MASTER OF SCIENCE GRADUATION THESIS

MULTIFUNCTIONAL FLOOD DEFENCES

RELIABILITY ANALYSIS OF A STRUCTURE INSIDE THE DIKE

J. van Mechelen

Delft, December 2013

TU Delft Delft University of Technology

Deltares Enabling Delta Life

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Preface

Upon completion of the Civil Engineering program and achieving the title Master of Science at the Delft University of Technology, the graduation thesis 'Multifunctional Flood Defences - Reliability Analysis of a Structure Inside the Dike' is presented in this report. This report gives a stimulus to multifunctional use of flood defences by means of the development of a framework for the design and safety assessment of dikes containing a structure.

Many people have, to a greater or lesser extent, contributed to the development of this report. First of all I would like to thank the members of my graduation committee prof. dr. ir. S.N. Jonkman, ir. W.F. Molenaar, ir. K.C. Terwel and dr. ir. G. van Meurs for their input, supervision and feedback. Secondly I would like to thank Deltares for sharing their knowledge and facilities. Last but not least, I would like to thank my family, friends, fellow students and colleagues for their interest, support and comments.

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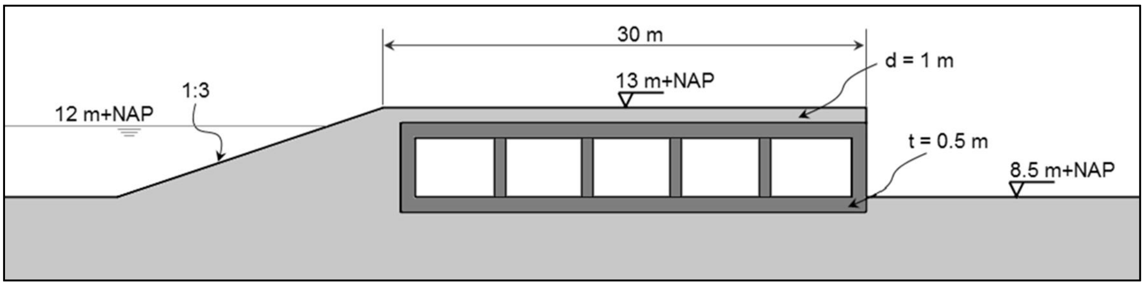
Abstract

Multifunctional use of the flood defences is as old as the flood defences themselves. Historically infrastructure, housing and livestock are located on the flood defences. Over the years, the cities along the rivers became bigger and bigger. With the still increasing demand for spatial development it is inevitable to also start combining other functions with the flood defences. The underground space of the flood defences is not yet used. Constructing a structure inside a dike is a solution to create more space.

With multifunctional use of the flood defences are a lot of different parties involved with each its own interest and purposes. This results in a difficult and devious design process. The technical difficulties are researched in the master thesis in order to optimise this design process. The Dutch guidelines for flood defences as well as the Eurocodes are relevant for the design of multifunctional flood defences. Dealing with the hydraulic loads on the structure leads to uncertainties for the reliability of the flood defence. Applying partial factors on forces resulting from design water level with already a very small exceedance probability seems to introduce excessive amount of reliability into the calculations. The assessment of the multifunctional flood defence has a lot of resemblance with the assessment of a hydraulic structure. Except the failure mechanism reliability closure, which is absent in the assessment because the multifunctional flood defence is permanently closed. The failure mechanisms that require the most attention are:

- Overtopping
- Piping
- Overall stability
- Structural strength
- Connections

A case study is performed to research the issues during the design process of a multifunctional flood defence. The Grebbedijk is chosen as location for the case study because of the demand for spatial development as well as the demand for flood safety. The Grebbedijk is a relatively small dike protecting a relatively large area with a high economic value. The Grebbedijk is often named as a possible first Delta dike. Together with the demand for parking close to the city centre of Wageningen, this location meets both criteria for a multifunctional flood defence. A design is made for a parking garage inside the dike, see the figure below.



Layout of the multifunctional flood defence

The design is assessed on the five previous stated failure mechanisms. The assessment confirmed the idea that the forces resulting from the water level are treated in a too conservative way by applying partial factors over design water levels with already small exceedance probabilities. This effect is obtained in the calculations for the failure mechanisms overall stability and structural strength. The other failure mechanisms do not result into insurmountable problems. Furthermore, the construction phases and the possibilities of changing the dimensions of the structure are analysed. Flood defences have to be at full strength during the storm season, leaving only the summer period to construct the multifunctional flood defence. The analysis of the construction phases showed that the construction time of the multifunctional flood defence is smaller than the available summer period. It should be noted that there have been a couple of assumptions to analyse the construction time. However, the construction time is in this case that important that this requires further research to confirm the required construction time. Changing the dimensions of the structure is in principle possible in all directions. The width of the multifunctional flood defence is very much depending on the specific location and the available amount of space at that location. Constructing in depth or height will not lead to problems which are not solvable from a structural point of view.

In order to analyse the reliability of the overall stability and the structural strength of the multifunctional flood defence, first order reliability method (FORM) analyses are carried out. Three failure mechanisms are considered: (i) horizontal stability, (ii) overturning stability and (iii) strength of the wall. The failure probability of each failure mechanism is calculated. All three failure mechanisms have a lower failure probability than the required failure probability. The partial factors used in the semi probabilistic approach are calibrated in order to find the correct partial factors. The calibrated partial factors for the force related to the water level showed a variation equal to the proportionality of the force to the water level. The partial factors are in this case not useful. The exceedance probabilities of the design water levels resulting from the FORM analyses are very similar and in the same order as the failure probabilities. This resulted in the conclusion that using a design water level with an exceedance probability equal to the failure probability introduces enough reliability into the calculations. The partial factors for the forces not related to the water level were all very close to one. Resulting in the conclusions that using partial factors of 1.1 for unfavourable force and 0.9 for favourable force, would be sufficient to obtain the target failure probability. The applicability of these partial factors is not unlimited. The water level is the most important parameter in these calculations. Other forces might become more important if for instance a tall building is constructed on top of this structure. The wind force on the tall building might be of much more importance for the overall stability than the water level. Further research to the limitations of these partial factors has to be carried out to say something about the applicability of the partial factors proposed in this master thesis.

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1 Introduction

1.1 Background

Anno 1000 the first dike was built in the Netherlands. A dike is a man-made flood defence which protects the hinterland against flooding. In the Netherlands, a distinction is made between two types of flood defences, primary flood defences and secondary flood defences. Primary flood defences protect against open water, which has an uncontrollable water level as the sea, major rivers and lakes. Secondary flood defences, often called regional flood defences, protect against internal waters which have a stable or controllable water level.

The Netherlands is a small country with a relatively large population. This gives a lot of pressure on spatial development, which results in multifunctional use of space. Multifunctional use of space is combining more than one purpose for the same area. With the pressure on spatial development, multifunctional use has attracted a lot of attention. Dikes are already used multifunctional, namely for roads, housing, recreation, nature and livestock. All of these have a relatively low impact, or are already well incorporated into the design and safety assessment of dikes.

The primary function of dikes is the protection of land against flooding, but the dike can be used for other purposes as well e.g. the underground space below the dike is not used yet, but with the increasing pressure on spatial development in combination with dike reinforcement, this may be necessary in the near future. On several dike reinforcement projects the option of the use of underground space is already considered as an alternative. The underground space can be used for parking, infrastructure, housing, shops or offices.

The delta in the Netherlands is the best protected delta in the world. In order to ensure it will remain so in the future, the Delta Program is created. The Delta Program has the task to guarantee the flood safety and freshwater supply in the Netherlands and has formulated five Delta Decisions which they will present to the government in 2014. The five Delta Decisions are on the following topics:

- Flood safety
- Freshwater strategy
- Water level management IJsselmeer
- Rhine-Meuse delta
- Spatial adaptation

In the Delta Program, the following question is asked in relation to multifunctional use of flood defences (Deltaprogramma, 2010):

> "The Delta Commissioner states that current policies and instrumentation related to flood defences, does not allow multifunctional use. Similar to the study of delta dikes, the Delta Commissioner recommends to examine how impediments can be removed while maintaining the flood defence function of existing flood defences. The government agrees with this recommendation."

The Delta Commissioner recommends examining the impediments of multifunctional use of flood defences and how these impediments can be eliminated, while maintaining the flood defence function. Feasibility studies and research on the impediments have already been performed with positive outcomes, but the available guidelines are not completely suitable for the design and safety assessment of multifunctional flood defences. Engineering companies want to design multifunctional flood defences but without guidelines it is difficult.

Multifunctional flood defences is a very broad term, any kind of shared use of the flood defence is basically multifunctional use. There are also a lot of different types of flood defences imaginable. The combination of any kind of shared use on different types of flood defences results in endless possibilities for multifunctional use of flood defences. All these combinations require a different approach, because different failure mechanisms play a role on different types of flood defences and other kinds of shared use result into other demands on the flood defence.

This thesis focuses on a specific kind of multifunctional use of flood defences; a structure inside of a dike. The function of this structure can for instance be parking, housing, shops or catering facilities. The function of the structure results in different possibilities for the location of the structure in the cross section. A parking garage could completely be located below ground level (except for one or more entrances) but a house needs at least a part of the structure to be above ground level.

1.2 Objective of the thesis

The central objective of this thesis is:

- Finding a way to design and assess a dike with a structure inside in a reliable manner.

This is achieved by answering the following research questions:

- What aspects of the design, safety assessment and the risk analysis of dikes and hydraulic structures can be used for a dike with a structure inside?
- Based on the case study: What are the attention points for the design and safety assessment of the dike with a structure inside?
- Based on the case study: What are the failure probabilities of the failure mechanisms and do the partial factors used in the semi probabilistic approach correspond with the ones obtained from the reliability analyses?

1.3 Structure of the report

The structure of the report and the relations between the chapters is presented in Figure 1-1. Chapter two, three and four provide the theoretical framework which is required for the case study presented in chapter five. The uncertainty in the failure probability of the case study is than further investigated in chapter six. The thesis is evaluated in chapter seven by presenting the conclusions and recommendations. For each chapter are the following key questions kept in mind:

Chapter 1: What is the objective of the thesis and how is it achieved?

Chapter 2: What are multifunctional flood defences and why are they not built yet?

Chapter 3: How are dikes and hydraulic structures designed and assessed and how can this be used for multifunctional flood defences?

Chapter 4: How is dealt with risk within the flood defence system and what does this mean for multifunctional flood defences?

Chapter 5: How can multifunctional flood defences be designed and what are the issues encountered in the design?

Chapter 6: What is the reliability of the multifunctional flood defence and what does this mean for the design of multifunctional flood defences?

Chapter 7: What are the answers to the key questions and what are the recommendations for further research?

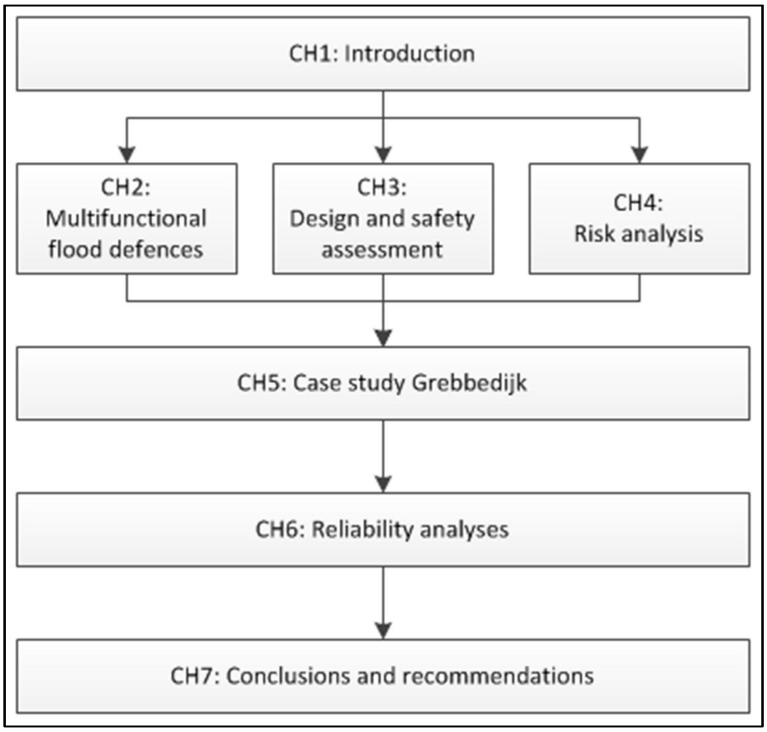

Figure 1-1: Structure of the report

2 Multifunctional flood defences

In this chapter multifunctional use is further elaborated and categorised in order to narrow the scope of this thesis. The impediments are described on various disciplines that come together with multifunctional use of the flood defences. The disciplines that are considered are; financial, spatial, technical and governmental.

2.1 Categorisation of multifunctional flood defences

The location of the structure with respect to the dike determines the influence on each other, so therefore different types are distinguished. In Figure 2-1 the different types of multifunctional use of flood defences in combination with a structure are shown. There are eight different types of flood defences distinguished, four (A, B, C and D) in which the structure and the flood defence are separated and four (E, F, G and H) in which the structure is part of the flood defence.

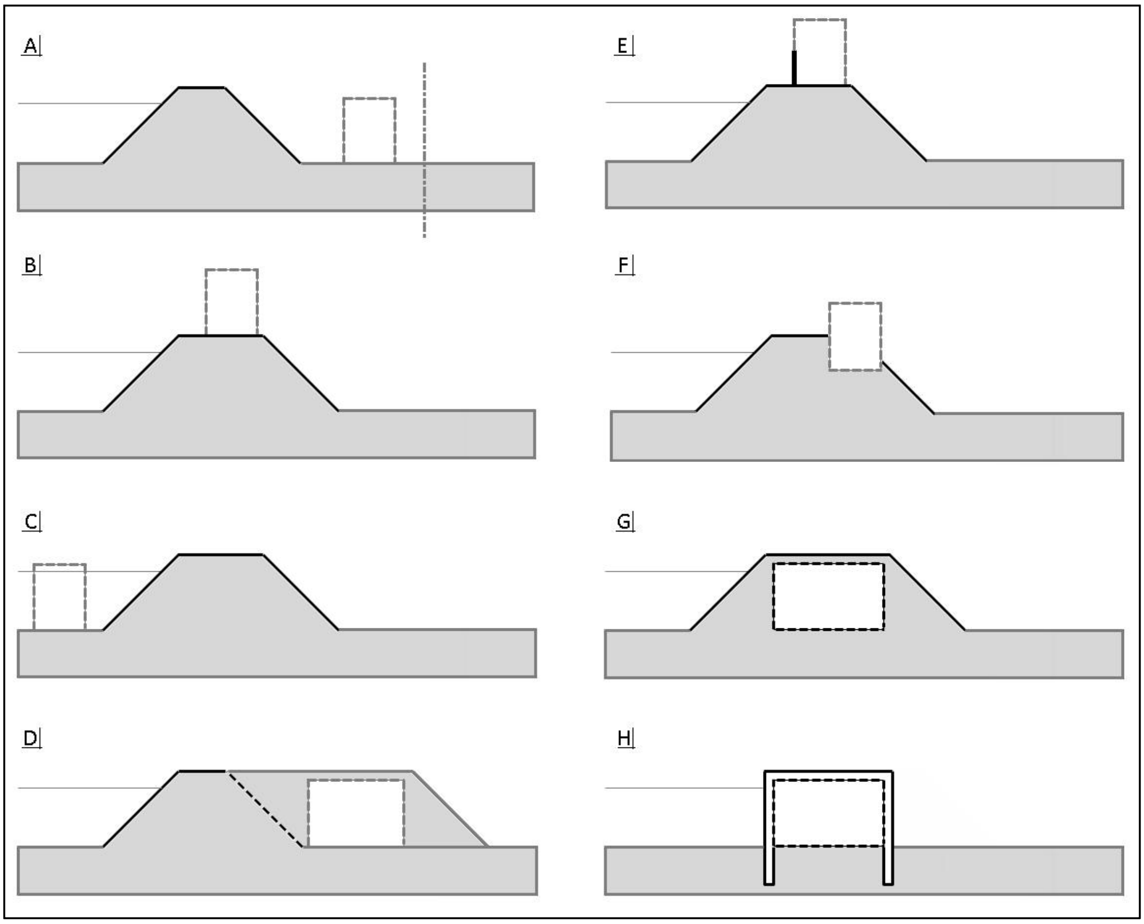

Figure 2-1: Different types of multifunctional use

The eight types of multifunctional flood defences are described as follows:

A. The structure and flood defence are completely separated but the structure is located within the influence zone of the flood defence. It has in this case little influence on a few failure mechanisms.
B. The structure is located on top of the flood defence (could also be located on the inner slope) but does not contribute to the water retaining function of the flood defence. The

influence of the structure can be included by a reduction of strength of the flood defence.

