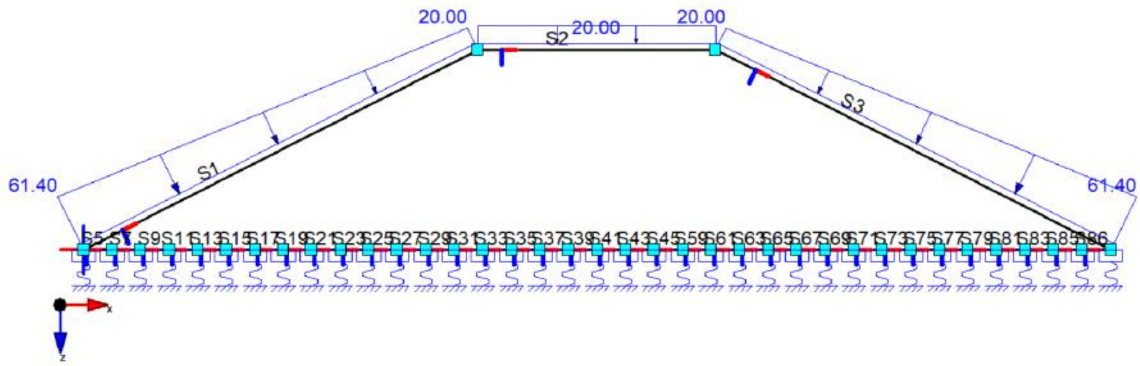


Figure M.13: Matrixframe model of the spillway for dike 5 with the loading of situation 1

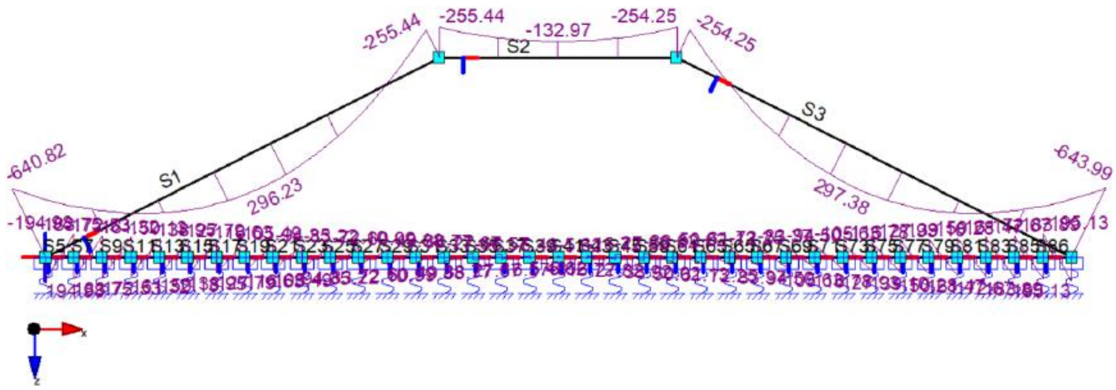


Figure M.14: Moments line of the spillway model for dike 5 with the loading of situation 1

Table M.9: Overview of the governing bending moments per spillway element and the corresponding required reinforcement area. The values are per unit width.

Spillway element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Top slab	-223	1816	0	0
Front Sloped slabs	-634	6036	276	2284
Back sloped slabs	-223	1816	112	884
Bottom slab/roof culvert	-163	1105	163	1105

Culvert

<u>MatrixFrame model</u>

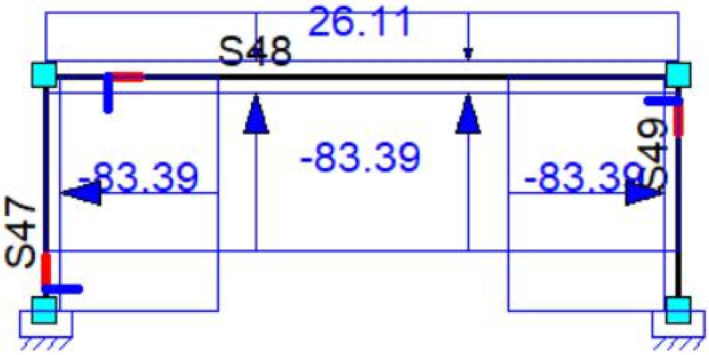


Figure M.15: Matrixframe model of the culvert for dike 5 with the loading of situation 1

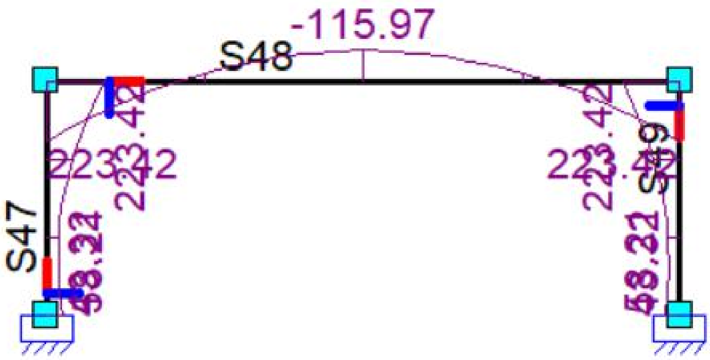


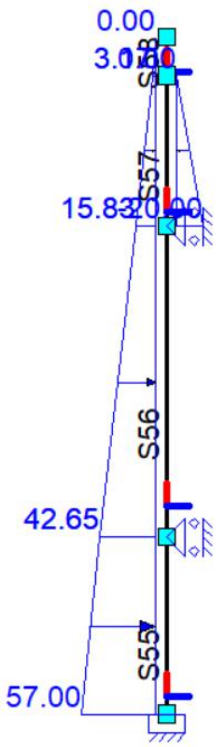
Figure M.16: Moments line of the culvert model for dike 5 with the loading of situation 1

Table M.10: Overview of the governing bending moments per culvert element and the corresponding required reinforcement area. The values are per unit width.

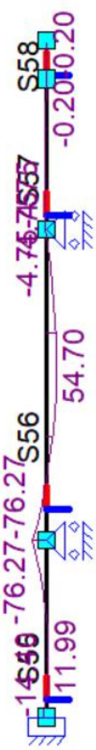
Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Top slab (transversal direction)	-122	820	234	1609
Beams	0	0	234	541

Retaining wall

<u>MatrixFrame model</u>



(a) Matrixframe model of the retaining wall for dike 5 with the loading of situation 1



(b) Moments line of the retaining wall for dike 5 with the loading of situation 1

Table M.11: Overview of the governing bending moments per Retaining wall and the corresponding required reinforcement area. The values are per unit width.

	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Retaining wall	-72	418	50	290

Concept design

The reinforcement net is based on the governing bending moment in the top slab.

$$M_{ed} = 255 \cdot 10^6 kNm$$

$$A_s = 2097mm^2$$

This leads to bar spacing (s) of:

$$n = \frac{A_s}{d} = \frac{2097}{25}2 = 4.27 \approx 5$$

$$s = \frac{b - 2 \cdot c}{n} = 180mm$$

The chosen net is $\phi 25 - 180mm$, which leads to an applied reinforcement area of

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 2726mm^2$$

The hogging moments in the sloped elements require more reinforcement. Thus, additional reinforcement is added.

$$A_{s,additional} = 6163mm^2 - 2726mm^2 = 3437mm^2$$

This lead to additional reinforcement with a diameter of $\phi 32 with a spacing of 180mm$.
This is applied over a length of approximately 3.5 meters. This length is just an assumption. This length can be determined with NEN-EN 1992.

Culvert
The culvert beams:

$$M_{ed} = 234 \cdot 10^6 kNm$$

$$A_s = 541mm^2$$

For the culvert beams, a net of $\phi 12 - 180mm$is applied. This leads to an applied reinforcement area of:

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 628mm^2$$

Culvert top/spillway bottom in transversal direction:

$$M_{ed} = 234 \cdot 10^6 kNm$$

$$A_s = 1609mm^2$$

For the reinforcement in the transversal direction of the culvert top, a net of $\phi 25 - 225mm$ is applied. This leads to a reinforcement area of

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 2180mm^2$$

Retaining walls

$$M_{ed} = 72 \cdot 10^6 kNm$$

$$A_s = 418mm^2$$

For the retaining walls, a net of $\phi 12 - 225mm$ is applied. This leads to an applied reinforcement area of:

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 502mm^2$$

M.6 Determining reinforcement dike 2

The same reasoning for all the matrixFrame models of dike 5 is applied for dike 2. However, the dimensions and loading do differ. Furthermore, for dike 2, situation 2 is governing for all structural components. Thus, situation 1 is not included in determining the reinforcement.

Dike 2, situation 2

Spillway
The method for determining the spring stiffness is also the same as for dike 5. Thus, only the input and output are given, see Table M.12.

Table M.12: Overview of the determined parameters of the bottom spillway/culvert roof element. The value of the bending stiffness does not include the reinforcement.

Input		
Width	b	1000 mm
Height	h	350 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9$ mm^4
Length	L	19080 mm
Output		
Spring stiffness	k	846 kN/m

Matrixframe model
The springs are evenly distributed with a distance of 0.5 meters.

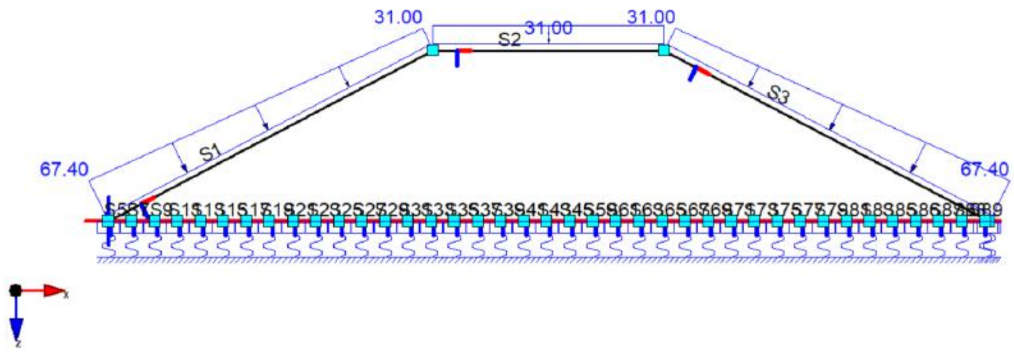


Figure M.18: Matrixframe model of the spillway for dike w with the loading of situation 2

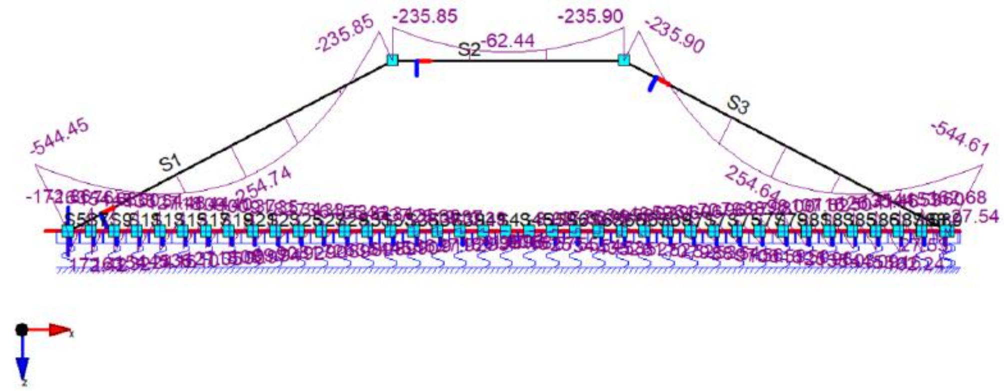


Figure M.19: Moments line of the spillway model for dike 2 with the loading of situation 2

Table M.13: Overview of the governing bending moments per spillway element and the corresponding required reinforcement area. The values are per unit width.

Spillway element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Top slab	-236	1929	0	0
Front Sloped slabs	-545	4977	255	2096
Back sloped slabs	-545	4977	255	2096
Bottom slab/roof culvert	-173	1175	173	1175

Culvert

Table M.14: Overview of the determined parameters of the culvert elements. The value of the bending stiffness does not include the reinforcement.

Input		
Culvert beams		
Width	b	1000 mm
Height	h	1000 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9$ mm^4

MatrixFrame model

Governing vertical characteristic reaction force from spillway = 30 kN.

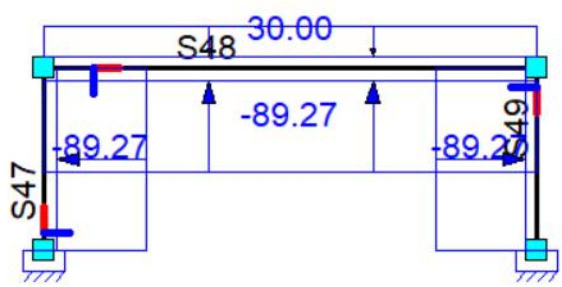


Figure M.20: Matrixframe model of the culvert for dike w with the loading of situation 2

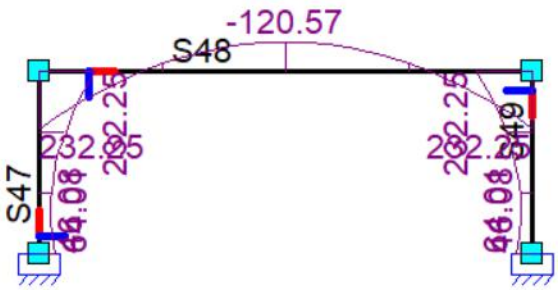


Figure M.21: Moments line of the culvert model for dike 2 with the loading of situation 2

Table M.15: Overview of the governing bending moments per culvert element and the corresponding required reinforcement area. The values are per unit width.

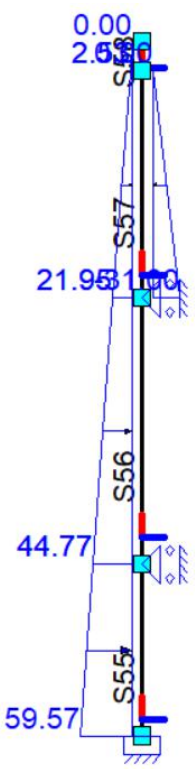
Culvert element	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Top slab (trans)	-121	813	233	1602
Beams	0	0	233	538

Retaining wall

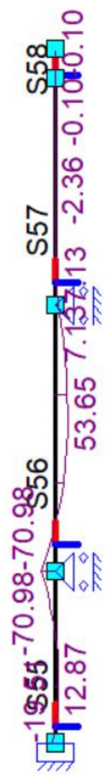
Table M.16: Overview of the determined parameters of the retaining wall. The value of the bending stiffness does not include the reinforcement.

Input		
Width	b	1000 mm
Height	h	400 mm
Reinforcement diameter	d	32 mm
Elastic modulus	E	34000 N/mm^2
Moment of inertia	I	$2.25 \cdot 10^9\ mm^4$

<u>Matrixframe model</u>



(a) Matrixframe model of the retaining wall for dike 2 with the loading of situation 2



(b) Moments line of the retaining wall for dike 2 with the loading of situation 2

Table M.17: Overview of the governing bending moments of the retaining wall and the corresponding required reinforcement area. The values are per unit width.