C. The structure and flood defence are completely separated but the structure is located within the influence zone of the flood defence. It is more or less the same as type A, but in case of a high water the object is exposed to the high water level and the resulting forces.
D. In this concept the flood defence and structure appear to be one from the outside, but within the flood defence there is clearly a separation of the two functions.
E. This type is the first type in which the structure contributes to the water retaining function of the flood defence. The structure can be used as an increase in strength, instead of traditional dike reinforcement.
F. More or less the same type as type B, but in this case the structure is located partly inside the dike. The resistance against several failure mechanisms is decreased in this case.
G. The structure is inside of the flood defence and is, in contradiction to type C, located in the water retaining part of the flood defence. Failure of the structure directly affects the water retaining function. The safety assessment of a dike is not adequate anymore; the flood defence should instead be assessed as a combination of a dike and a hydraulic structure.
H. The flood defence consists only of a structure, no soil materials are used for the water retaining function. The multifunctionality is introduced by building a structure in the flood defence.

Since there are dikes people intend to live on and next to dikes for several reasons. Types A, B and C are therefore common types of shared use, i.e. multifunctional use of the flood defence. Type D is a variation of the previous three types, but has still a clear separation of the two functions within the soil body. A recently new type is type H, but in this case the flood defence is not a dike anymore but completely a structure. Types E, F, G and H are the most interesting types to research, because the functions of building and flood defence are integrated. The focus of this thesis is on type G because this type has the largest interaction between structure and dike and therefore the largest influence on each other.

Multifunctional use of a flood defence offers the opportunity to generate revenues, based on the demand for spatial development and flood safety, see Table 2-1 (Briene et al., 2012).

Table 2-1: Perspective for revenues of multifunctional use

Perspective for revenues		Spatial development		
		No demand	Low demand	High demand
Flood safety	No demand	--	-	+/-
	Low demand	-	+/-	+
	High demand	+/-	+	++

The combination of demand for flood safety and spatial development may lead to synergy. This means that the costs minus revenues of the two separated components are lower than the two components together. The degree of synergy determines the demand for multifunctional use of flood defences. High demand for spatial development may lead to premature adaptation of a flood defence. When the demand for flood safety is high, multifunctional use may lead to cost distribution over the different users. Simply stated, this means that the costs minus revenues of the multifunctional flood defence must be lower than those of the traditional dike reinforcement plus the structure located somewhere else. All the costs and revenues over the design lifetime of the structure must be considered.

2.2 Impediments for multifunctional use

Although multifunctional use is an opportunity to use the flood defence for more than one purpose, there are also some impediments stated in the literature. The impediments for multifunctional use of flood defences are divided over different disciplines. This paragraph describes the impediments of the different disciplines to give an overall impression of the impediments. The impediments are divided into four categories: financial, spatial, technical and governmental.

2.2.1 Financial impediments

Financial impediments are related to the financial consequences of multifunctional use for the realisation, management and maintenance of a flood defence. The following impediments are identified (Ellen et al., 2011a):

- Change of insight and climate can result in dike improvement, which can lead to costs for the other user. If it is unclear how the costs are distributed over the users and the flood defence owner, the users can renounce of shared use.
- Several water boards in the Netherlands claim that they should have no extra costs due to multifunctional use of a flood defence. This attitude makes it hard to realise multifunctional use of a flood defence.
- Due to multifunctional use of a flood defence the flood risk can change. It is not clear who carries this increase of flood risk. In principle the water board is responsible for the flood safety in the Netherlands. The flood risk is a boundary condition for a flood defence. So if the flood risk increases due to multifunctional use this should be compensated, this entails extra costs and liability for the other user.
- It is difficult to balance the extra costs of multifunctional use against the benefits. Also the distribution of the costs and benefits over the different parties is unclear.
- Multifunctional use of a flood defence asks for an integral approach, but the water boards have no expertise on urban development. Hiring people with those skills is costly and they are not subject to the primary task of the water board, ensuring flood safety.

2.2.2 Spatial impediments

Obviously spatial impediments for multifunctional use occur locally, but only the spatial impediments related to flood safety are considered. The following impediments are identified (Ellen et al., 2011a):

- Due to the high costs, multifunctional use is often only possible through cooperation with private partners. Private partners find spatial objectives and exploitation of more importance than the flood safety. For the water boards this is a threatening situation because they are primarily responsible of the flood safety. Due to the different interests of the parties, the water boards renounces from multifunctional use.

- The water boards have an inflexible attitude towards multifunctional use. They have insufficient expertise on spatial development and within the water boards there is not enough opportunities provided to explore the field of multifunctional use (Van Peperstraten, 2010).
- Spatial quality is easy to realise in new situations. Maintaining the existing situation may lead to loss of spatial quality because it focuses on adaptation and integration (Klijn et al., 2010). Spatial development may result in many different cross sections, which makes the safety assessment devious.

2.2.3 Technical impediments

Multifunctional use raises new problems that have not been encountered yet. In the context of the safety assessment a couple of impediments are identified (Ellen et al., 2011a):

- For not water retaining objects a permit is necessary. The water board is not eager to grant a permit because it makes the safety assessment more extensive. If the water boards have improved ways for the safety assessment of not water retaining structure, the granting of a permit would be easier.
- New insights in the flood safety may lead to earlier adjustments to flood defences than expected. The consequences of the adaptability of the multifunctional flood defence can limit the possibilities for the second user.
- The safety assessment in the Dutch guideline is not designed for multifunctional use. Provisions have been made to impose additional requirements by means of the advanced assessment, but this is a unique set of requirements for individual projects and costs a lot.
- The most important impediment is that there are uncertainties about some effects which occur with multifunctional use of flood defences, for example: hard elements in a sandy flood defence, smart soils, windmills and vegetation in the foreshore.

2.2.4 Governmental impediments

Before the governmental impediments are identified the tasks of the different governmental bodies are explained. The division of tasks between governmental bodies in the current situation is as follows (Weijers et al., 2009):

- The water board is responsible for the construction, management and maintenance of the primary flood defences.
- The province has two tasks: monitoring the technical quality of the management of the water board and supervising the harmony between municipality and water board. The development and maintenance of the norms for secondary flood defences is also a provincial task.
- The state is responsible for: legislation, supreme control of the system of water boards, the management of water defences that protect various dike ring areas and the management of the large waters and rivers.

- The municipality is responsible for the zoning plans in which flood defences must find a place. In the case of a flooding the municipality is responsible for the emergency plan.

The current government has the plan to reduce the amount of water boards to about 10 to 12 and merge the provinces to 5 country parts. On the long term (2025) the 12 provinces and the 25 water boards will disappear and the tasks will be taken over by 5 country parts. The municipalities have to be combined to municipalities with at least 100,000 residents. The reason for these plans is the fact that the government wants to save money by reducing the amount of political offices (Rutte et al., 2012).

The impediments that are identified are only the ones in relation to the laws, policies and public-private agreements (Ellen et al., 2011a):

- The primary function of a flood defence is to ensure the flood safety. For other kind of use of a flood defence a permit is necessary. The water board must check the multifunctional use of flood defences with the Water Act. The impediment that might occur is that the flood safety prevails over the other kind of use according to the Water Act.
- The municipality is responsible for the zoning plans with the goal of good spatial development. For multifunctional use the area requires two purposes: flood safety and nature or housing for example. Multifunctional use is only possible when the second function is described in the management plan of the water board. During the preparation of the zoning plan the municipality is obliged to consult with the water boards and the provinces. This consult can be used to reach an agreement between the different governmental bodies.
- The state has several policies in which multifunctional use is encouraged or restricted. The national water plan states that large scale measures may lead to excess investments, because the expected climate change might be overestimated. By multifunctional use of a flood defence the excess investments can be compensated by the revenues of the second function. On the other hand, the policy on rivers allows only activities in the floodplains if they are inseparable of the river, except for a few experiments.
- Allowing other functions on flood defences introduces other regulations to be dealt with. For example when allowing houses on flood defences the Housing Act has to be taken into account. The Housing Act prohibits a governmental body to make private agreements that are contrary to public regulations.
- The initiator of multifunctional use needs permission of the owner of a flood defence to build on a flood defence. The owner of a flood defence usually gives permission if the initiator has the responsibility of the damage to a flood defence and its consequences. It is doubtful whether such responsibility for the initiator is desirable.

2.3 Conclusion

Type G is chosen as the most interesting type from the different types of multifunctional use obtained in the first paragraph of this chapter, because it has the largest interaction between the structure and the soil body. The results for type H are probably partially applicable for types

E, F and H as well, because they have a lot of resemblance with the chosen type. As described above, there are impediments in many different disciplines. The water board is often the one that has no confidence in multifunctional use and therefore avoids multifunctional use. This is usually based on trust, risk, liability, cost and benefit aspects and not on technical issues. Those aspects should be researched as well to be able to apply multifunctional use of flood defences on an integral scale for spatial development. The focus of this thesis is on the technical aspects of multifunctional use, in order to demonstrate that multifunctional use is possible and can serve as a basis for the solution to the other impediments. The next chapter describes how flood defences are assessed in general in order to obtain how multifunctional flood defences should be assessed.

3 Design and safety assessment

This chapter elaborates on the design and safety assessment of the multifunctional flood defences. For the design and safety assessment of multifunctional flood defence three different guidelines are stated: the guidelines for dikes, hydraulic structures and buildings. The main guideline for primary flood defences is the 'Voorschrift Toetsen op Veiligheid Primaire Waterkeringen' (VTV, 2006), which is prescribed by the Water Act (Waterwet). This guideline refers to the several other guidelines on specific topics of the design and safety assessment of dikes and hydraulic structures. Beside these Dutch guidelines for the assessment of flood defences, also the European guidelines for buildings (the Eurocodes) are of interest for multifunctional flood defences which are prescribed by the Housing Act (Woningwet) and the Building Regulations (Bouwbesluit). The design and safety assessment of the dikes, hydraulic structures and buildings is elaborated in the next paragraphs.

3.1 Dikes

The different types of multifunctional flood defences that are distinguished have a lot of similarity with traditional flood defences like dikes. The design and safety assessment of the dikes is therefore elaborated in this paragraph. The guidelines are based on the concept of failure mechanisms. Each failure mechanism is assessed in a corresponding assessment track. The difference with multifunctional flood defences is described as the influence on those assessment tracks.

3.1.1 Failure mechanisms and assessment tracks

In the Dutch guidelines (VTV, 2006) the failure mechanisms of an earthen dike are distinguished, see Figure 3-1.

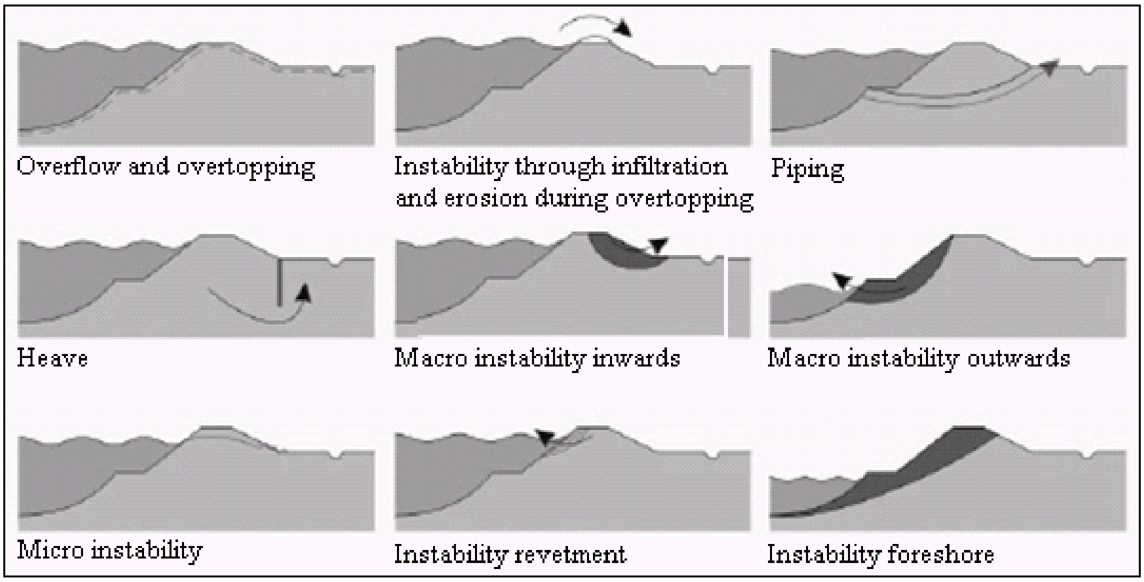

Figure 3-1: Failure mechanisms of dikes (in Dutch) (VTV, 2006)

Overflow and overtopping

The crest height of the dike should be able to withstand the overflow of water of the dike. Overtopping of the dike can cause the revetment on the crest or on the inner slope to fail or create an unmanageable situation. The restriction of the overtopping discharge is in most cases the governing failure mechanism.

Instability due to infiltration and erosion during overtopping

The top layer can become saturated due to infiltration, which leads to a reduction of the effective stress and hence the resistance to shearing. At the same time the volumetric weight of the saturated top layer is large and thereby also the driving force of shearing. This results in cracks perpendicular to the inner slope of the dike. Overtopping can cause erosion of the inner slope. The combination of erosion and infiltration can enhance each other.

Piping

The water level difference over the dike causes water to flow through the dike. When the flow velocity becomes too large, the soil particles are carried along with the flow of water. Due to the transportation of the soil particles, an erosion channel is formed under the dike. Eventually, the erosion channel becomes too large and the dike collapses.

Heave

Due to the vertical flow of water through the soil, the effective stresses in the soil disappear and quicksand can be formed. Heave often occurs behind structures, for example behind seepage barriers.

Macro instability inwards

Macro instability is the sliding of large parts of the dike. The soil properties and the pore pressure determine the resistance against macro instability. A higher level of the outer water results in an increase of the phreatic line inside the dike, hence an increase of the pore pressure. If a water bearing layer is covered with a poorly permeable layer, the water pressure in the water bearing layer lifts up the poorly permeable layer. This effect reduces the resistance against macro instability.

Macro instability outwards

The outward macro instability is caused by the rapid decline of the outer water level after a high water level. The volumetric weight of the soil has become higher due to the high water level. The rapid decline of the outer water level leaves no time for the water to flow out of the soil and the effective weight of the soil becomes higher.

Micro instability

Micro instability is the loss of stability of the top layers on the inner slope as a result of the flow of water through the dike. The flow of water through the dike can cause the wash out of soil particles or the high water pressure under an impermeable layer can lift up that layer.

Instability revetment

The revetment protects the core of the dike from eroding. The revetment may fail due to wave attack, longitudinal flow or static water pressure.

Instability foreshore

If the foreshore is composed of soft clay and peat layers or loosely packed sand, shearing and liquefaction of the foreshore can occur.

Not water retaining structures

The presence of not water retaining structures can have an effect on the resistance against other failure mechanisms or may affect the water retaining capacity of the dike after failure of the not water retaining structures.

In Figure 3-2 the coherence between the failure mechanisms and the assessment tracks is presented.

Faalmechanisme	Deelspoor beoordeling	Katern - §
Overloop en overslag	Hoogte (HT)	Katern 5 - 4.1
Instabiliteit door infiltratie en erosie bij overslag	Bekledingen (STBK)	Katern 8
Piping	Piping en heave (STPH)	Katern 5 - 4.2.2
Heave	Piping en heave (STPH)	Katern 5 - 4.2.2
Macro-instabiliteit binnenwaarts	Macrostabiliteit binnenwaarts (STBI)	Katern 5 - 4.2.3
Macro-instabiliteit buitenwaarts	Macrostabiliteit buitenwaarts (STBU)	Katern 5 - 4.2.4
Micro-instabiliteit	Microstabiliteit (STMI)	Katern 5 - 4.2.5
Instabiliteit van bekleding buitentalud	Bekledingen (STBK)	Katern 8
Instabiliteit van het voorland (afschuiving en zettingsvloeiing)	Voorland (STVL)	Katern 9
Invloed van niet-waterkerende objecten	Niet-waterkerende objecten (NWO)	Katern 10

Figure 3-2: Coherence between failure mechanisms and assessment tracks (in Dutch) (VTV, 2006)

3.1.2 Influence of multifunctional use on the assessment tracks

The structure inside of the dike has an influence on the stability of the dike. A feasibility study on a parking garage inside of the dike in a dune structure in Katwijk resulted in some points of attention but there were no insuperable difficulties (Wessels et al., 2009). The influence on the different assessment tracks is described to discover the points of attention for the application of a large structure inside of dikes.

Height

The height of the dike is determined by the allowable overtopping discharge. The structure itself has no effects on the loading part, but future adjustments on the loading part (e.g. climate change) need to be implemented in the design or the dike has to be adaptable to withstand this change in loading. The requirements on the structure, regarding the loading part, need to be adjusted to the design storm conditions. The structure also has to withstand the loads from a possible elevation of the dike height. The transitions between structural elements and the dike are an important design aspect because they are sensitive to erosion.

Piping and heave

The resistance against piping and heave is less due to the structure inside of the dike, because it is easier to form a small channel on the interface between the concrete and the soil. Also when the structure or the foundation of the structure, is situated in an impervious layer, piping and heave can occur on the borders of the structure. Around locks piping and heave problems have been solved by using seepage barriers to increase the seepage length, the same can be done in this case.