	Hogging moment [kNm]	Required reinforcement area [As,req] [mm2] top part	Sagging moment [kNm]	Required reinforcement area [As,req] [mm2] bottom part
Retaining wall	71	413	54	313

Applied reinforcement

Spillway
The reinforcement net is based on the governing bending moment in the top slab.

$$M_{ed} = 236 \cdot 10^6 kNm$$

$$A_s = 1929mm^2$$

This leads to bar spacing of (s):

$$n = \frac{A_s}{d} = \frac{1929}{25} = 3.93 \approx 4$$
$$s = \frac{b - 2 \cdot c}{n} = 225mm$$

This chosen net $\phi 25 - 225mm$ leads to an applied reinforcement of:

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 2180mm^2$$

The hogging moments in the sloped elements require more reinforcement. Thus, additional reinforcement is added.

$$A_{s,additional} = 4977mm^2 - 2180mm^2 = 2797mm^2$$

The additional reinforcement is chosen to be the same as dike 5. Thus, an additional reinforcement of $\phi 32mm$ with a spacing of 225 mm. This is also applied over a length of approximately 3.5 meters.

Culvert
The culvert beams:

$$M_{ed} = 233 \cdot 10^6 kNm$$

$$A_s = 539mm^2$$

For the culvert beams, a net of $\phi 12 - 180mm$ is applied. This lead to an applied reinforcement area of

$$A_s, applied = \frac{0.25 \cdot \pi \cdot d^2}{\frac{s}{1000}} = 628mm^2$$

Culvert top in transversal direction:

$$M_{ed} = 233 \cdot 10^6 kNm$$

$$A_s = 1602mm^2$$

For the reinforcement in the transversal direction of the culvert top, a net of $\phi 25 - 225mm$ is applied. This leads to a reinforcement area of :

$$A_{s,applied} = \frac{0.25 \cdot \pi \cdot^2}{\frac{s}{1000}} = 2180mm^2$$

Retaining walls

$$M_{ed} = 71 \cdot 10^6 kNm$$

$$A_s = 413mm^2$$

For the retaining walls, a net of $\phi 12 - 225mm$ is applied. This leads to an applied reinforcement area of:

$$A_{s,applied} = \frac{0.25 \cdot \pi^2}{\frac{s}{1000}} = 502mm^2 \quad \text{(M.17)}$$

M.7 Results of stresses from DIANA

The following results are the stresses in the steel reinforcement for dike 5 in situation 1.

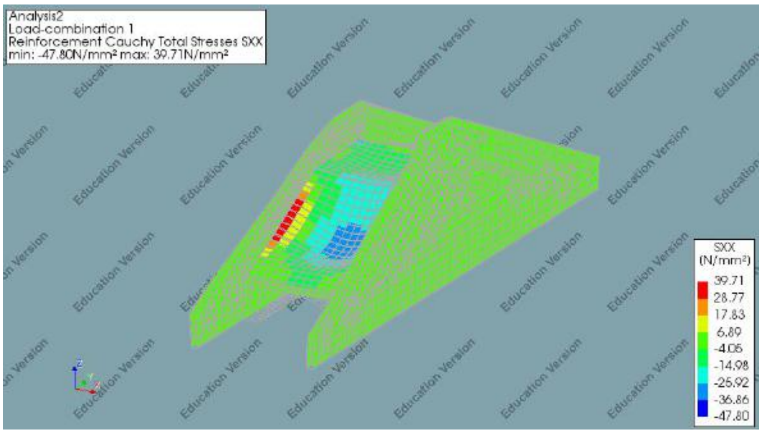


Figure M.23: σ_{xx} in N/mm² for steel reinforcement

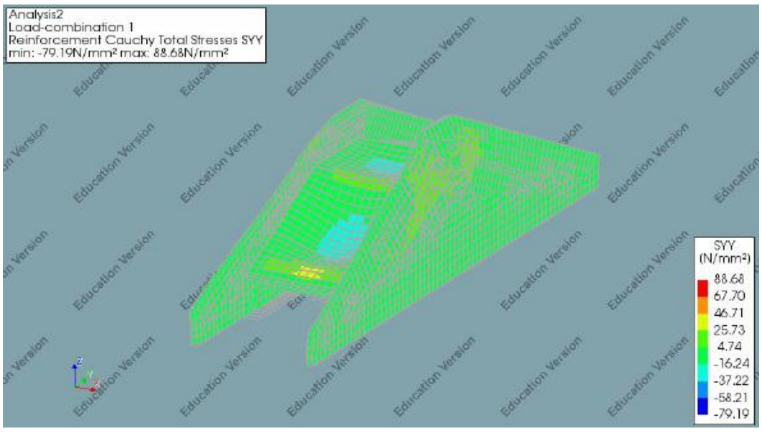


Figure M.24: σ_{yy} in N/mm² for steel reinforcement

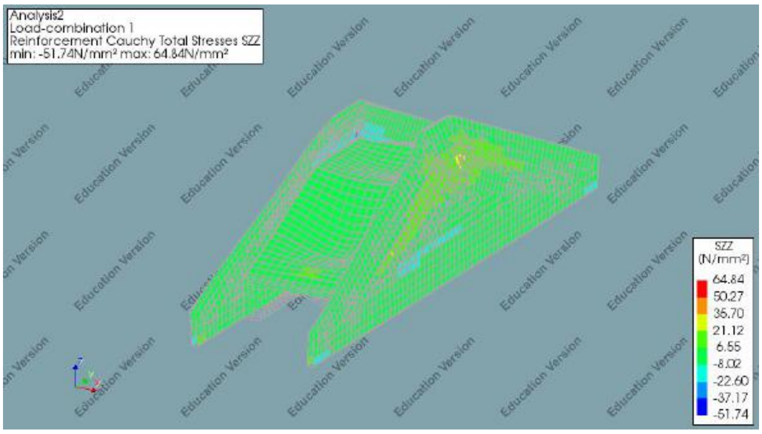


Figure M.25: σ_{zz} in N/mm² for steel reinforcement

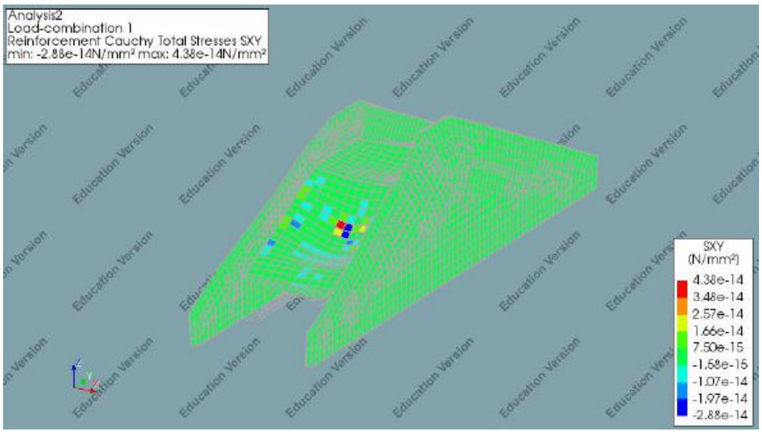


Figure M.26: σ_{xy} in N/mm² for steel reinforcement

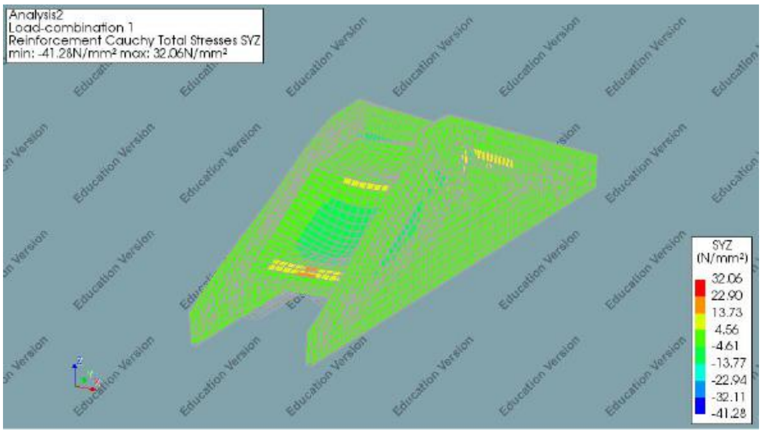


Figure M.27: σ_{yz} in N/mm² for steel reinforcement

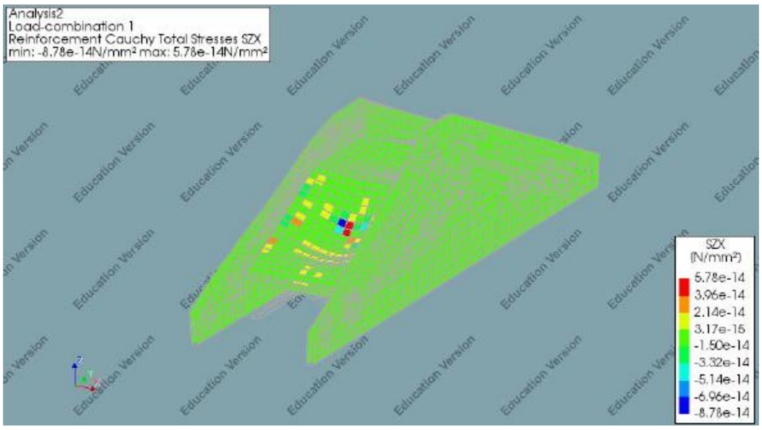


Figure M.28: σ_{zx} in N/mm² for steel reinforcement

The following results are the stresses in the concrete for dike 5 in situation 1.