Macro stability inwards

A higher water pressure under the structure lifts up the structure and reduces the friction between the structure and its subsoil; this may cause the structure to move horizontally. Again future adjustments to the design water level have to be taken into account, because a higher water level means a larger lifting force under the structure.

Macro stability outwards

No effects on the macro stability outwards are likely to occur.

Micro stability

If the structure itself forms the inner slope of the dike, micro stability is not able to occur. The structure can lower the phreatic line in the dike and would therefore only have a positive effect on the micro stability.

Revetment

In case of overtopping, the structure reduces the storage capacity of the dike and leads to higher flow velocities on the inner slope.

Foreshore

No effects on the foreshore are likely to occur.

Not water retaining structures

The structure has no effects on other not water retaining structures. The structure, if it is a not water retaining structure itself, obviously has effects on the other assessment tracks. In case that the water retaining function of the flood defence is (partially) fulfilled by the structure, the structure cannot be assessed as a not water retaining structure because failure of the structure inevitably leads to failure of the flood defence.

Table 3-1 presents the influence of the structure on the assessment tracks arising from safety assessment of dikes. No influence holds only that nothing changes in the assessment track, but does not mean it should not be done.

Table 3-1: Influence on the assessment tracks

Assessment track	Influence
Height	Possible
Piping and heave	Possible
Macro stability inwards	Possible
Macro stability outwards	None
Micro stability	None
Revetment	Possible
Foreshore	None
Not water retaining structures	None

3.2 Hydraulic structures

If the flood defence is not a dike but a hydraulic structure the design and safety assessment of the dike is not sufficient enough. The multifunctional design treated in this graduation thesis is somewhere between a dike and a hydraulic structure. The soil body of the dike together with the structure form the flood defence that has to protect the hinterland from flooding. This paragraph describes the guidelines for the hydraulic structures.

The structure inside of the dike has influence on the assessment tracks of the dike. In addition, the structure itself needs to be assessed. In the safety assessment of the dike the assessment track not water retaining structures is defined. This assessment track is not suitable for the assessment of large structures inside of dikes, because the structure may have the water retaining function and failing of the structure likely means that the dike as a whole fails. The safety assessment of water retaining structures offers a solution for the assessment of large structure inside of dikes. The safety assessment consists of the following assessment tracks:

- Height
- Stability of structure and soil body
- Strength of structural elements
- Piping and heave
- Foreshore
- Reliability closure

The piping and heave, height and foreshore assessment tracks coincide with the assessment tracks of dikes and are therefore not explained. The reliability of closure of the water retaining structure is only applicable for moving structures, therefore not applicable in this case. There is no influence of the structure on the assessment tracks that are elaborated in this paragraph because the described assessment tracks already treat the structure. This leaves only the following two assessment tracks to be explained:

Stability of structure and soil body

This assessment track holds the stability of the structure as a whole and the influence on the surrounding soil. The interaction between the structure and the soil plays an important role. The resistance of the structure against movement and deformation has a large influence on the interaction between the structure and soil.

Strength of structural elements

This track assesses the strength of the individual elements of the structure. The soil bodies that are connected to the structure are not considered as structural elements and therefore not assessed. The soil bodies surrounding the elements can influence or pass on loads to the structural elements.

3.3 Buildings

The Eurocodes are European standards for the assessment of the structural safety for all kinds of buildings. For the assessment of a structure inside of the dike the following Eurocodes are of interest:

- NEN-EN 1990 Basis of structural design
- NEN-EN 1991 Actions on structures
- NEN-EN 1992 Design of concrete structures
- NEN-EN 1997 Geotechnical design

NEN-EN 1990 Basis of structural design
This Eurocode defines the principles and requirements for the safety, serviceability and durability of structures. It is based on the concept of the limit state and uses the method of partial factors. In combination with NEN-EN 1991 to 1999 it is intended to use NEN-EN 1990 directly in practice for the design of new structures. The NEN-EN 1990 distinguishes two limit states, the ultimate limit state and the serviceability limit state. The ultimate limit state refers to the safety of people and the safety of the structure, the serviceability limit state refers to the performance of the structure, the comfort of the people and the appearance of the structure. By using design values for the loads and resistance in combination with the partial factors a certain amount of reliability of the safety of the structure can be achieved.

NEN-EN 1991 Actions on structures
The possible loads on the structures are described in separate parts of this Eurocode; the first part consists of the following components:

- 1991-1-1 Densities, self-weight, imposed loads for buildings
- 1991-1-2 Actions on structures exposed to fire
- 1991-1-3 Snow loads
- 1991-1-4 Wind actions
- 1991-1-5 Thermal actions
- 1991-1-6 Actions during execution
- 1991-1-7 Accidental actions

Not all components of the first part are of interest for the assessment of a large structure inside of dikes. In the preliminary design the first part is of importance for the underground structures. Wind and snow load on the structure are not the governing load cases for an underground structure. In case of a large structure on top of the underground structure wind loads become governing. Fire, thermal, execution and accidental actions are not considered in this case but need to be considered for final design of a structure.

NEN-EN 1992 Design of concrete structures
The structure inside of the dike is most likely to be build out of concrete because other materials are less suitable in this case. This Eurocode describes the outline of the design of concrete structures and consists of three parts:

- 1992-1-1 General rules and rules for buildings
- 1992-1-2 General rules - Structural fire design
- 1992-2 Concrete bridges. Design and detailing rules
- 1992-3 Liquid retaining and containing structures

The first part of this Eurocode is a continuation of NEN-EN 1990 specifically on the design of concrete structures. There are various aspects which have an impact on the design of the construction, for example strength properties, load combinations and detailing. All those aspects and how to deal with them are described in this Eurocode. The aspects that should be taken into account vary for different kinds of structures. The function of the structure largely determines the layout of the structure and therefore the aspects that should be taken into account. The ultimate limit state is particularly important for the safety of the dike because the ultimate limit state refers to failure of the structure. The serviceability limit state refers to the possibility to perform the function of the structure. If the structure fails in its function, the flood safety function of the dike as a whole should not be affected.

NEN-EN 1997 Geotechnical design

This code gives the general principles and requirements of the geotechnical design and is intended for the safety, usability and durability of the foundations of structures and needs to be used in combinations with NEN-EN 1990 and NEN-EN 1991. This code is divided into two parts:

- 1997-1 General rules
- 1997-2 Ground investigation and testing

Since the structure is located inside of the dike geotechnical failure also plays a role. Eurocode 7 part 1 describes how to deal with foundations (shallow and on piles), soil retaining structures, geotechnical failure and overall stability. The second part is not of importance for structures inside of dikes. The most import aspect is the overall stability of the structure since this may lead to failure of the flood defence. For geotechnical failure refers the national annex to guidelines to assess geotechnical failure.

3.4 Conclusion

With the introduction of the Eurocode there is a discrepancy between the Eurocode and the guideline for hydraulic structures. The Eurocode redefined the consequence classes and the corresponding reliability classes for the assessment of structures. The highest consequence class of the former Dutch code and the guideline corresponded, see Figure 3-2. The highest consequence class of the Eurocode now proposes stricter requirements on structures.

Table 3-2: Reliability index (50 year reference period) according to the Eurocode and NEN

Reliability class	Reliability index	
	EC	NEN
RC1	3.3	3.2
RC2	3.8	3.4
RC3	4.3	3.6

A structure inside of a flood defence is classified in the highest consequence class because failure of the structure, i.e. the flood defence, would lead to very large consequences. The same structure placed on another location than in a flood defence would in most cases be classified in the middle consequence class. The guideline hydraulic structures states that hydraulic structures need to be designed according to the standards of the highest consequent classes; in that case the structure fulfils the function of a flood defence on its own. In this case the structure is located inside of the dike and together they form the flood defence. The hydraulic loads on the structure are in this case not directly submitted onto the structure but via the soil and groundwater on the structure. Due to the transmission of the loads via the soil and groundwater the variability of the loads becomes less than the variability when the loads are directly applied on the structure. The Eurocode allows hydraulic loads to be schematised as permanent or variable loads depending on the change of the load in size over time. The schematisation as permanent or variable load results into different load factors. In addition, the guideline states load factors for the hydraulic loads on the structure. With the design of the multifunctional flood defence should be carefully looked to the loads and the corresponding partial load factors that are applicable in a specific case.

The guidelines and Eurocodes that have to be used for multifunctional use are briefly described in this chapter and the resulting problems for the assessment are introduced. Chapter four describes how multifunctional flood defences should be dealt with in the risk analysis and what the influence of the introduction of the Eurocode on the risk analysis is. The concepts of the flood defence system and probabilistic design are explained in the first sections of chapter four.

4 Risk analysis

This chapter describes the risk analysis of multifunctional flood defences. The flood defence system and the location for multifunctional flood defences within this system are described in the first paragraph. The failure mechanisms of the flood defence system can be assessed with one of the three levels of probabilistic approach which are presented in the second paragraph. The third paragraph covers the introduction of the Eurocode and the consequence for the probabilistic design of multifunctional flood defences. The partial factors from the level I probabilistic analysis according to the guideline hydraulic structures and the Eurocode are presented in the fourth paragraph.

4.1 Flood defence system

The Netherlands is for a large part situated below sea level and there are rivers that discharge water towards the sea. To protect the land against flooding from either the sea or rivers the land is divided into different areas that are surrounded by flood defences. Flood defences come in all shapes and sizes; from the more natural flood defences like dunes and dikes up to sluices and storm surge barriers. A single area surrounded by flood defences is called a dike ring area which protects an area that represents a certain value. The damage in case of a flooding consists of casualties, material, economical and immaterial damage. Risk is the damage multiplied by the probability of occurrence of that damage. The risk is divided into three types: individual risk, group risk and economical risk. Individual risk is the probability of death for a person on a specific location, the group risk is the probability of casualties during a flooding and the economic risk is the probability of the direct and indirect costs as a result of a flooding. The acceptable amount of risk for each dike ring area is a political decision and is a balance between the investment costs against the risk of a flooding. The acceptable amount of risk is translated to four levels of probability of failure of a dike ring area per year: 1/1250, 1/2000, 1/4000 and 1/10000 (HR, 2006). To each dike ring area a certain probability of failure is assigned related to the expected amount of damage. The dike ring area is subdivided into different components of the same type. Each component is again subdivided in to sections; each section has more or less the same characteristics or a governing profile. The possible failure mechanisms are presented for each section. In Figure 4-1 presents the flood defence system for an arbitrary dike ring area.

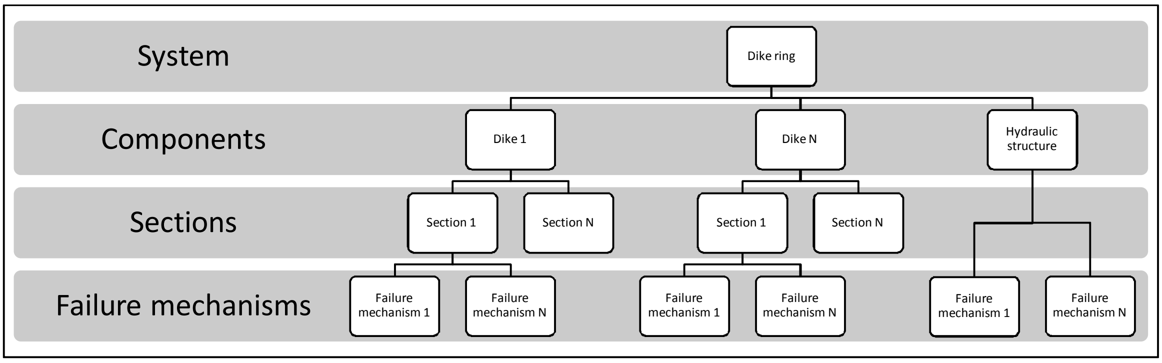

Figure 4-1: Flood defence system of a dike ring area

The approach for the design of flood defences and the verification is different. The probability of failure of a dike ring is translated to design water levels with an exceedance probability equal to the failure probability for the design of flood defences. All the failure mechanisms are than designed in such a way that they can withstand the force resulting from the design water level. The result of each failure mechanism is than 'meets requirements' or 'does not meet requirements', when all the failure mechanisms meet the requirements the dike ring area is designed correct. For hydraulic structures the design water levels applied are on the geotechnical failure mechanisms, but there are also other failure mechanisms involved for hydraulic structures. The other failure mechanisms have a limited failure space. The failure space of the failure mechanism 'reliability closure' is 10% of the total failure space and for structural strength and stability is only 1% of the total failure space available (TAW, 2003). The assignment of the limited failure space is done because failure can lead to a sudden strong worsening and uncontrollable situation where very large volumes flow inward, leading to severe flooding of the hinterland.

The failure probability is directly translated to requirements on the failure mechanisms for the design of flood defences. The verification of the dike ring areas works the other way around. Probabilistic calculations on the failure mechanisms result into the failure probability of a section, the failure probabilities of the sections result into the failure probability of a component and the failure probabilities of the components lead to the failure probability of the flood defence system, i.e. dike ring area. The probabilistic calculations of the failure probabilities in a series system include the correlation between the individual failure mechanisms. The failure probability of a system is the sum of all failure probabilities of the sub systems in case of uncorrelated subsystems. In case of fully correlated subsystems the probability of failure of the system becomes equal to the largest failure probability of the subsystems. The difference in failure probability between a fully correlated and an uncorrelated system is quite large. The correlation between failure mechanisms is mainly caused by the load. All failure mechanisms have more or less the same load, but the difference is the influence of the load on the failure mechanism itself. The length effect also plays a role in the flood defence system, another correlation effect. A longer flood defence increases the probability of failure because of the spatial variation of the resistance of the flood defence. The flood defence system consists out of components and sections, the more components and sections there are, the more possibilities there are for the flood defence system to fail.

Multifunctional use with a structure inside of a dike complicates the safety assessment of a flood defence system. The structure inside of the flood defence introduces more failure mechanisms to be dealt with in one section than for a hydraulic structure or traditional dike. Six failure mechanisms remain from the safety assessment of a traditional dike. The structure introduces failure mechanisms on the total stability of the structure and the strength of the elements of which the structure is composed. The flood defence system for this type of multifunctional flood defence is presented in Figure 4-2. The structure also introduces an additional remaining strength of the flood defence. The remaining strength of a flood defence is the possibility of a flood defence to withstand the water level after one of the failure mechanisms has occurred. This is an important topic in the project VNK (Veiligheid Nederland in Kaart). The remaining strength is of major importance in order to have a correct estimate of the probability of a flooding. The same holds the other way around. The elements of a hydraulic structure, like a sluice, experience the forces directly from the water. But with the dike enveloping the structure the forces of the water are passed on to the structure via the ground water, which reduces the forces on the structure. The enveloping dike also protects a structural element in case of failure of this element, again increasing the remaining strength of the flood defence.

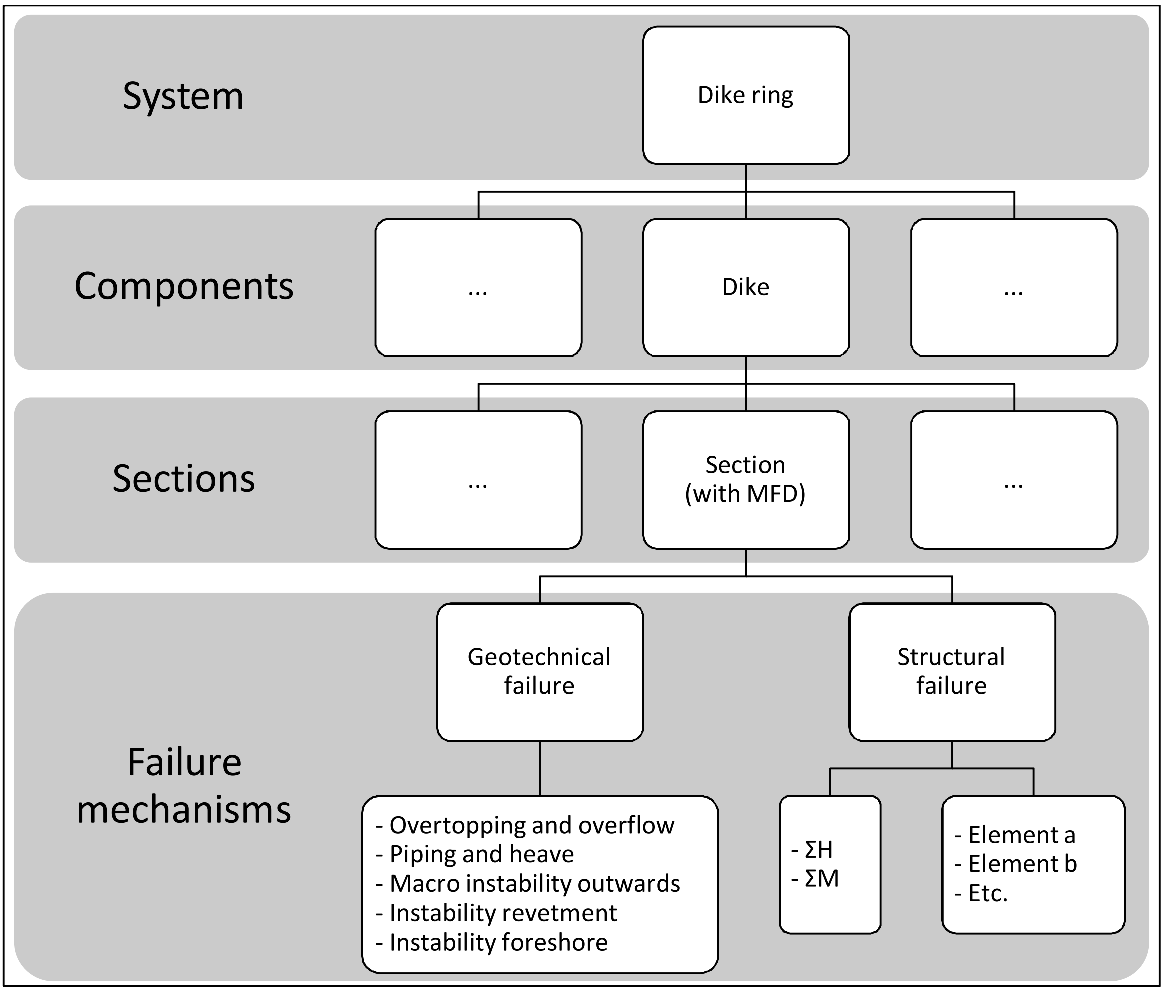

Figure 4-2: Flood defence system of dike ring area including a multifunctional flood defence

The remaining strength of the flood defence in case of failure of a particular element is hard to describe. The remaining strength is therefore not included in the design process but it is good to keep it in mind. In case of failure of an element in a sluice door, the remaining strength is zero because it will immediately lead to a flooding of the hinterland. This is translated to a more severe requirement on the structural failure; 1/100 of the failure probability of the dike ring area. This also holds for the overall stability of the hydraulic structure, because this also immediately leads to flooding of the hinterland. The same requirement is adopted for multifunctional flood defences, but keeping in mind that there is still a hidden reliability in the possibility that the flood defence will not fail if a single element fails. In addition, failure of a structural element that is caused by loading other than the water pressure is not likely to cause a flooding because having both an extreme water pressure as well as another extreme load is even rarer than the separated events.

The future approach of the flood defence is going to change. The present approach which uses the norm frequency of the dike ring area to obtain design water levels with an exceedance probability equal to the norm frequency is outdated. The future approach is going to use failure probabilities assigned to dike ring sections as the requirement for the flood defences. This way the length of the flood defences is much better incorporated in the safety assessment of the flood defences.

4.2 Probabilistic design

For the design can be chosen for a semi probabilistic approach or a fully probabilistic approach. A semi probabilistic approach is based on experience and the reliability of the structure is not known. The reliability is a very important aspect of the design of flood defences, is therefore always a probabilistic approach used. The probabilistic approach is divided into three levels; a high level holds a more complicated calculation but it gives a more accurate result. In most cases the first level is used because the improvement for the use of a higher level of probabilistic approach does not deliver the desired reduction in costs. In other words, the costs to do a comprehensive probabilistic calculation are higher than the reduction in costs that results from the probabilistic calculation. A short description of the three distinguished levels:

4.2.1 Level I

The first level is not a fully probabilistic approach but a semi probabilistic approach. In the semi probabilistic approach partial factors are used on the representative values for the load and strength to incorporate safety into the design. The representative value of the load is usually the 5% exceedance probability value and the representative value of the resistance is usually the 95% exceedance probability value. The partial factors for the load and strength depend on the reliability class and are stated in the guidelines and the Eurocode. A visual impression of the concept of partial factors and representative values is presented in Figure 4-3.

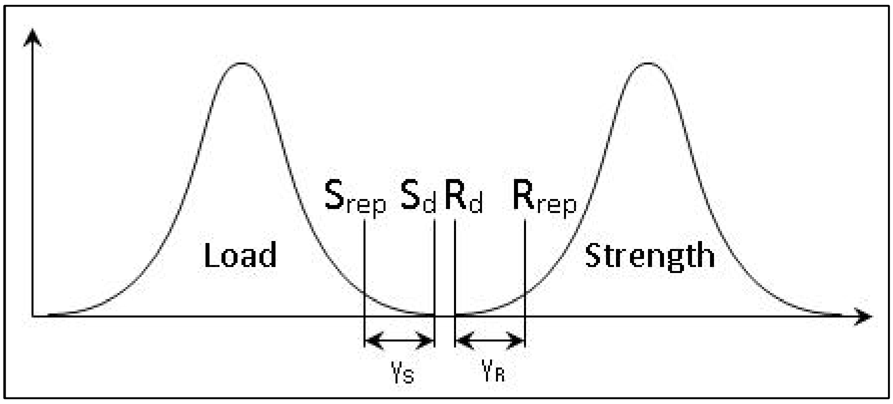

Figure 4-3: Partial factors for the semi probabilistic approach

The values of the partial factors are determined in such a way that for each case at least a certain amount of safety is introduced into the calculations. For a generally applicable approach the value of the partial factors are standardized so that for every situation the same partial factors can be used. So the use of partial factors is a more conservative way than a fully probabilistic approach. The calculation uses a performance function being the resistance minus the load. When the performance function is lower than 1, i.e. the load is higher than the resistance, it fails. In formulas:

$Z(X_1, \dots, X_n) = R - S \qquad Z < 0 \rightarrow \text{failure} \qquad Z > 0 \rightarrow \text{no failure}$

Where:

Z performance function
R resistance (strength)
S solicitation (load)

4.2.2 Level II

With the second level approach the failure probability and the reliability can be calculated by the use of a First Order Reliability Method (FORM). The reliability is expressed in the reliability index which is related to the failure probability:

$$\beta=\frac{\mu_z}{\sigma_z} \quad \rightarrow \quad P_f=\Phi(-\beta)$$

Where:

β reliability index
P_f failure probability
μ_z mean value of the Z function
σ_z standard deviation of the Z function
$\Phi(..)$ standard normal distribution

The mean value of a non-linear Z function can be approximated by the first two terms of the Taylor-polynomial of the Z function, which is normally distributed according to the central limit theorem:

$$\mu_z = Z(X_1, \ \dots \ ,X_n) + \sum_{i=1}^{n}\left(\frac{\partial Z(X_1, \ \dots \ ,X_n)}{\partial X_i}\cdot(\mu_{X_i} - X_i)\right)$$

The same holds for the standard deviation:

$$\sigma_z = \sqrt{\sum_{i=1}^{n}\left(\frac{\partial Z(X_1, \ \dots \ ,X_n)}{\partial X_i}\cdot\sigma_{x_i}\right)^2}$$

The point at which the reliability index is found is called the design point and is obtained after an iterative process:

$$X_1^* = \mu_i + \alpha_i \cdot \beta \cdot \sigma_i$$

In which:

$$\alpha_i = -\frac{\dfrac{\partial Z(X_1^*, \ \dots \ ,X_n^*)}{\partial X_i}\cdot\sigma_{x_i}}{\sqrt{\sum_{i=1}^{n}\left(\dfrac{\partial Z(X_1^*, \ \dots \ ,X_1^*)}{\partial X_i}\cdot\sigma_{x_i}\right)^2}}$$

4.2.3 Level III

The level III approach is a fully probabilistic approach and is the most accurate level of the three approaches. The Monte Carlo simulation is the most common method used for this approach. During the Monte Carlo simulation the performance function is calculated many times by drawing values for the parameters according to their distribution. By repeating this process many times all combinations of the parameters are drawn. The number of times the performance function fails over the total number of times the calculation is done is the failure probability:

$$P_f = \text{Prob}(Z < 0) = \frac{n_{failure}}{n_{total}}$$

4.3 Introduction of the Eurocode

The Eurocode is included in the Dutch building code in 2012 and replaced the Dutch NEN codes. The introduction of the Eurocode changed the approach for the design of flood defence. Before the introduction of the Eurocode, the Dutch guidelines for flood defence were prevailing over the Dutch building code. This is best shown by the reliability indexes which are presented in as well the former NEN as in the Eurocodes, see Table 4-1. Especially for the highest class, the reliability index changed the most.

Table 4-1: Reliability index (50 year reference period) according to the Eurocode and NEN

Reliability class	Reliability index	
	EC	NEN
RC1	3.3	3.2
RC2	3.8	3.4
RC3	4.3	3.6

The guideline for hydraulic structures (TAW, 2003) presents a formula to translate the used exceedance probability to a reliability index:

$$\beta = -\Phi^{-1}(\xi \cdot f_n \cdot \text{norm}) = -\Phi^{-1}(0.01 \cdot 10 \cdot \frac{1}{1250}) \approx 3.8$$

Where:

ξ	failure space	$[-]$
f_n	time factor	[years]
norm	norm frequency	[1 / year]
Φ^{-1}	inverse normal distribution	$[-]$

So it can be concluded that the design of a hydraulic structure, based on the guideline for hydraulic structures also fulfils the less severe requirement of the NEN guideline on the reliability of the structure. But with the introduction of the Eurocode also the required reliability classes changed. For hydraulic structures a reliability index is required which results in a much higher norm frequency than is required according to the guideline for hydraulic structures.

$$\beta = 4.3 \quad \rightarrow \quad \text{norm} = \frac{\Phi(4.3)}{0.01 \cdot 10} = 8.5 \cdot 10^{-5} = \frac{1}{11710}$$

The norm resulting from the higher reliability index from the Eurocode is higher than the norms stated in the Dutch guidelines for flood defence. So in all cases (1/1250, 1/2000, 1/4000, 1/10000) the Eurocode prevails over the Dutch guidelines.

4.4 Partial load factors

4.4.1 Eurocode

The partial factors that have to be used are divided in different groups in the Eurocode and presented in Table 4-2. The partial factors of group A need to be used for the overall stability of structures, group B for the strength of structural elements and group C for foundations.

Table 4-2: Partial load factors from the Eurocode

Group	Permanent load		Variable load
	Unfavourable	Favourable	
A	1.1	0.9	1.5
B	1.2	0.9	1.5
C	1.0	1.0	1.3

The differentiation of the partial load factor over the reliability classes is done by the use of the reliability differentiation factor K_{FI}. Multiplying the partial load factor by the factors (presented in Table 4-3), results in the partial load factors presented in Table 4-4.

Table 4-3: Reliability differentiation factors

	RC1	RC2	RC3
K_{FI}	0.9	1.0	1.1

Table 4-4: Partial load factors differentiated over the reliability classes

Group	Reliability class	Permanent load		Variable load
		Unfavourable	Favourable	
A	RC1	1.0	0.9	1.35
	RC2	1.1		1.5
	RC3	1.2		1.65
B	RC1	1.1	0.9	1.35
	RC2	1.2		1.5
	RC3	1.3		1.65
C	RC1	1.0	1.0	1.2
	RC2			1.3
	RC3			1.45

The partial factors are for a design lifetime of 50 years but hydraulic structures have to be designed with a lifetime of 100 years. The different design lifetime can be taken into account with an increase of the representative value of the load, with the following formula:

$$F_t = F_{t_0} \cdot \left(1 + \frac{1 - \psi_0}{9} \cdot \ln\left(\frac{t}{t_0} \right) \right)$$

Where:

F_t value for the design lifetime
F_{t_0} value for the basic reference period
ψ_0 factor for combinationvalue of variable load
t design lifetime
t_0 basic reference period

The increase of the load in this case is:

$$\Delta = 1 + \frac{1-\psi_0}{9} \cdot \ln\left(\frac{100}{50}\right)$$
$$\psi_0 = 0 \quad \rightarrow \quad \Delta \quad = 1.077$$
$$\psi_0 = 1 \quad \rightarrow \quad \Delta \quad = 1.00$$

So the maximum increase of the load value is approximately 8%. This increase in load is only applicable to loads which are specifically for a design lifetime of 50 years instead of the design lifetime of hydraulic structures of 100 years.

The Eurocode 0 states in A1.3.1 that:

(3) The static equilibrium of building structures should be assessed using the design values of the loads according to group A

(5) Design of structural elements (shallow foundations, piles, basement walls, etc.) with geotechnical loads and the resistance of the soil involved should be assessed with the use of one of the following three approaches for geotechnical loads and resistances complemented by EN 1997:
- Approach 1: ...
- Approach 2: ...
- Approach 3: Applying the design values from group C for the geotechnical loads and the simultaneously application of partial factors from group B for other loads on / from the construction.

NOTE: The choice for the use of approach 1, 2, or 3 is made in the national annex.

The national annex states the following:

(5) For the assessment of geotechnical structures design approach 3 must be used.
Group B should be used for all types of loads on:
- Foundations on steel, concrete verification,
- On steel foundations, soil bearing capacity verification,
- Pile foundation, time plus normal force,
- Pile foundation, soil bearing capacity and
- Underground roof / wall constructions.

Group C should be used for geotechnical loads on:
- Overall stability of the foundation,
- Slope stability and
- Piling calculation.

As mentioned before the use of partial load factors is a conservative and relatively easy way to design and assess hydraulic structures. A fully probabilistic approach is more difficult and more time consuming. The approach with partial factors is often used in practice for the assessment of structures. The difficulty in this case is the use of a partial factor for hydraulic loads on the structure. A too large partial factor over the hydraulic load may lead to unrealistic and even impossible loads on a structure. The Eurocode is not completely clear about the partial load factor. In Eurocode 0 in article 4.1.1 (NEN-EN 1990, 2011) following statement is made:

(3) Loads due to water (pressure) may be considered as permanent and/or variable loads depending on their variation of magnitude in time.

So water pressures resulting from little varying groundwater levels can be interpreted as permanent loads. Groundwater levels inside of dikes depend on the water level of the adjacent water body. The water levels in rivers are varying in time and should be interpreted as variable loading according to the previous statement. But with the partial factors for variable loads the schematised water level will be unrealistic.

Eurocode 7 in article 2.4.4 (NEN-EN 1997, 2012) states (in contradiction to Eurocode 0) the following about water levels:

(1)P The surface level, slope, water levels, levels of layer separation, excavation levels and the dimensions of the geotechnical structure must be considered as geometrical data.

The design value of geometrical data is not obtained with partial factor but with an additional water height:

$$a_d = a_{nom} \pm \Delta a$$

Where:

a_d design value of geometrical data
a_{nom} nominal value of geometrical data
Δa addition or reduction to obtain the design value

Eurocode 7 also states in 2.4.6.1 that:

(7) In some cases, extreme water pressures, in accordance with 1.5.3.5 of EN1990, may be interpreted as exceptional loads.

It can be concluded that the Eurocode gives a couple of possibilities to handle hydraulic pressures. The Eurocode also states that apart from the rules stated, engineering judgement plays a role in the evaluation of water pressures. The next paragraph elaborates how the water pressures are handled in the Dutch guideline for hydraulic structures.

4.4.2 Guideline hydraulic structures

The Housing Act and the Building regulations prescribe the use of the Eurocode for buildings. For flood defences the Water Act is also applicable which refers to the guideline hydraulic structures (Leidraad Kunstwerken). The partial factor for hydraulic loads is equal to 1.25 and applicable for all exceedance probabilities and reliability classes. The introduction of the Eurocode has led to stricter reliability classes as is presented in the previous paragraph. The partial factor is in case of the Eurocode not applicable anymore. This guideline also states a formula for the partial factor for hydraulic loading on structures:

$$\gamma_H = \frac{u - B \cdot \log\left(\Phi\left(\alpha_s \cdot \beta_N\right) \cdot f_N\right)}{u - B \cdot \log\left(norm\right)} \cdot \exp\left(\alpha_R \cdot \left(\beta_N - 3.6\right) \cdot \sqrt{\ln\left(1 + V_R^2\right)}\right)$$

Where:

γ_H	partial coefficient for hydraulic loading	[–]
u	water head difference with an exceedance frequency of 0.63 per year	[m]
B	decimation height	[m]
β_N	reliability index for reference period N	[–]
f_N	influence factor for the reference period N	[year]
norm	exceedance frequency	[1 / year]
$\Phi(..)$	standard normal distribution	[–]
α_S	influence factor for uncertainty in load	[–]
α_R	influence factor for uncertainty in strength	[–]
V_R	coefficient of variation of the strength	[–]

The reliability index from the Eurocode together with the exceedance probability of the water level determines the value of the partial factor. The reliability indexes for the water level corresponding to the exceedance probability following from the guideline are stated in Table 4-5 and can be calculated with the following formula:

$$\beta = -\Phi^{-1}\left(\xi \cdot f_n \cdot norm\right)$$

Where:

β reliability index
ξ failure space factor
f_n lifetime factor
norm norm frequency
Φ^{-1} inverse normal distribution

The failure space factor assigned to structural failure of the flood defence as is stated in the guideline is equal to 0.01. The lifetime factor is limited to 10 years because of the correlation between the probabilities of failure in individual years during lifetime. In case of long structures (along the direction of the dike), length effects play also a roll. The reliability index should be increased by 10% for structures longer than 100 m.