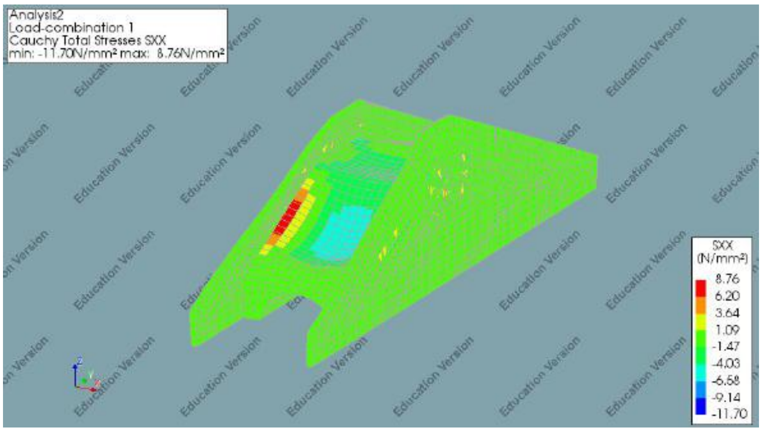


Figure M.29: σ_{xx} in N/mm² for concrete elements

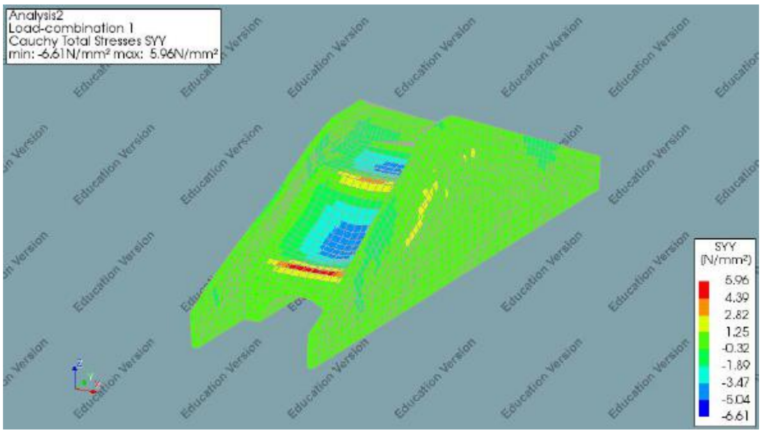


Figure M.30: σ_{yy} in N/mm² for concrete elements

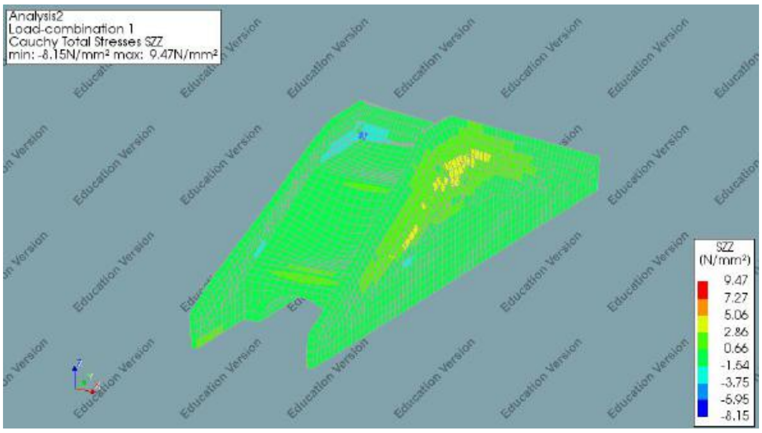


Figure M.31: σ_{zz} in N/mm² for concrete elements

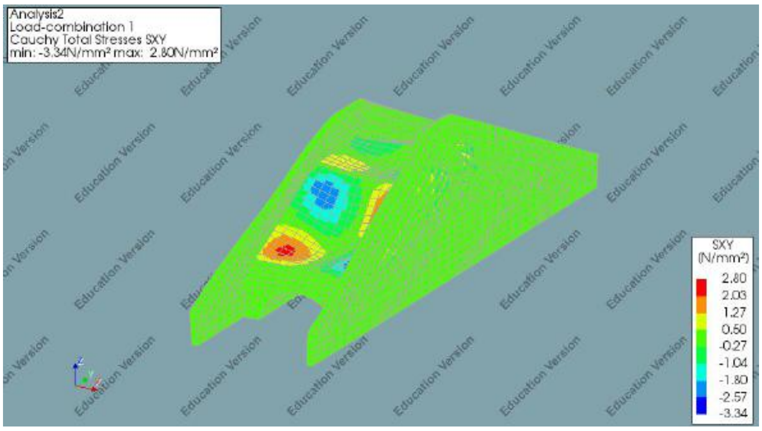


Figure M.32: σ_{xy} in N/mm² for concrete elements

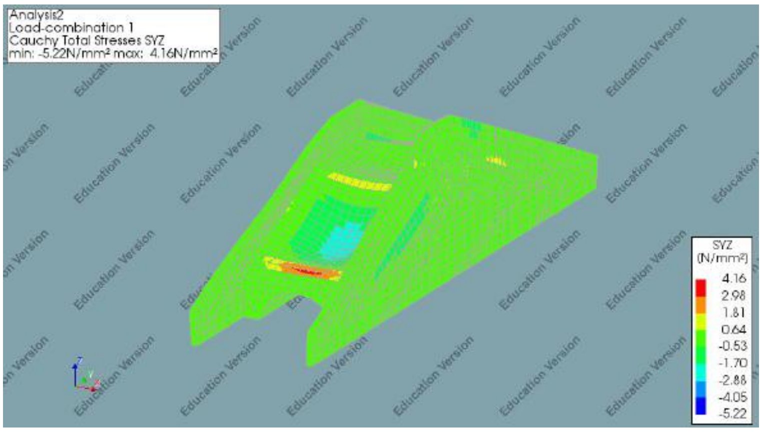


Figure M.33: σ_{yz} in N/mm² for concrete elements

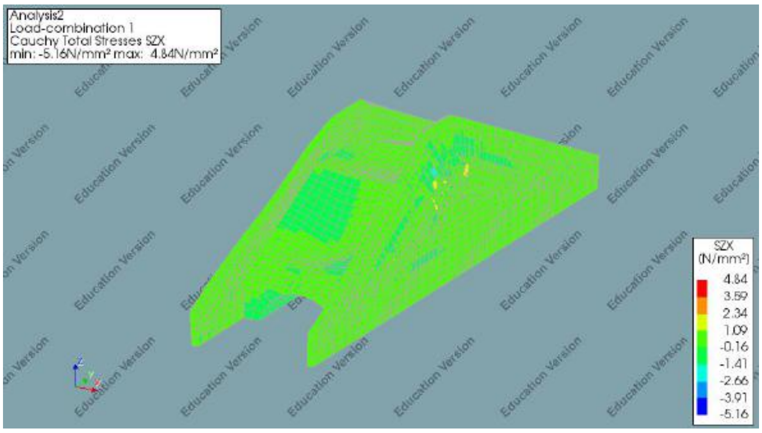


Figure M.34: σ_{zx} in N/mm² for concrete elements

The following results are the stresses in the steel reinforcement for dike 5 in situation 2.

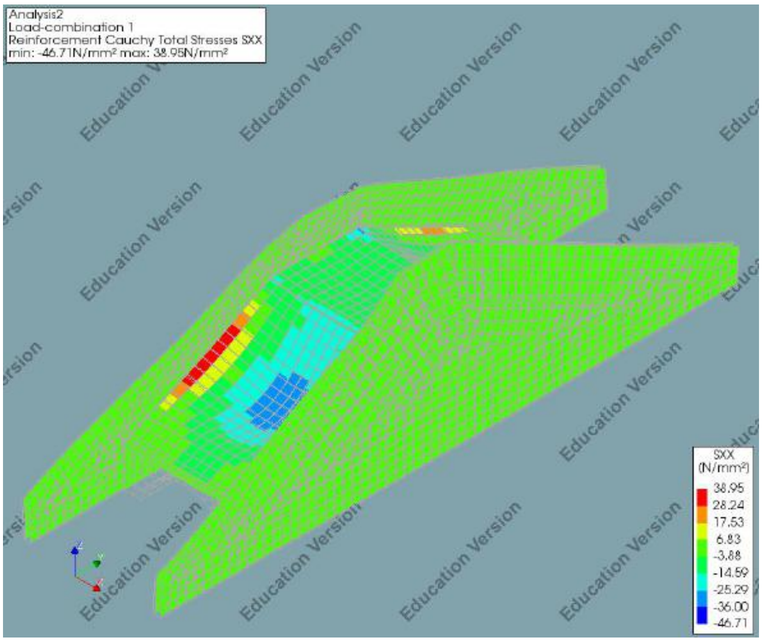


Figure M.35: σ_{xx} in N/mm² for steel reinforcement

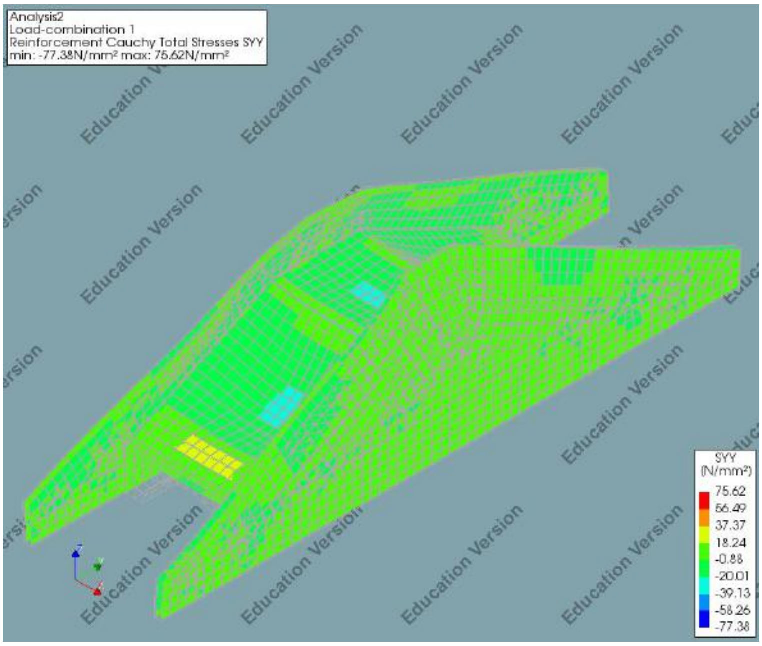


Figure M.36: σ_{yy} in N/mm² for steel reinforcement

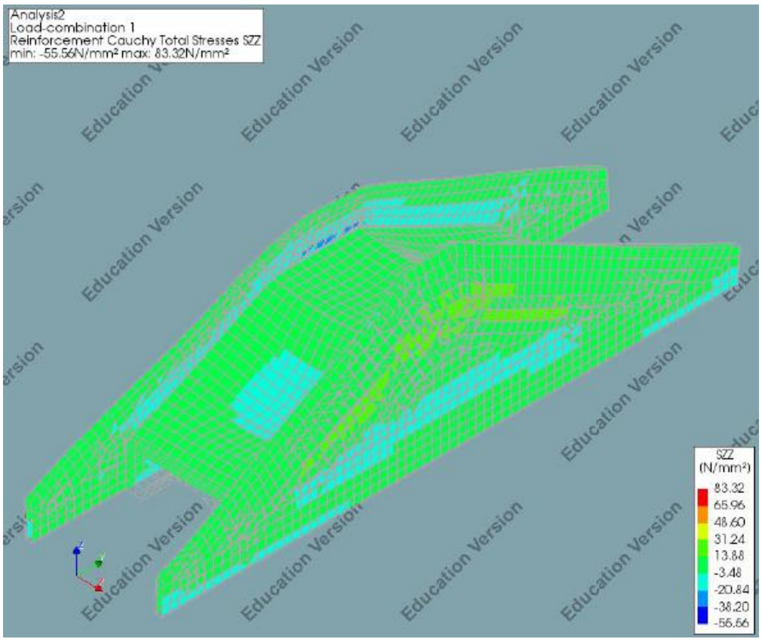


Figure M.37: σ_{zz} in N/mm² for steel reinforcement

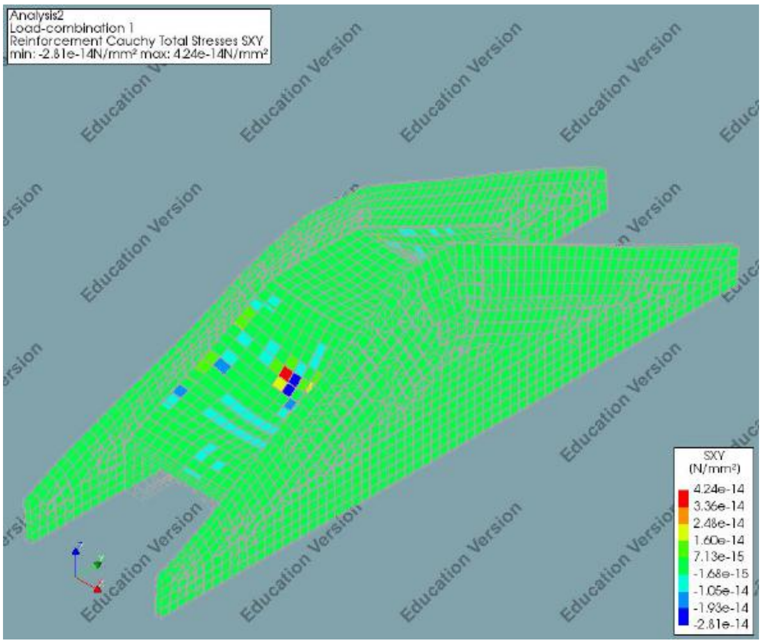


Figure M.38: σ_{xy} in N/mm² for steel reinforcement

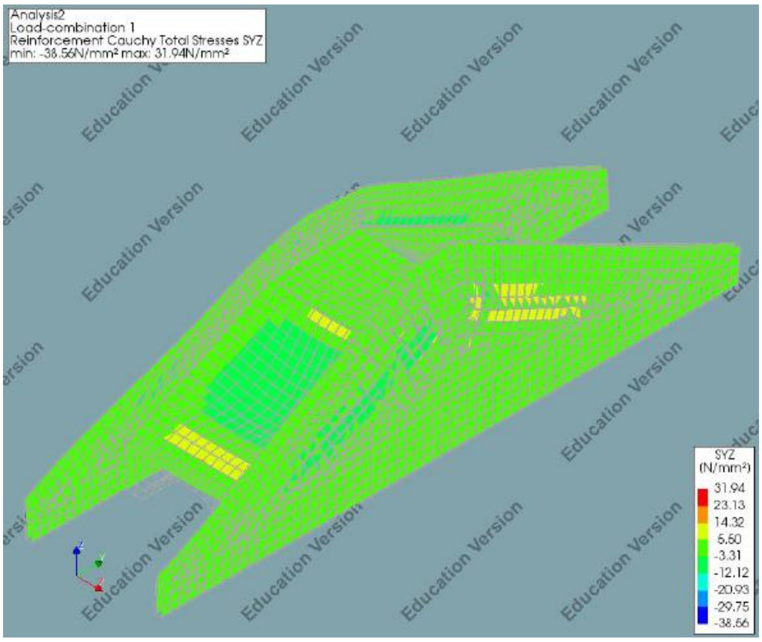


Figure M.39: σ_{yz} in N/mm² for steel reinforcement

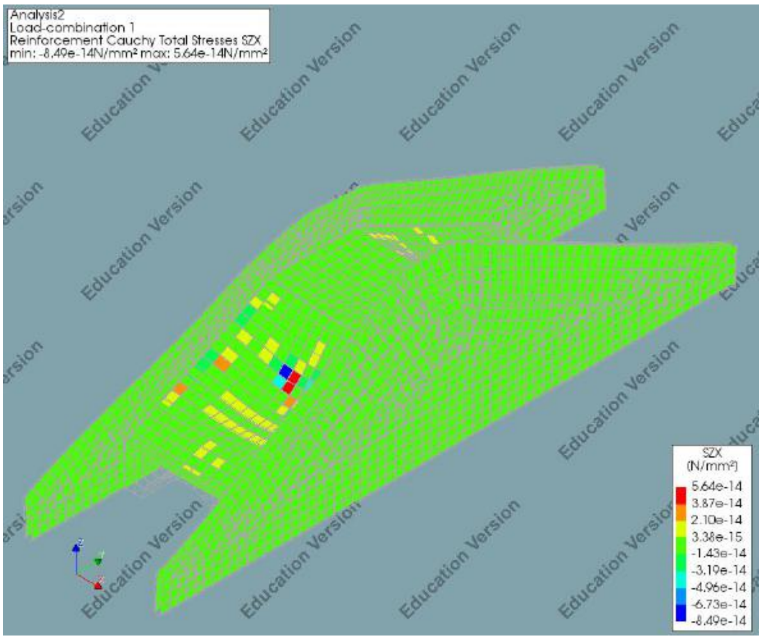


Figure M.40: σ_{zx} in N/mm² for steel reinforcement

The following results are the stresses in the concrete for dike 5 in situation 2.