Table 4-5: Reliability index (for the lifetime) by the corresponding exceedance probabilities

Norm frequency	β	β^*
1/1,250	3.78	4.15
1/2,000	3.89	4.28
1/4,000	4.06	4.46
1/10,000	4.26	4.69
* including 10% length effect		

The Eurocode states reliability indices for the consequent classes defined in the Eurocode. The failure of the structure can result in failure of the flood defence hence very large consequences. The applied consequence class should be the highest; RC3. The former NEN also used different reliability classes with corresponding reliability indices. The NEN is replaced by the Eurocode and the reliability indices increased with the introduction of the Eurocode. The reliability indices according to the Eurocode and NEN are presented in Table 4-6.

Table 4-6: Reliability index (for different reference periods) according to the Eurocode and NEN

Reliability class	Eurocode			NEN		
	β_1	β_{50}	β_{100}	β_1	β_{50}	β_{100}
RC1	4.2	3.3	3.0	4.2	3.2	3.0
RC2	4.7	3.8	3.6	4.4	3.4	3.2
RC3	5.2	4.3	4.2	4.5	3.6	3.4
β_* where * = reference period						

So for the calculation of the partial factor for hydraulic loading the input of the reliability index changed with the introduction of the Eurocode. The maximum value of the partial factor can be calculated by rewriting the formula to:

$$\gamma_H = \frac{\frac{u}{B} - \log\left(\Phi\left(\alpha_s \cdot \beta_N\right) \cdot f_N\right)}{\frac{u}{B} - \log\left(\text{norm}\right)} \cdot \exp\left(\alpha_R \cdot \left(\beta_N - 3.6\right) \cdot \sqrt{\ln\left(1 + V_R^2\right)}\right)$$

The partial factor becomes larger for a smaller value of u/B but it is limited to 1 for flood defences. The partial factors are calculated and presented in Table 4-7 with the following values of the parameters:

$u/B \quad = 1$

$\beta \quad = \beta_{100}$ (NEN and Eurocode) $= \beta$ and β^* (guideline)

$f_N \quad = 100$ (NEN and Eurocode) $= 10$ (guideline)

$\alpha_S \quad = -0.7$

$\alpha_R \quad = 0.8$

$V_R \quad = 0.2$

Table 4-7: Partial factors for hydraulic loading

Exceedance probability [1/year]	NEN			Eurocode			Guideline	
	RC1	RC2	RC3	RC1	RC2	RC3	β	β^*
1/1,250	1.16	1.20	1.24	1.16	<u>1.28</u>	<u>1.55</u>	1.10	1.26
1/2,000	1.10	1.14	1.18	1.10	1.22	<u>1.48</u>	1.09	1.26
1/4,000	1.03	1.07	1.10	1.03	1.14	<u>1.38</u>	1.09	1.26
1/10,000	1.00[1]	1.00[1]	1.01	1.00[1]	1.05	<u>1.27</u>	1.08	1.26

* including 10% length effect
[1] minimum value of 1.00 required

From Table 4-7 can be concluded that in the previous case with the NEN and the guideline the value of 1.25 for the partial factor was satisfying, but with the introduction of the Eurocode the value is not satisfying anymore (see underlined numbers). The largest partial factor is now 1.55, using this value will result in unnecessary large reliability because it is even larger than the partial factor for variable loads. The formula seems not applicable anymore with the introduction of the Eurocode. Another remark on this formula is that the water pressure is not proportional to the water head but to the square of the water head. The moment force generated by the water pressure is even proportional to the third power of the water head. Applying the same partial factor on the different forces will result into different applied water heads for different calculations, which seems incorrect in principle. This was also the case with the former Dutch guideline but this should be corrected as well.

4.5 Conclusion

There is an increasing use of probabilistic analyses for the design and assessment of the flood defence system, where also multifunctional flood defences are part of. A structure inside of the dike leads to a slightly different assessment than for dikes or hydraulic structures. The guidelines for flood defence were prevailing over the building codes before the introduction of the Eurocode. The Eurocode redefined the reliability classes leading to more severe requirement than from the guidelines for flood defences. The Eurocode as well as the guidelines presented partial factors to be used in the semi probabilistic approach. The use of partial factors on forces resulting from a water level with a certain exceedance probability introduces twice an amount of reliability into the calculations, probably over dimensioning the flood defence. In order to be able to find the difficulties with the design of multifunctional flood defences, a case study is performed in the next chapter with the purpose to identify the attention points.

5 Case study Grebbedijk

To gain more insight in multifunctional use of flood defences is chosen for a case study. There is searched for a suitable location in the Netherlands. The location has to meet a number of things, like it has to be a primary flood defence and there must be a demand for spatial development and flood safety. In appendix B a couple locations are mentioned and the reasoning for the choice of the Grebbedijk is presented.

5.1 Introduction

The Grebbedijk is the connection between the Utrechtse Heuvelrug and the Veluwe, both higher grounds. The gap between the two higher grounds is closed off by the 5.5 km long Grebbedijk. The Grebbedijk is part of dike ring area 45 that stretches up north all the way to Bunschoten-Spakenburg. The Grebbedijk is the most important part of the dike ring because a breach will affect the whole dike ring area due to the descending hinterland.

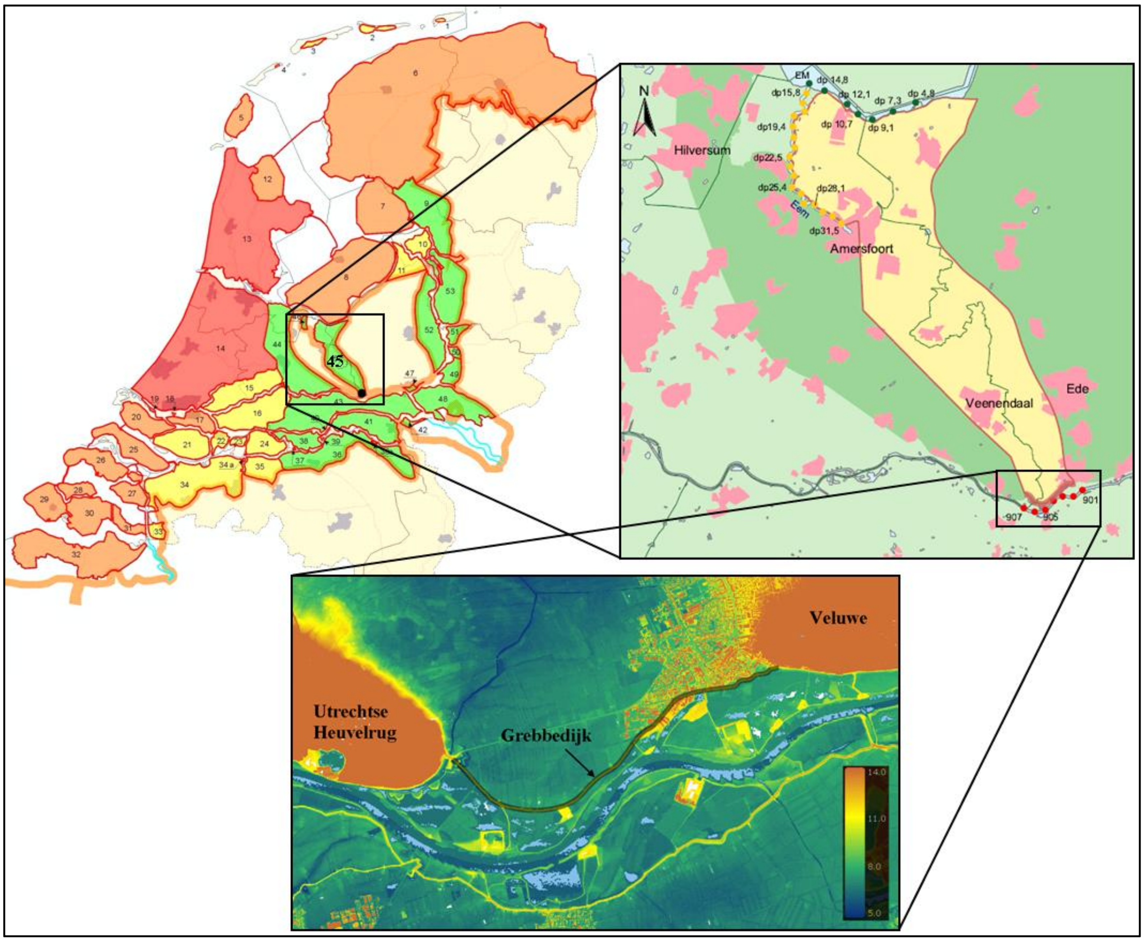

Figure 5-1: Dike ring area 45

The Grebbedijk is also taken into consideration for an upgrade to delta dike. A delta dike is a dike with higher safety demands than a traditional dike and has a larger space occupation than a traditional dike. The larger space occupation is complex in cultivated areas because of the density of buildings, houses and infrastructure. A delta dike requires an integrated approach in cultivated areas and the combination with other functions seems inevitable because of the large space occupation. The eastern part of the Grebbedijk is situated in a cultivated area and is excellent for a multifunctional delta dike.

5.1.1 Changing the probability of failure

There are many words (see Table 5-1) for the idea of a dike that is so strong that it can survive every storm condition. Theoretically it is not possible that the probability of failure becomes zero. But if the probability becomes an order 10 to 100 lower than the required probability, the probability becomes so small that the flood defence can be described as an 'unfailable' dike.

Table 5-1: Dutch and English words for the same idea

Dutch	English
Klimaatdijk	Climate dike
Superdijk	Super dike
Deltadijk	Delta dike
Doorbraakvrije dijk	Unbreachable dike
Onbezwijkbare dijk	Unfailable dike

The Grebbedijk is the ideal candidate for an 'unfailable' dike because the Grebbedijk has a relatively small length and protects a relatively large area. In the area it protects live 250,000 people that are affected by a flooding of the Grebbedijk and an estimated economic loss of 10 billion euro due to a flooding (Wijnacker, 2013).

The choice for the probability of failure is a political and economic choice. The political aspect is the acceptable amount of damage and loss of life, which is hard to quantify. The economical aspect is the efficiency of the investment costs versus the risk costs. The costs can be optimised by comparing the extra costs of dike heightening and the reduction of costs due to a lower probability of a flooding. It is very hard to define the costs accurately, especially the risk costs. The present probability of flooding for dike ring area 45 is 1/1,250 per year. With the costs of a breach known the costs for the risk of a flooding can be calculated with the help of the following formula:

$$C = \sum_{n=1}^{N} \frac{P_f \cdot S}{(1+r')^n} \quad \rightarrow \quad \lim_{n\to\infty} \quad C = \frac{P_f \cdot S}{r'}$$

Where:

C	present value of the risk	[€]
N	design lifetime	[years]
P_f	probability of flooding	[1 / year]
S	estimated economical damages	[€]
r'	actual rate of interest	[–]

With:

$N = 100 \approx \infty$ [years]

$S = 10 \cdot 10^9$ [€]

$r' = 0.05$ [–]

This results in:

$$\left.\begin{array}{llll} P_f = 1/1{,}250 & [1/\text{year}] & \rightarrow & C = 160\cdot10^6 \quad [€] \\ P_f = 1/10{,}000 & [1/\text{year}] & \rightarrow & C = 20\cdot10^6 \quad [€] \end{array}\right\} \rightarrow \Delta C = 140\cdot10^6 \quad [€]$$

So the choice for a lower probability results in a decrease of € 140 million for risk costs over the lifetime of the flood defence. The investment costs to realise this lower probability need to be lower than the decrease in risk costs. The sum of the investment costs and the risk costs depending on the probability can be optimised to a minimum. This principle is presented in Figure 5-2.

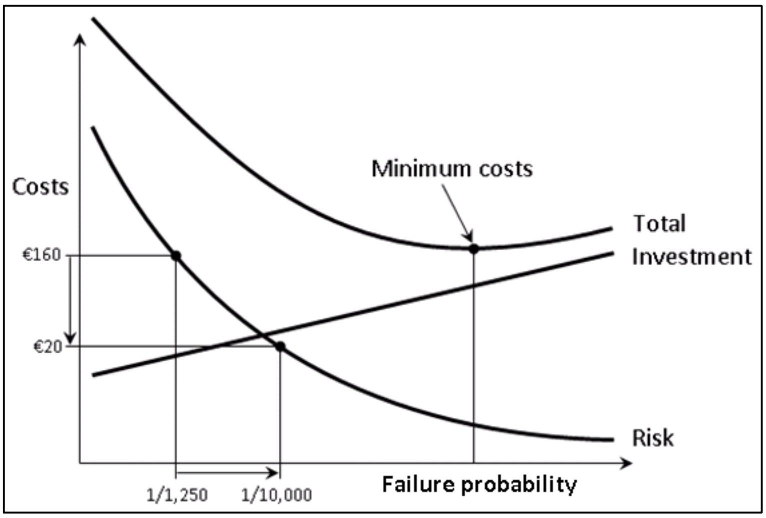

Figure 5-2: Costs versus failure probability of a flood defence system

The decrease of costs of € 140 million results in an available amount of € 25.9 million per km for the investment costs for the 5.4 km long Grebbedijk. Considering that dike reinforcement costs between € 1 and € 5 million per kilometre, it seems feasible to lower the probability. Of course this should be differentiated over different dike selections, because the dike is not continuously over its length. This does not mean that the multifunctional flood defence for the Grebbedijk is a feasible design; this only holds that there should be invested in the Grebbedijk because of the very large consequences of a breach of the dike and the relative short length of the Grebbedijk. This shows that there is a demand for flood safety at the Grebbedijk. For the feasibility of multifunctional flood defences is earlier stated that it requires a demand for flood safety as well as a demand for spatial development. The demand for flood safety is demonstrated with the possible benefit due to risk costs reduction. The new design for the Grebbedijk is based on a probability of flooding of 1/10,000 per year.

5.1.2 Boundary conditions

As starting point for the design of the flood defence the boundary conditions need to be stated. In this case the design water levels and the soil profile are analysed and formulated.

Design water levels

The current design water levels are stated in Table 5-2 and are based on an exceedance probability of 1/1,250 per year. These are the design water levels for on the centre line of the river and are stated for every kilometre. The water board also states the design water level, but in this case they are appointed to a dike section, see Figure 5-4.

Table 5-2: Design water levels for the Lower Rhine (HR, 2006)

Location	Description	Design water level [m+NAP]
901	Wageningen	11.7
902		11.6
903		11.5
904		11.5
905		11.4
906		11.4
907	Grebbeberg	11.3

The Grebbedijk is often named as candidate for an unfailable dike and so the water board Vallei en Veluwe has already stated the design water levels for an unfailable dike. The water board has determined the water levels with an exceedance probability of 1/100,000 per year for an unfailable dike. The multifunctional design for the Grebbedijk is based on an exceedance probability of 1/10,000 per year. The corresponding design water levels can be calculated because the design water level is Gumbel distributed. With the design water levels along the Grebbedijk with an exceedance probability of 1/1,250 and 1/100,000 per year, the design water levels with an exceedance probability of 1/10,000 are calculated. The results of that calculation are shown in Table 5-3.

Table 5-3: Design water levels for the Grebbedijk (WVE, 2012)

Dike section [hm]	Design water level 1/1,250 [m+NAP]	1/10,000 [m+NAP]	1/100,000 [m+NAP]
0 - 4	11.72	12.15	12.62
4 - 8	11.67	12.10	12.57
8 - 11	11.67	12.10	12.57
11 - 13	11.67	12.00	12.37
13 - 22	11.55	11.94	12.37
22 - 31	11.55	11.84	12.17
31 - 41	11.55	11.84	12.17
41 - 45	11.44	11.79	12.17
45 - 52.5	11.28	11.61	11.97
52.5 - 53.7	11.28	11.61	11.97

Soil information

To get some insight in the composition of the soil structure of the Grebbedijk the DINOloket is consulted. The bores and probes found are presented and analysed in appendix B.3.2. The conclusion from the analysis is that the surrounding area is based on a sandy soil structure, with some layers of clay. In Figure 5-3 the assumed composition of the soil structure is shown.

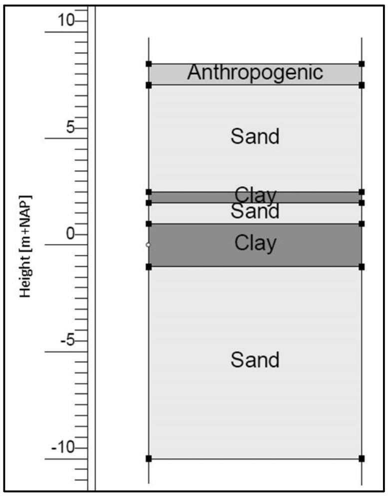

Figure 5-3: Composition of the soil structure of the Grebbedijk

5.1.3 Requirements and assumptions

In addition to the boundary conditions there are some requirements and assumption stated for the design of the Grebbedijk. The following requirements are taken into account:

- The probability of failure of the flood defence is 1/10,000 per year.
- The design lifetime of the flood defence is 100 years.
- Gradient of the inner as well as the outer slopes must be at least 1:3.
- The allowable overtopping discharge is 0.1 l/m/s.
- The soil layer on the structure has to be at least 1 m because of ecological reasons.
- The highest consequence class of the Eurocode should be applied on the structure.