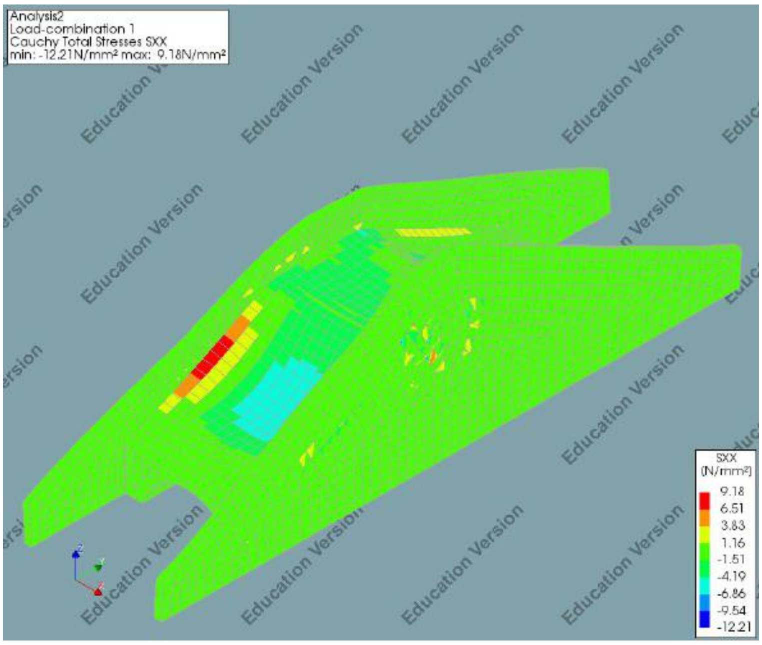


Figure M.41: σ_{xx} in N/mm² for concrete elements

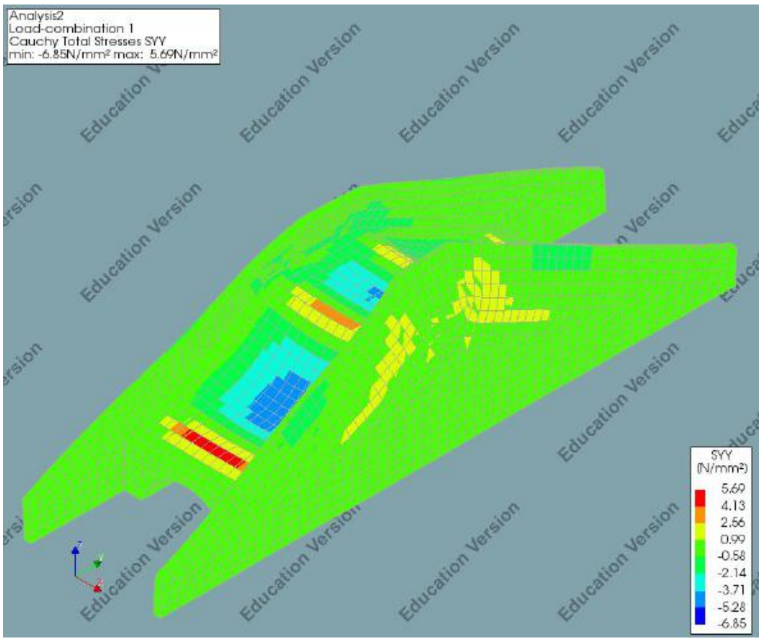


Figure M.42: σ_{yy} in N/mm² for concrete elements

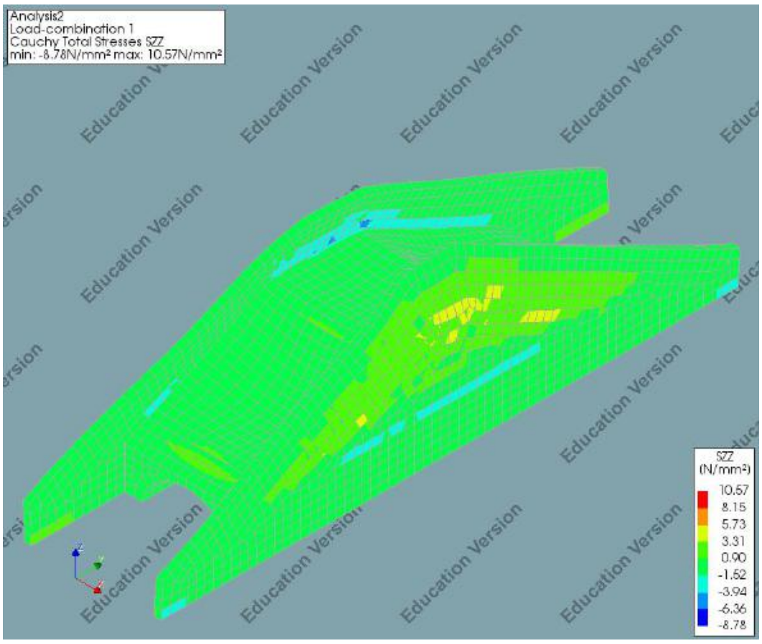


Figure M.43: σ_{zz} in N/mm² for concrete elements

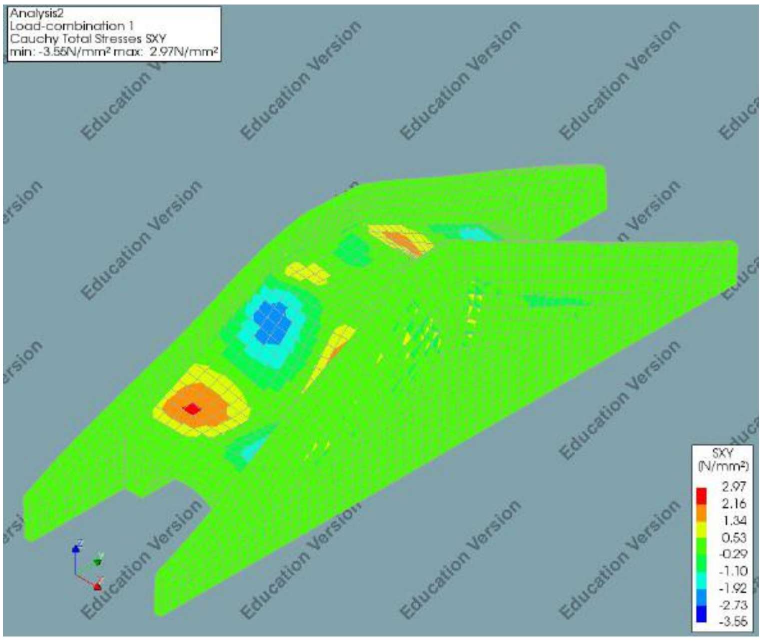


Figure M.44: σ_{xy} in N/mm² for concrete elements

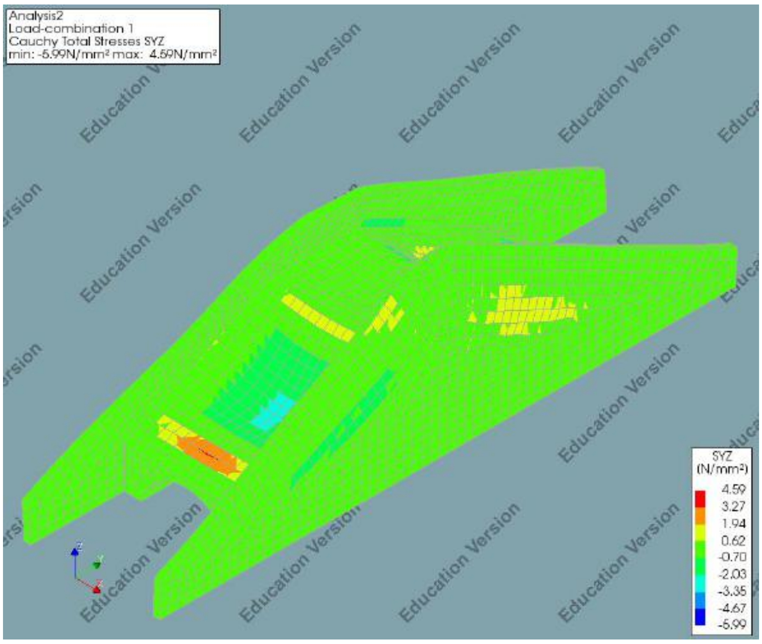


Figure M.45: σ_{yz} in N/mm² for concrete elements

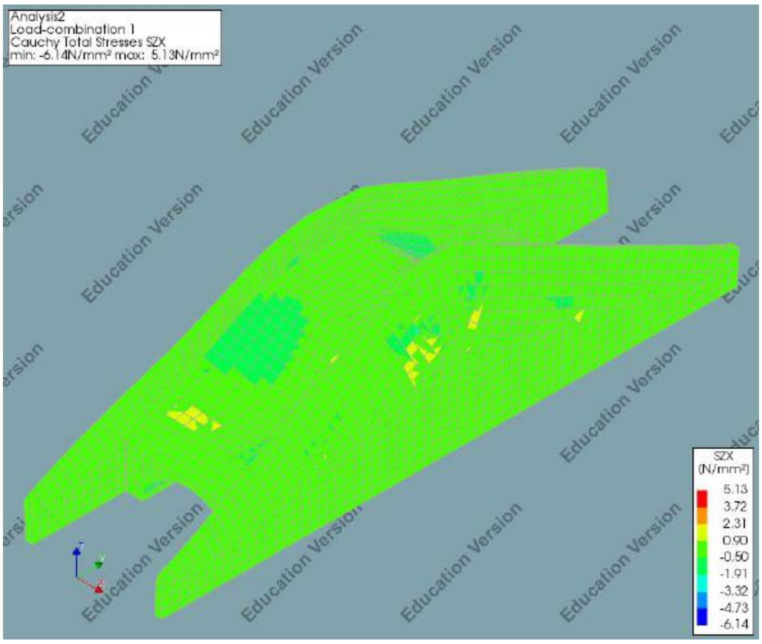


Figure M.46: σ_{zx} in N/mm² for concrete elements

Appendix N: Log of Multidisciplinary Project

Weeknr	Week	Activity	Starting date	Own deadline	Check	Official deadline	Submitted?
15	0	- Kick-off meeting with all supervisors	11/04/2022	-	Y	-	
		- Arrival at Cartagena	16/04/2022	-	Y		
16	1	- Group meeting: discussion on expected work attitude, office hours and getting ready with software downloads.	19/04/2022	-	Y	-	
		- Start at UPCT: tour on campus + getting started with project. - First attempt at HEC-RAS simulation example, - First Sketchup model, - Writing Chapter 1 (introduction). Due to time constraints, the first supervisor meeting is possible on Monday 25 April. Email for supervisor meeting.	20/04/2022	-	Y	-	
		- Implementing first discharge data provided by supervisor García Bermejo. - Writing of chapter 2 (function analysis, stakeholder analysis, process analysis). - Writing of appendix A (extreme discharge value).	21/04/2022	-	Y	-	
		- Starting to debug HEC-RAS model. Still attempting to get useful results. Prepared questions to ask supervisors on Monday for a meeting. - Start Chapter 3 (Boundary conditions and project requirements), - Illustrations of ZAC in use.	22/04/2022	-	Y	-	
		- Reminder to Spanish supervisors to meet on Monday as no reply has been received.		22/04/2022	Y		
		Submit 1st weekly report		22/04/2022			Y
-							
17	2	- Meeting with Spanish supervisors	25/04/2022		N	-	
		- Corrections after feedback on: - Report - Start of Model locations alternatives (chapter 4) - Finalizing chapter 3 - HEC-RAS first model is running (1D), check by supervisor next day. - Supporting figures created	25/04/2022		Y	-	
		- Writing multi-criteria analysis chapter 4 and decision on location - Meeting with José Carrillo on HEC-RAS application. - Corrections on report after feedback of Kees Sloff	26/04/2022		Y		
		- Mini meeting with Juan Bermejo on neglicence of sediment transport in the model. - Start hand calculations on ZAC structure. - 2 new people getting started with modelling in HEC-RAS.	27/04/2022		Y	-	
		- Start of modelling in 2D in HEC-RAS. Aim to know about the differences between 1D and 2D modelling.	28/04/2022		Y	-	

		- First attempt of implementing a dike with a culvert in 2D HEC-RAS. - Continuation on ZAC hand calculations to estimate first order of dimensions. - Introduction of HEC-RAS 1D modelling for new group members.					
		- Collect basic measurement equipment at UPCT to bring on trip.	28/04/2022		N		
		Trip to project locations by car. Information to be gathered: - Feeling of area - Soil type - Photos of locations - Measuring basic dimensions: O(m)	29/04/2022		Y		
		Plan mid-term meeting with all supervisors (date to be discussed, preferably start week 5: Monday 16 May)		29/04/2022	Y		
		Submit 2nd weekly report		29/04/2022			Y
18	3	- Continue on ZAC hand calculations: Analytical maple file has been made for 1 ZAC Reservoir. Result is a hydrograph which is approximately the same as expected hydrographs. Changes in dimensional parameters can be made to evaluate the corresponding hydrograph.	02/05/2022	04/05/2022	Y		
		- Apply series of dikes + culverts in 2D HEC-RAS model to mimic ZAC structure. The 2D model is showing expected results but is not yet optimized. Spillways and dams are modelled. Culverts are being modelled.	02/05/2022	06/05/2022	Y		
		- model dike+ culvert in 1D in HEC RAS. Model without ZAC is working and shows results as can be expected. 1D - Model with ZAC is still not without bugs and is under construction.	02/05/2022	06/05/2022	Y		
		- Meeting with José Carrillo on 12.30 on the progress that has been made in 1 week. HEC-RAS models are shown and results were discussed.	03/05/2022	03/05/2022	Y		
		- Sketches on design situations of structure that could determine ULS and start of hand calculations (strength, stability, piping). Sketches on failure mechanisms. Update 06/05/22: The 2D and 1D with ZAC structure are still under construction. This takes more time than expected and therefore the modelling has a priority compared to making sketches of the failure mechanisms	-	04/05/2022	N		
		-Meeting José Carrillo for a discussion about the results from the 1D model including ZAC structure.	06/05/2022	06/05/2022	Y		
		Starting with the culvert & spillway analysis modelling program HY-8	06/05/2022		Y		
		Submit 3rd weekly report		06/05/2022			Y
19	4	- Different culvert shapes are considered in HY-8. HY-8 is combined with self-created Maple script to be able to optimize dimensions.	09/05/2022		Y		

		- Finding method to quantify overflow area using QGIS (required for cost-analysis in the future)					
		- 1D model in HEC-RAS is finalized. It is decided to continue with 2D modelling only. - Model of culvert built in Revit - 2D HEC-RAS new alternative modelled (channel downstream) - Meeting with Spanish supervisor	10/05/2022		Y		
		- The elements that need to be designed are identified for structural modelling. - Constructability of the ZAC is considered.	11/05/2022		Y		
		- Start modelling with structural modelling software DIANA. - Start with chapter Functional design. - Determination of required Q-plan in 1D HEC-RAS model by iteratively trying when overflow happens.	12/05/2022		Y		
		- Adaptation and improvement of 2D HEC-RAS model. More precise structures and grids. - Rewriting of chapter 5: explanation of 1D model set-up and limitations.	13/05/2022		Y		
		Submit 4th weekly report		13/05/2022			Y
20	5	- Finalizing chapter 5 – 1D model. - Start with structural design alternatives - 2D HEC-RAS model optimization (Dike, spillway, channel)	16/05/2022		Y		
		- Continuation chapter 6: Functional requirements - Meeting with Spanish supervisor - Determining schematics of design loads on structural elements	17/05/2022		Y		
		- Organize mid-term report better - Start preparation mid-term presentation - Failure mechanisms and load combinations	18/05/2022		Y		
		- Preparing PowerPoint presentation figures and animations - Constructability and MCA criteria for structural design	19/05/2022		Y		
		- Mid-term presentation - Start writing Chapter 7 – Structural design		20/05/2022 (send presentation)		20/05/2022	Y
		Submit mid-term report				20/05/2022	Y
21	6	- Structural design: Efficiency of spillways determined (C_D values) - Optimization on culvert and spillway dimensions - Start of writing of 2D model in chapter 5	23/05/2022		Y		
		- Meeting with supervisors - Finished applying feedback after midterm presentation (project initiation explained, functional design split better from structural design) - Dike profile dimensioned in D-geostability	24/05/2022		Y		
		- Alternatives of types of spillways, wing walls and culvert shapes in MCAs	25/05/2022		Y		

		- 2D HEC-RAS model final dimensions of dikes (redesign of inundation of dikes), spillways and culverts - Continuation of writing chapter 6 and 7					
		- Structural models: Built in Revit - Analyzing the impact of 2 consecutive large storms (not design storms).	26/05/2022		Y		
		- Creating the structural model in DIANA - Model of ZAC alternative in the case of ZAC without channel is modelled	27/05/2022		Y		
		Submit 6th weekly report		27/05/2022			Y
22	7	- Chapter 7 section Dikes written - Structural design: hand calculations on loads - Setting up DIANA model	30/05/2022		Y		
		- Structural design: hand calculations on reinforcement and crack width - Chapter 6: ZAC volume calculation - Finalizing chapter 5	31/05/2022		Y		
		- Chapter 6 parameters of spillway and culvert - Setting up DIANA model	01/06/2022		Y		
		- Chapter 6 ordered and rewritten - Setting up DIANA model - Construction execution pictures made - Scour protection in US & DS channel calculated	02/06/2022		Y		
		- Constructability written in chapter 7 and appendix J - Matrixframe calculations - Scour protection written in chapter 7 - Start of structural design of channels	03/06/2022		Y		
		Submit 7th weekly report		03/06/2022			Y
23	8	- Chapter 6 finalizing - Structural design of downstream channel - Deciding on output from DIANA model - Writing appendix K - First check on readability of main report - Collecting information from report for chapter 8 Final Design	06/06/2022		Y		
		- Writing chapter 7 Structural Design - First check on readability of main report - Writing chapter 8, creating figures	07/06/2022		Y		
		- Continuation on writing chapter 7 Structural Design - First check on readability appendices - Writing of recommendations and sources of uncertainty (8.3 & 8.4)	08/06/2022		Y		
		- Packing all stuff and cleaning apartment	09/06/2022		Y		
		- Departure from Cartagena	10/06/2022		Y		
		No weekly report this week		-			
24	9	- Dike calculations writing in appendix L - Writing conclusion in 8.2 - Writing 8.1.4 & 8.1.5 - Finalizing chapter 7 - Finalizing appendix I	13/06/2022		Y		
		- Checking appendices - Writing summary - Finalizing appendix I	14/06/2022		Y		

		- Writing appendix L					
		- Improving lay-out, readability and grammar check on final draft report according to assessment form - Finalizing weekly log	15/06/2022		Y		
		-	16/06/2022		-		
		Submit final draft report				17/06/2022	Y
25	10	Feedback on final draft report by supervisors				24/06/2022	Voorendt: Y Sloff: 27-06-22 Bermejo: N Carrillo: N
26	11	- Improving on feedback	27/06/2022		Y		
		- Improving on feedback	28/06/2022		Y		
		- Improving on feedback - Preparing final presentation	29/06/2022		Y		
		- Preparing final presentation	30/06/2022		Y		
		Final presentation & submit final report				01/07/2022	Y