5.2 Area and stakeholder analysis

In order to understand the Grebbedijk and its surroundings better, an area analysis and a stakeholder analysis are conducted. The area analysis investigates the possible locations for a multifunctional flood defence and the functions of the surrounding areas. The stakeholder analysis investigates the parties that are involved and influenced by the design of a multifunctional flood defence.

5.2.1 Area analysis

As is mentioned in the introduction to the Grebbedijk, the Grebbedijk is the connection between two higher grounds (Utrechtse Heuvelrug and Veluwe). The area surrounding the Grebbedijk is divided into four different parts, each of which has different characteristics. The characteristics of the areas determine the functions of the proposed multifunctional flood defence and hence also the layout of the flood defence. More information on the surrounding area of the Grebbedijk can be found in appendix B.2.

Figure 5-4: Area functions around the Grebbedijk

The area around the Grebbedijk is divided into four parts (from east to west in Figure 5-4).

- Residential area

The residential area is the most western part and connects to the higher grounds of the Veluwe. From east to west there are three different hinterlands distinguishable, a new build neighbourhood Rustenburcht, the stronghold of the old city centre and an old neighbourhood. This part of the Grebbedijk is the only part of the Grebbedijk that has no main road on top of the dike, only a cycling path is located on top of the dike.

- Industrial area

The industrial area is located around the harbour of Wageningen (the Rijnhaven) and is mainly used for bulk cargo. On the land side of the Grebbedijk business parks are located that are not directly related to the Rijnhaven.

- Marina area

The marina has 159 berths and is almost entirely surrounded by industrial area. It forms an obstacle for larger wildlife to pass from the Utrechtse Heuvelrug to the Veluwe. Wildlife passes from the Utrechtse Heuvelrug eastward through the flood plains of the Nederrijn but then encounters the marina. If the marina is relocated, large wildlife is able to swim across the entrance of the Rijnhaven and pass through the flood plains of the Nederrijn further to the Veluwe.

- Rural area

Most of the surrounding area of the Grebbedijk is tagged as rural area. On the river side of the Grebbedijk it is mainly natural en recreational area and on the land side it is mostly used for agriculture.

The design of the multifunctional flood defence does not have to follow the alignment of the Grebbedijk. In the area analysis, several other possible alignments are mentioned, see Figure 5-5. Shifting the flood defence towards the river side is not allowed, because in the Netherlands the current understanding is that there should be more room for the rivers instead of less, so any alignment that is shifting towards the river is excluded. The eastern part of the Grebbedijk is challenging to change the alignment because there is the city of Wageningen located behind the Grebbedijk. For the western part there are various possibilities for another alignment. The spatial layout is the basis for different alignments, so roads are a good option for a new alignment and straight across properties or farmlands are less desirable solutions. The different alignments have one thing in common; they all are shorter than the present alignment.

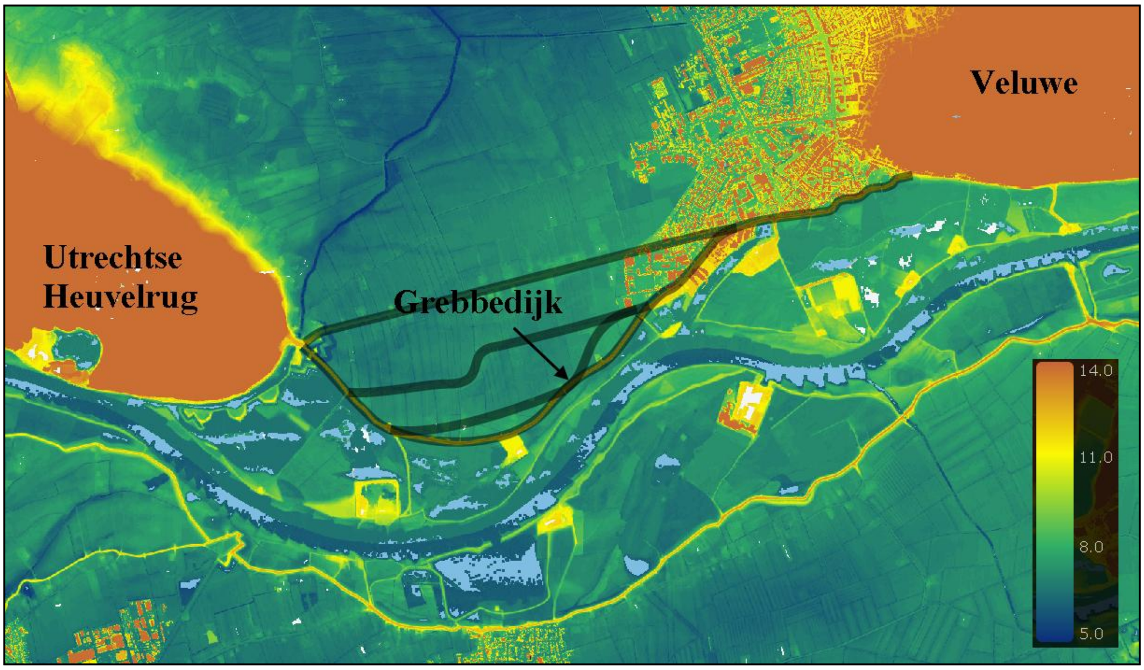

Figure 5-5: Possible other alignments of the flood defence

The western part of the Grebbedijk is a rural area with mostly agriculture on the landside of the Grebbedijk and nature and recreation on the river side. A landward change of alignment would only be an advantage for the length of the dike, but this does not compensate with the difficulties it introduces to the land that becomes a flood plain. The most efficient solution for this part of the Grebbedijk is to keep the present alignment. In the eastern part of the Grebbedijk there is no demand for spatial development with the use of any kind of structure. This part of flood defence should hold its natural character and applying any structure would counteract that. For the western part holds more or less the same as for the eastern part, the dike and the flood plains have nature character and the cultivation behind the dike prevents the dike to develop landwards. The middle part of the Grebbedijk (industrial and marina area) is located close to the centre of Wageningen, which makes it possible to have a multifunctional design for the flood defence. For impressions of the Grebbedijk and its surroundings see appendix B.2.3.

5.2.2 Stakeholder analysis

In order to given some insight in the stakeholders that are involved with a new design for the Grebbedijk a stakeholder analysis is performed. The Grebbedijk crosses a provincial boarder as well as a municipality boarder; this complicates the design process of the Grebbedijk even more because different provinces and municipalities can have different desires. Luckily the multifunctional part of the flood defence is completely located in one of the two provinces and municipalities. The challenging part is that the multifunctionality of the flood defence also involves multiple governmental parties. The municipality is responsible the zoning plans in which the destinations of the land is defined. Having two destinations for the same land, i.e. multifunctional use of land, is possible by law. The water boards are responsible for the management and maintenance of the flood defences, but the government pays for dike reinforcements. The water boards are initially not interested in multifunctional use because their primary responsibility is the protection against flooding. Multifunctional use will only complicate the management and maintenance of the flood defences, resulting in higher costs for the water boards.

Multifunctional use of flood defences asks for a wider view of all involved governmental bodies than the primary objectives. At first sight the total costs of the combination of a flood defence and a structure should be lower than a separation of the structure and flood defence because the land use is smaller. The distribution of costs over the different stakeholders that are involved is of less importance, first the lower costs of the multifunctional flood defence should be verified and when that is the case the distribution of costs should be possible in some way. This is not only the case for the construction costs but also for the costs over the lifetime of the multifunctional design there should be made arrangements in advance.

Of course there are also private parties involved for the construction of a multifunctional flood defence. The local residents and nearby companies should be informed and given participation in the early stage of the design process so they can have their influence on it, but this is not different than in other cases of dike reinforcements. Also nature organisations are involved since the flood plains of the Nederrijn near Wageningen are protected natural areas. See appendix 0 for a more extensive elaboration of the stakeholders that are involved in the design of a multifunctional flood defence for the Grebbedijk.

5.3 Multifunctional design

The multifunctional design of the Grebbedijk is illustrated in the next three paragraphs. The first paragraph threats the layout of the dike, the second paragraph the function that is assigned to the structure and the last paragraph determines the dimensions of the flood defence as a starting point for the calculations.

5.3.1 Area layout

The part of the Grebbedijk that is found to be feasible for multifunctional use with the application of a structure inside of the flood defence is the marina and industrial area, see Figure 5-6. The reason that it is suitable in this area is that the spatial density is high and there is a demand for development of the marina, harbour and industrial area. Additionally, traditional dike reinforcement in this area is complex due to the spatial density and the industry.

Figure 5-6: Location of the multifunctional flood defence

The area and stakeholder analysis show that a multifunctional flood defence, in this case a structure inside of the dike, is not feasible to be applied over the whole length of the dike. The western part is an area that is used for agricultural, nature and recreation and those functions should be maintained. If this part of the dike is reinforced, the combination with other functions can be integrated into the design. This can be obtained in various ways, for example by heightening, milder slopes and a berm on the inner slope of the dike. The natural function can be enhanced by moving the road from the crest to the berm, in this way the traffic is not visible from the river side. The recreational function can be enhanced by using the current road for cyclists and pedestrians; a cross section of this idea is shown in Figure 5-7. The multifunctionality of this part of the flood defence is not obtained by the use of a structure inside of the dike and therefore falls outside the scope of this research.

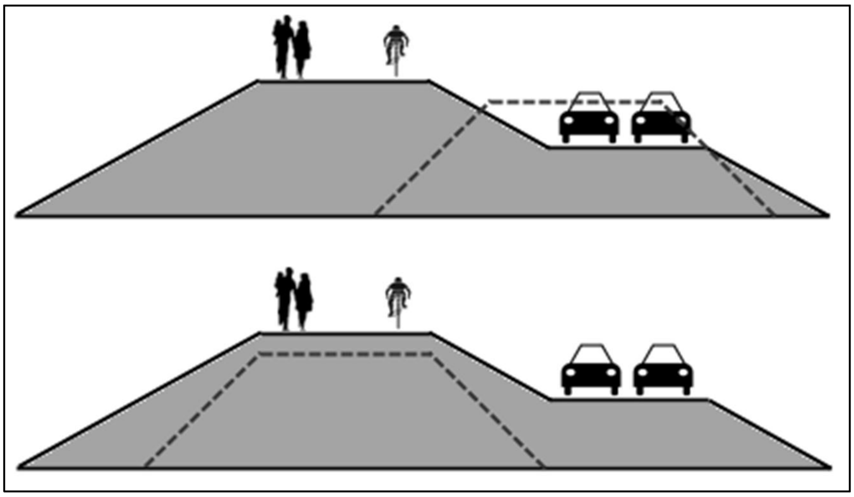

Figure 5-7: Possible reinforcement western part of the Grebbedijk

Behind the eastern part of the Grebbedijk are the old city centre and a new built neighbourhood located with only a small road for cyclists on top of the dike. The recreational and natural functional of the flood plains determines the appearance of this part of the dike, so the combination with a structure inside of the dike is not feasible. Due to the lack of demand, dike reinforcements are likely to be done in a traditional way, leaving this part of the Grebbedijk out of the scope of this research.

5.3.2 Functions

As mentioned in the previous paragraph the part of the Grebbedijk that is designed with a structure inside of the dike is surrounded by the harbour and industrial area. The function of the structure can be varied. It could be used as offices, workspace, shops, parking garage, housing, infrastructure or a combination of the previous functions. As function of the structure is chosen for a parking garage because it is close to the city centre and the city centre has problems with parking space (De weekkrant Wageningen, 2013). The other functions are not likely because the surrounding area of this part of the Grebbedijk is an industrial area, so to use the structure for housing is not a logical option. Offices and workspace are not likely because the small amount of natural light, the possible noise pollution from the surrounding industry and infrastructure and the mental aspects of working underground. First is looked to this structure as the basic design, later on can be varied in the dimensions of the structure which also allows other functions to be assigned to the structure.

5.3.3 Dimensions

The dimensions of the flood defence are stated in Figure 5-8 and are the result of the calculations done in the upcoming paragraphs. The calculations resulted in adjustments to the design and the different calculations influenced each other. The calculations are based on the assumptions stated in Table 5-4. The width of the structure is chosen using of available amount of space between the existing buildings. The reason for the chosen height of the structure is the infrastructure which needs to be located on top of the structure, so a multiple layer structure above ground level is not possible.

Table 5-4: Assumption as a starting point for the calculations

Parameter	Value	Unit
Gradient outer slope	1:3	-
Design water level	12.0	m+NAP
Ground level hinterland	8.5	m+NAP
Crest height	13.0	m+NAP
Crest width	30	m
Thickness top soil layer	1	m
Thickness concrete structure	0.5	m
Length of the structure	300	m

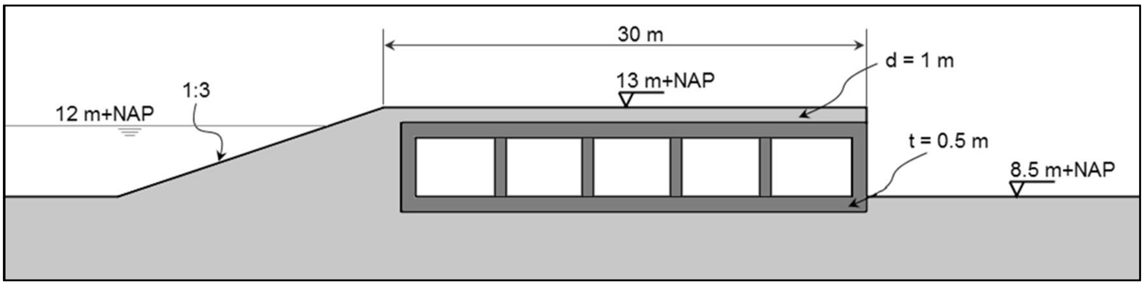

Figure 5-8: Cross section of the multifunctional flood defence

The landward side of the flood defence is designed as a vertical wall and not as a slope. This is done because the structure itself can be used as the landward side of the flood defence. Normally the soil body of the crest needs the support of the inner slope to be stable but in this case the structure takes up most of the crest. The soil layer on top of the flood defence needs in this case a border on the landward side in order to be stable. The difficulty with this vertical landwards side of the flood defence is in case of overtopping. The water that is overtopping the dike will fall vertically on the surface level, resulting in large impact forces. This can be solved by allowing a minimal amount of water overtopping the flood defence (i.e. decreasing the load on the surface level) and by applying a stronger revetment on the surface (i.e. increasing the strength). Another reason for not having a slope on the landward side is the land use. With a surface level difference of 5 m and a slope of 1:3 this would increase the width of the flood defence by 15 m.

Some parts of the present dike have a high foreshore on which the harbour area is located. In the present design the harbour area and the flood defence area are separate. The influence of the higher foreshore on the flood defence is obviously positive; it reduces the load on the flood defence. Other parts of the present dike do not have the higher foreshore in front of the dike. The location with the highest loads (hence no high foreshore reducing the loads) is in this case the governing situation; therefore the higher foreshore is not taken into account in this design. Another reason for not taking this into account is the fact that this is a case study. Taking into account the specific situation with the higher foreshore keeps this design from being a representative situation for other locations.

5.4 Failure mechanisms

This design of the Grebbedijk does not look like a traditional dike and cannot be assessed as one. In order to assess this multifunctional flood defence the failure mechanisms of a traditional dike and a hydraulic structure as flood defence are evaluated. The failure mechanisms of a traditional dike and a hydraulic structure are stated in Table 5-5. From these failure mechanisms four are selected that require a better look. The other failure mechanisms can either be assessed in a similar way or are not applicable for a multifunctional flood defence.

Table 5-5: Failure mechanisms reviewed of the multifunctional flood defence

Type of flood defence			
Traditional dike	Hydraulic structure		Multifunctional flood defence
Height	Height		Height
Piping and heave	Piping and heave		Piping
Macro stability inwards	-		x
Macro stability outwards	-		x
Micro stability	-		x
Revetment	-	→	x
Foreshore	Foreshore		x
Not water retaining objects	-		x
-	Stability of soil and structure		Stability of soil and structure
-	Strength of structural elements		Strength of structural elements
-	Reliability closure		x
-	Connections		Connections

In the first paragraph the reasons are given why some failure mechanisms are not reviewed. The second paragraph treats the failure mechanisms that are reviewed and in the last paragraph some other attentions points are mentioned.

5.4.1 Not reviewed failure mechanisms

The design has positive effects on some of the failure mechanisms because it takes away the possibility of occurrence or the treatment of the failure mechanisms does not differ from the normal approach. For the following failure mechanisms there are positive effects noted:

Macro stability inwards

This failure mechanism is not able to occur because the there is no inner slope of the flood defence. Even if there would be an inner slope, the sliding circles are limited to the inner crest line and the occurrence of this failure mechanism would not result in failure of the flood defence because this structure and the outer slope are not affected by failing of the inner slope.

Macro stability outwards

This failure mechanism occurs when there is a sudden drop in water level and the phreatic level inside of the flood defence is high. The structure inside of the flood defence has zero to a slightly positive effect because the structure takes up partly the space of the driving soil part of the sliding circle.

Micro stability

Because of the large width of the flood defence and the structure inside of the flood defence the phreatic level is not likely to exit the inner slope of the flood defence. When the phreatic level does not exit the inner slope this failure mechanism is not able to occur, so a positive effect for the failure mechanism.

Revetment

It is assumed that the structure inside of the flood defence does not change the strength of the revetment or the loading on the outer slope. The revetment on the land side of the flood defence can be loaded with falling water from the top of the flood defence. The impact of the falling water on the surface level is high, but can structurally be solved by applying a paved surface directly behind the structure.

Foreshore

The foreshore failure mechanism is not influenced by the structure inside of the flood defence and therefore not reviewed in the research.

Not water retaining objects

Not water retaining objects need to be assessed individually and cannot be assessed in a general way. The issue with not water retaining objects is that it reduces the strength and increases the load on the flood defence. It is very likely that the crest of the flood defence will be used for other purposes like infrastructure, nature or buildings. The increase in load and reduction in strength are not a problem on the crest because the flow velocities will be small due to the length of the crest.

Reliability closure

This failure mechanism is only applicable for moving hydraulic structures like sluices. In this case there are no parts that ever open or close, so this failure mechanism does not occur.

Other

There are also other causes than the described failure mechanisms that could lead to the failure of the flood defence; one of them is human interaction with the structure. Inattention during for instance maintenance or adjustments of the structure may lead to reduction of the strength of the structure. Also the use of the flood defence introduces extra interaction with the structure. An accident with a car of the explosion of a car damages the structure. The probabilities of these events during design conditions are assumed to be much lower than the failure probabilities of the other failure mechanisms. Further research to these events is required but outside of the scope of this thesis.

5.4.2 Reviewed failure mechanisms

The failure mechanisms that are reviewed are stated in this paragraph. The other failure mechanisms are not reviewed because they are not applicable in this situation or being not different from the situation of a traditional dike. This holds that there are five failure mechanisms left which needs to be assessed in order to have a safe design of the flood defence:

- Overtopping
- Piping
- Stability of the structure and soil body
- Strength of the structural elements
- Connections

The first four failure mechanisms are assessed for the cross section of the multifunctional flood defences. The connections between the traditional dikes at both ends are also possible failure mechanism, because a transition between a concrete structure and a grass revetment is very sensitive to erosion. The five failure mechanisms are elaborated in the upcoming paragraphs.

5.4.3 Overtopping

For the overtopping criterion the crest height is the most important parameter. The amount of water overtopping can result in two ways of failure, an excess amount of water behind the flood defence or erosion of the flood defence causing a breach. The erosion is the more important aspect because the consequences of a breach are far worse than the consequences of the excess amount of water and preventing the erosion from happening results in such small volumes of water overtopping the dike causing no excess amount of water behind the dike.

The overtopping criterion is based on an allowable amount of water that is overtopping the dike, depending on the quality of the grass cover of the inner slope. The allowable overtopping discharge results in a sufficiently low load on the grass cover for the corresponding quality i.e. strength of the grass cover. In Table 5-6 are the distinguished overtopping discharges stated.

Table 5-6: Allowable overtopping discharge depending on the quality of the grass cover

Allowable amount of overtopping discharge	Description
0.1 l/m/s	Low quality of the grass cover
1.0 l/m/s	Good quality of the grass cover
10.0 l/m/s	Excellent quality of the grass cover

The allowable overtopping discharge is equal to 0.1 l/m/s, because the transitions between the dike and the structure are very sensitive to erosion. In these areas the load on the grass cover is larger and at the same time the quality, hence the strength, of grass cover is lower. The vertical inner slope is another reason to allow only the smallest amount of water overtopping the dike because the falling water from the top of the structure down on the surface area results in very large impact forces. If the load reduction on the surface area is not enough due to the lowest allowable overtopping discharge, the surface area could be protected by a revetment.

For the calculation of the crest height resulting in an overtopping discharge of 0.1 l/m/s the software PCOverslag is used. The input data for the program is based on the following assumptions:

- Angle of the outer slope is 1:3.
- The outer slope is covert with a grass layer.
- The water level is 12.0 m+NAP.
- The wave height is 0.5 m.
- The wave period is 2.2 s.

The outcome of the calculation is that a crest height of 13.0 m+NAP is sufficient enough to have a 0.1 l/m/s overtopping discharge. The overtopping discharge is calculated at the outer crest line, not taking into account anything that happens behind the outer crest line. As said before the overtopping discharge is a measure for the load on the inner slope of the dike. In case of a traditional dike, the water that passes the outer crest line flows over the crest on the inner crest. In this case, the water has to flow over a considerably longer crest, reduces the amount of energy i.e. the load of the wave. Taking this into account the program overestimates the amount that really reaches the inner crest line.

In order to investigate the water on the crest of the flood defence two comparisons are made. The first comparison is the comparison with precipitation. Since the crest width is quite large the overtopping discharges of 1.0 and 0.1 l/m/s are compared to precipitation to see whether it can be schematised as precipitation. If it could be schematised as precipitation, the overtopping on the inner crest line can be neglected because of infiltration and the limited surface flow. From this analysis is concluded that an overtopping discharge of 1.0 l/m/s can be compared with precipitation for a very short period, in the order of minutes, so not to the storm duration. An overtopping discharge of 0.1 l/m/s can be compared to precipitation with a duration in the order of hours, but the occurrence of this precipitation is less often than once every 10 years, so the overtopping discharge cannot be schematised and therefore neglected as precipitation. The second comparison is made with a storage volume on the crest of the dike. Both overtopping discharge are considered again, trying to store the volume of water of an 8 hour storm under the assumption that the water not infiltrate the soil and will not flow back towards the river side. Both volume where so large that border on the inner crest line should be unacceptably large.

The calculations on the overtopping discharge and the two comparisons made are presented in appendix C. The final outcome of the calculations is not conclusive about the overtopping discharge on the inner crest line, so the crest height corresponding to the 0.1 l/m/s overtopping discharge of 13.0 m+NAP is kept as a conservative approach, while remarking that this could be optimised.

5.4.4 Piping

The failure probability of the western part of the Grebbedijk consists largely of the failure probability due to piping (Van der Scheer et al., 2012). This is the result of former branches of the Nederrijn that were meandering under the present location of the Grebbedijk. The former branches have formed sand layers through which the water flows easily, resulting in large probabilities for piping. The part of the Grebbedijk that has a multifunctional design is situated partly in this area and therefore the piping failure mechanism is of great importance. Preventive measures against the failure mechanism piping are difficult to implement and expensive. There are different measures available, preventing on different aspects of the failure mechanism. The measures that are distinguished:

- Increase of the seepage length, horizontal or vertical
- Preventing uplift of the clay layer
- Decrease of the water difference over the flood defence
- Preventing the wash out of sand

The most common preventing measures to piping problems are the establishment of a piping berm on the inner side of the flood defence or a clay layer on the river side of the flood defence; both measures increase the seepage length. With this multifunctional design of a flood defence a vertical increase of the seepage length with the application of sheet pile wall seems a good solution if the piping criterion is not fulfilled.

At first the design rules of Bligh and Lane are used to have an estimate whether this design has problems with piping or not. The empirical formula of Bligh and Lane give a critical ratio between the water difference and the seepage length. In the formula of Bligh the horizontal and vertical parts of the pipe are of equal importance. In the formula of Lane the vertical pipes are of more importance because, with the gravity acting in the same directions as the pipe alignment, the vertical pipe is more likely to collapse than a horizontal pipe. Since there is nearly no vertical seepage length in the original design the rule of Bligh is applied:

$$L = \sum L_v + \sum L_h \geq C_B \cdot \Delta H$$

$$\begin{array}{lll} \sum L_V & = 0.5 & [m] \\ \sum L_h & = 30 + 3 \cdot (13 - 8) = 45 & [m] \\ L & = 45.5 & [m] \\ \Delta H & = 12 - 8.5 = 3.5 & [m] \end{array}$$

$$\begin{array}{lllll} C_B = 12 \text{ (coarse sand)} & \rightarrow & L \geq C_B \cdot \Delta H & \rightarrow & 45.5 \geq 42 \\ C_B = 15 \text{ (fine sand)} & \rightarrow & L \geq C_B \cdot \Delta H & \rightarrow & 45.5 \leq 52.5 \end{array}$$

The coefficient is depending on the type of soil, for sand the value is somewhere between 12 and 15. So the seepage length is just or just not sufficient to withstand the water difference over the flood defence. Soil investigation should be carried out in order to have exact data to determine the soil parameters. The formula of Bligh is not conclusive about the safety on the piping failure mechanism so a sheet pile wall is applied at the river side of the structure, see Figure 5-9.

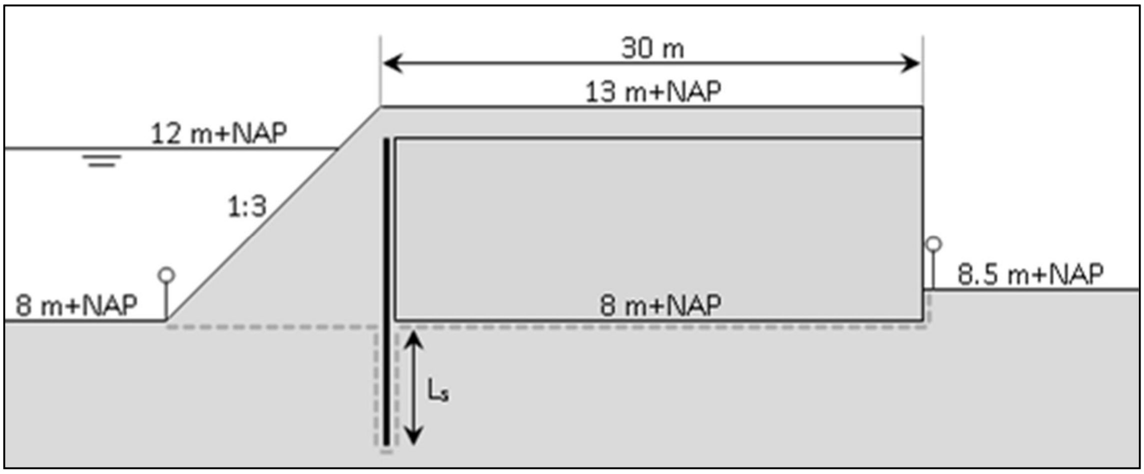

Figure 5-9: Application of a sheet pile wall and the seepage length

With the help of the formula of Lane is the length of the sheet pile wall calculated:

$$L = \sum L_v + \frac{1}{3} \sum L_h \geq C_L \cdot \Delta H$$

$$\begin{array}{lll} \sum L_V & = 0.5 & [m] \\ \sum L_h & = 30 + 3 \cdot (13 - 8) = 45 & [m] \\ L & = 15.5 & [m] \\ \Delta H & = 12 - 8.5 = 3.5 & [m] \end{array}$$

$$\begin{array}{lllllllll} C_L = 5 \text{ (coarse sand)} & \rightarrow & L \geq C_L \cdot \Delta H & \rightarrow & 15.5 \leq 17.5 & \rightarrow & 17.5 - 15.5 = 2 & \rightarrow & L_S = 1.0 \quad [m] \\ C_L = 7 \text{ (fne sand)} & \rightarrow & L \geq C_L \cdot \Delta H & \rightarrow & 15.5 \leq 24.5 & \rightarrow & 24.5 - 15.5 = 9 & \rightarrow & L_S = 4.5 \quad [m] \end{array}$$

The formula of Lane uses other coefficients but still depending on the soil type, in this case between the 5 and 7. In order to have a better insight in piping a calculation is made with the help of the program MSeep. MSeep is based on the formula of Sellmeijer and is more accurate than the previous used empirical formulas of Bligh and Lane. The input and output of the program together with the more extensive elaboration of the calculations of the Bligh and Lane formulas can be found in appendix 0.

From the calculations with the help of MSeep can be concluded that a sheet pile wall till 6.5 m+NAP is necessary (see Figure 5-10) if the permeability of the sand layer is equal to 10^{-3}. For a permeability of 10^{-4} is no sheet pile wall necessary; again concluding that the actual soil data should reveal the necessary measures to be taken.

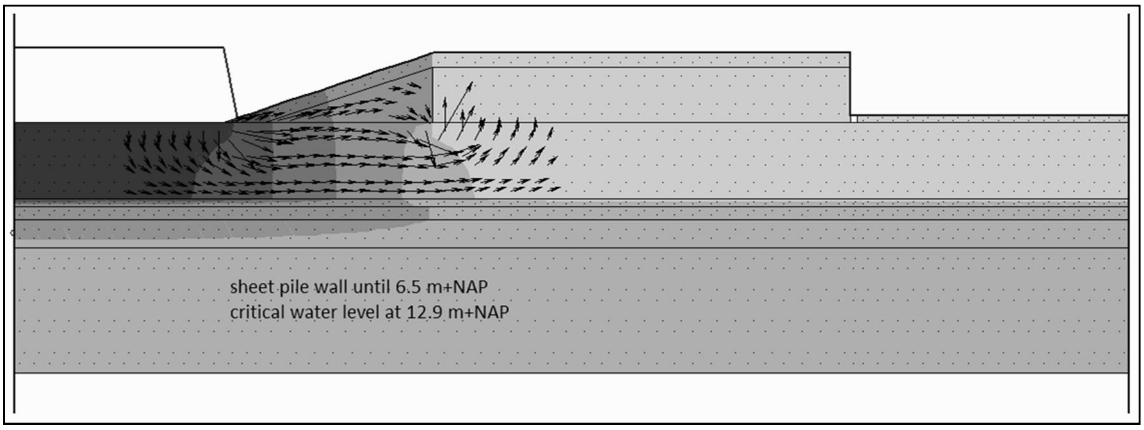

Figure 5-10: Potentials and flow velocities in case of a sheet pile wall

During the construction of the multifunctional flood defence a sheet pile wall is necessary to guarantee the safety. This sheet pile wall can be used as a permanent part of the flood defence instead of a temporary part so it not only guarantees the safety during construction but acts as a seepage barrier as well. The required length of the sheet pile wall during the construction phase will be longer than the required length for the seepage length, see paragraph 5.4.7 about the construction phase.

5.4.5 Stability of the structure

This paragraph treats the overall stability of the structure. The overall stability consists of the horizontal stability and the overturning stability. The calculations are presented in appendix C.3.

Horizontal stability

The overall stability of the structure is an important aspect for the safety assessment. Failing of the horizontal stability of the structure will lead to large consequences: a sudden and immediate flooding of the hinterland. For traditional dikes this is also an important aspect, horizontal shearing of the entire dike body caused in 2003 a sudden flooding in Wilnis in the Netherlands. The failure probability of this failure mechanism is 1% of the norm frequency for hydraulic structures, given the sudden failure without any signs before failure and the immediate flooding that occurs. This is also assumed for this design of the multifunctional flood defence.

The horizontal stability is the balance between the horizontal forces acting on the structure and the friction force between the structure and the soil. The friction force is generated by the vertical forces from the structure and the soil on top of the structure minus the upward pressure of the water under the structure. Other forces such as the soil resistance on the right side of the structure and weight of the columns in the structure are neglected to simplify the calculations, both being conservative assumptions. The schematization of the force is according to Figure 5-11.

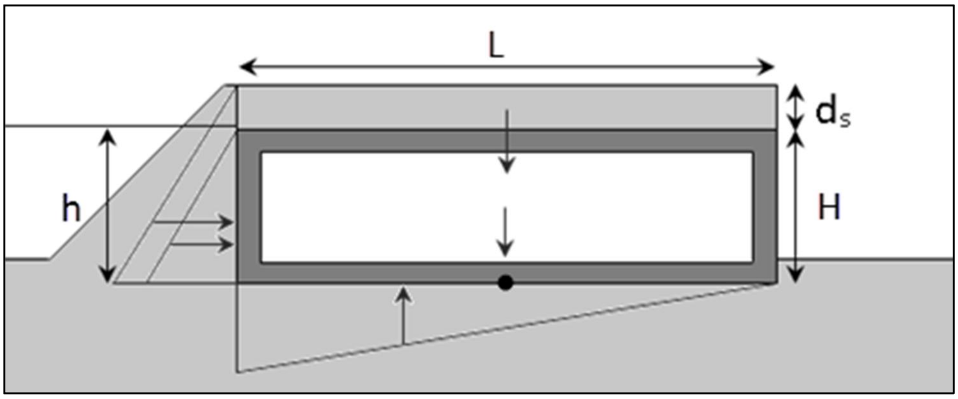

Figure 5-11: Schematization of the forces on the structure

The vertical forces generating the resistance have to be multiplied by the coefficient of friction. Dividing the resistance by the loading results in the following formula:

$$f \cdot \sum V > \sum H \quad \rightarrow \quad \frac{f \cdot \sum V}{\sum H} > 1$$

Where:

$\sum H$ sum of the horizontal forces
$\sum V$ sum of the vertical forces
f coefficient of friction

Partial factors need to be applied on the individual force in order to have a safe design. The Eurocode prescribes the partial factors and are presented in Table 5-7.

Table 5-7: Partial factors used for the horizontal stability

Reliability class	Permanent unfavourable	Permanent favourable	Variable
EC RC2 (≈ NEN RC3)	1.10	0.90	1.50
EC RC3	1.20	0.90	1.65

Applying the partial factors for variable loads of 1.50 or 1.65 on the forces resulting from the water pressures with an exceedance probability of 1/1250 per year will result in unrealistic values of the water forces. The guideline hydraulic structure therefore presents partial factors for hydraulic pressures depending on the exceedance probability of the water level and the prescribed amount of reliability of the structure. In Table 5-8 are the partial factors for hydraulic pressures presented.

Table 5-8: Partial factors for force resulting from the water level

Failure probability	1/1250	1/2000	1/4000	1/10000

EC RC2 (≈ NEN RC3)	1.28	1.26*	1.26*	1.26*
EC RC3	1.55	1.48	1.38	1.27
*Guideline prevailing over the Eurocode				

As is explained before, before the introduction of the Eurocode the guideline was prevailing over the former NEN codes. The introduction of the Eurocode resulted in a higher reliability class in which hydraulic structure are placed. Performing the calculations (see appendix 0) led to the results presented in Table 5-9.

Table 5-9: Results of the deterministic calculation of the horizontal stability

Failure probability	1/1250		1/10000	
Reliability class	RC2	RC3	RC2	RC3
Partial factor	1.28	1.55	1.26	1.27
Unity check	0.50	0.67	0.59	0.62

From the results can be concluded that the difference is quite large between the four cases that are considered. It is remarkable that the unity check for an exceedance probability of 1/1250 in RC3 has the highest value, especially because this design is based on a design water level with an exceedance probability of 1/10000 per year.

Overturning stability

For the overturning stability of the structure the same situation is considered as for the horizontal stability. The overturning stability checks whether the pressure distribution on the soil under the construction does not exceed the limits. The limits are: (i) everywhere under the structure pressures greater than or equal to zero and (ii) no exceeding of the bearing capacity of the soil. This is transformed to a geometrical requirement on the resulting force. The resulting force has to be within 1/6 of the length of the structure from the centre point of the structure to be sure there are only pressures greater than or equal to zero, see Figure 5-12. In formula:

$$\frac{\sum M}{\sum V} < \frac{1}{6}L \quad \rightarrow \quad \frac{L\sum V}{6\sum M} > 1$$

Where:

$\sum M$ sum of the moments
$\sum V$ sum of the vertical forces
L length of the structure

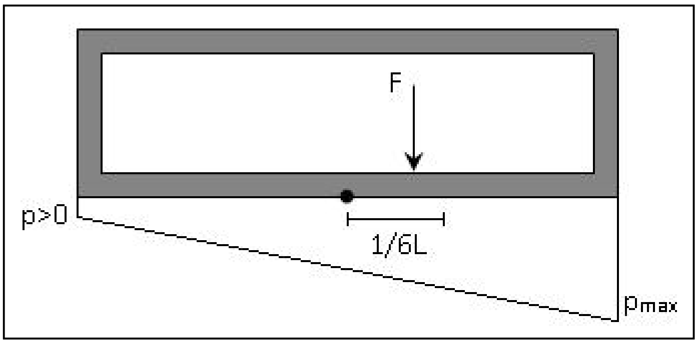

Figure 5-12: Overturning stability criteria

The forces resulting from the weight of the structure and the weight of the soil on top of the structure do not generate any moments because the resulting forces intersect with the centre point under the structure, but they do contribute to the sum of the vertical forces. The partial factors that need to be used are the same as for the horizontal stability. The results from the calculations are presented in Table 5-10. The same phenomena as for the horizontal stability can be obtained; with the use of the prescribed partial factors the situation with an exceedance probability of 1/1250 per year and reliability class 3 has the highest unity check, which is not unexpected given the partial factors. This confirms the doubt about the applicability of the partial factors again.

Table 5-10: Results of the deterministic calculation of the horizontal stability

Failure probability	1/1250		1/10000	
Reliability class	RC2	RC3	RC2	RC3
Partial factor	1.28	1.55	1.26	1.27
Unity check	0.75	1.05	0.88	0.89

The application of the prescribed partial factors on the different forces and moments seems incorrect because the forces and moments do not have the same proportionality to the water level. The force resulting from the water pressure on the left side of the structure is proportional to the squared of the water level and the force resulting from the water pressure under the structure is proportional to the water level. The same holds for the moments resulting from these forces; on the left side is the moment proportional to the third power of the water level, the moment under the structure still proportional to the water level.

The maximum pressure under the right side of the structure can be calculated with the following formula:

$$\sigma = \frac{\Sigma V}{B \cdot L} + \frac{\Sigma M}{1/6 \cdot B \cdot L^2}$$

Where:

σ	pressure
B	width of the structure
L	length of the structure
ΣV	sum of the vertical forces
ΣM	sum of the moment

The loads that were taken into account for the calculation of the minimum pressure under the left side of the structure are the forces resulting from the water pressure, the soil pressure and the weight of the structure and soil. For the maximum pressure under the structure is also the variable load in and on the structure taken into account. With the following input values for the parameters:

B	1	m
L	30	m
ΣV	1843	kN
ΣM	2875	kN

The resulting pressure under the structure is equal to 90 kN/m^2. The bearing capacity under the structure depends on the soil under the structure. The soil on this location consists largely out of sand. The bearing capacity of sand is in the range of 200 kN/m^2 to 500 kN/m^2, so the soil is able to carry structure.

Conclusion

For the calculations on the stability of the structure are both times four different cases obtained, varying in reliability class and exceedance probability of the design water level. On beforehand there was already a big difference obtained by only looking at the partial factor. Using these partial factors in the calculations has given a remarkable outcome; the highest result is obtained by using the third reliability class and a design water level with an exceedance probability of 1/1250 per year. The idea of the partial factors in combination with a design water level with a certain exceedance probability is that it does not matter which design water level is used in the calculations because the partial factor (depending on the exceedance probability) will correct this to have equal reliability of both calculations. This is not obtained in the output of the calculations for the stability of the structure. To be able to obtain the reliability of the calculation is higher level probabilistic approach required, this in researched in chapter 6.

5.4.6 Strength of structural elements

The structure is composed out of several elements which could fail individually due to overloading. For elements of hydraulic structures like the sluice door of a sluice complex, failure will result in an immediately flooding of the hinterland. For that reason is in the flooding risk analysis of hydraulic structures the failure space for structural failure limited to 1/100 of the norm frequency. The second argument for the failure space of 1/100 of the norm frequency is the correlation between the failures of the different elements. Failure of one element may lead to failure of the whole structure due to the correlation by the water load. This also holds for multifunctional flood defences. The structure is divided into different elements, which can be assessed separately. The elements that are assessed are the roof and bottom slab, the wall and the columns inside the structure. Failure of some elements does not directly lead to failure of the flood defence. If a column fails, the flood defence remains intact due to the robustness in the structure. The structure loses in that case the function of parking garage but not as flood defences.

For the stability of the structure is already demonstrated that application of different design water levels and partial factors does not give the right answers. This also holds for the structural calculation on the strength of the elements. Therefore is chosen for these calculations to obtain only one case; reliability class 3 and a design water level with an exceedance probability of 1/10000 per year. The partial factor for hydraulic loads should be equal to 1.27 according to the guideline hydraulic structures, which is even lower than the partial factor for permanent loads according to the Eurocode. Therefore is chosen to use the same partial factor for hydraulic loads as for permanent loads, see Table 5-11.

Table 5-11: Partial factors used structural calculations

Type of load	Partial factor
Permanent (unfav.)	1.30
Permanent (fav.)	0.90
Variable	1.65
Hydraulic	1.30

The same holds for the strength calculations as for the stability calculations; in order to obtain the reliability of the calculations there must be chosen for a higher level probabilistic approach. The essence of the calculations is elaborated in the upcoming sections; the complete calculations are presented in appendix C.4.

Wall

The wall of the structure experiences the water pressure, soil pressure and an axial force resulting from the loads on top of the structure. The wall is schematised as a beam on two supports loaded with the water pressure and the soil pressure both linear increasing over the depth.

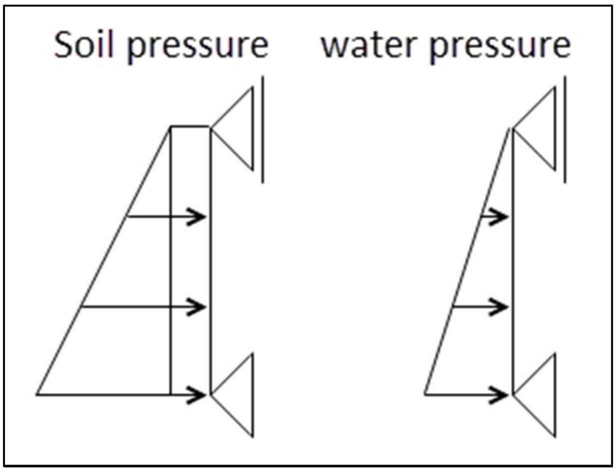

Figure 5-13: Schematisation of the wall

Assumed is that failure of the wall is caused by an acting bending moment that exerts the resisting bending moment of the wall. The acting moment is the result of the soil and water pressure and the resisting moment is generated by the reinforcement steel in the wall. Buckling could also occur due to the axial load on the wall. This is checked by calculating the compressive concrete stress resulting from the bending moment as well as the axial force in the wall. The total compressive stress in the concrete must be lower than the allowable compressive stress in order to prevent buckling. From the calculations presented in the appendix follows that the wall meets the requirements on buckling and bending moment resistance.

Roof and bottom slab

The roof slab is the second element of the structure that is assessed. The roof slab is loaded by the weight of the soil on top of the roof, the weight of the roof slab itself, the axial force from the horizontal water and soil pressure and a variable load that may be present on the soil on top of the structure. The roof slab is schematised as a beam on two supports with a distance between the supports equal to the centre to centre distance between the columns. The bottom slab is the opposite of the roof slab; the water pressure under the structure causes bending in the upward direct. The axial force in the bottom slab results from the same forces as for the roof slab, it only takes twice the loading as for the roof slab because of the force distribution on the wall, see Figure 5-14.

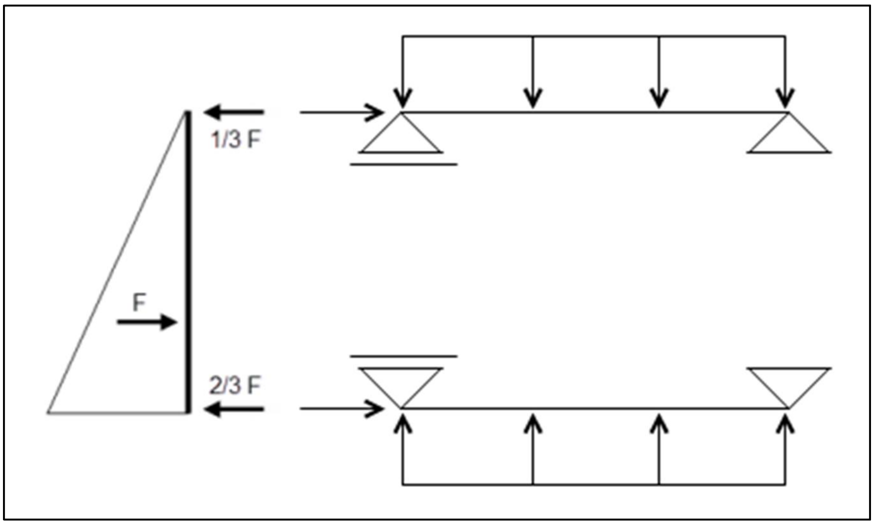

Figure 5-14: Schematisation of the forces on the roof and bottom slab

However, the horizontal forces in the roof slab are concentrated to the intermediate walls resulting in high local axial forces in the roof slab, later on more on global distribution of the forces through the structure. The distribution in the bottom slab is constant in the length of the structure. The roof and bottom slab are assessed in bending moment resistance and buckling. The results from the calculation presented in the appendix are that the roof and bottom slab fulfil the requirements on bending moment resistance and buckling.

Columns

The columns support the roof slab, so the loads on the column are the same as for the roof slab; variable load on top of the soil, the weight of the soil on top of the structure and the weight of the roof slab itself. The weight of the column itself is neglected because it is very small compared to the other loads and also the acting surface area of the forces is different.

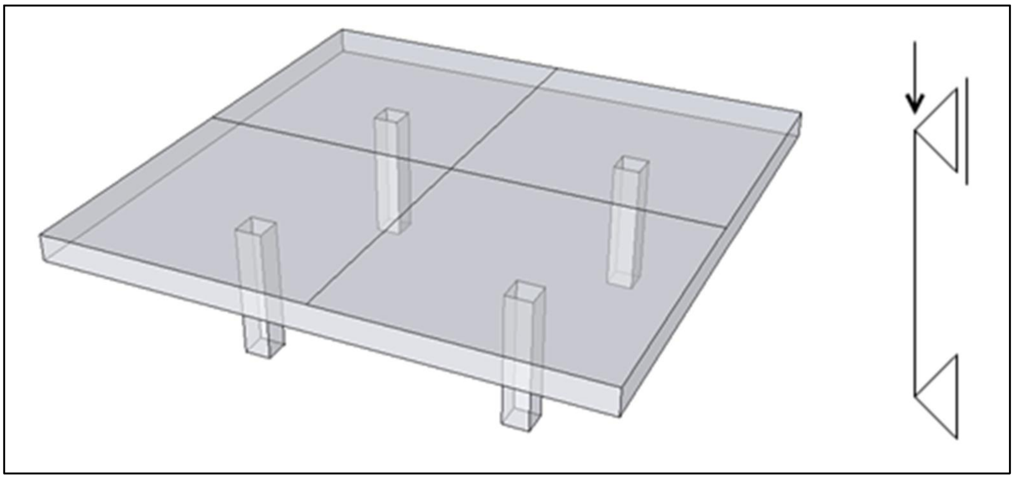

Figure 5-15: Schematisation of the columns

The column is assessed on the maximum concrete pressure as well as buckling. Buckling is checked by comparing the critical buckling length to the buckling length of the column. Assumed is that the columns each take the same amount of force equal to a square area with the dimensions equal to the centre to centre distance between the columns, see Figure 5-15. From the calculations presented in the appendix follows that the column fulfils the requirements on compressive strength and buckling.

Global force distribution

In addition to the assessed elements, there must also be looked at the global force distribution within the structure. The water and soil pressure act on the wall of the structure resulting in two support reactions in the bottom and roof slab. The wall is continuously supported by the bottom and roof slab resulting in a constant force distribution over the length of the structure along the dike. The force in the bottom slab stays continuously distributed over the width of the structure because it balances directly with the friction between the structure and the soil. Part of the horizontal force is also balanced by the soil pressure on the landside of the structure but is neglected to simplify the calculations. The most important force is the horizontal force in the roof slab generated by the water and soil pressure. Since there is no support on the landside of the structure, this force has to be transferred down to the bottom slab to balance with the horizontal friction between the bottom slab and the soil. The transfer of the force down to the bottom slab is achieved with the aid of intermediate walls directed from the waterside to the landside. The force in the roof slab is not continuously distributed because the force will concentrate on the intermediate walls, see Figure 5-16. The actual force distribution in the roof slab is depending on the stiffness ratio between the intermediate walls and the end walls.

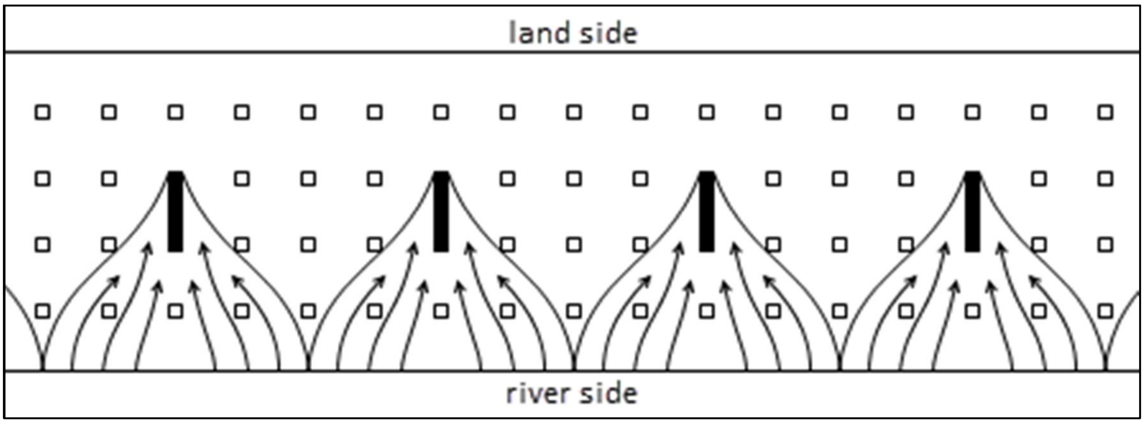

Figure 5-16: Top view of the force distribution in the roof slab

The force in the wall depends on the distance between the intermediate walls, the larger the distance between the intermediate walls the larger the force in a single intermediate wall. The force in the wall is transferred down via the compressive zone in the intermediate wall. The compressive zone in the concrete is from the upper left corner to the lower right corner, see Figure 5-17. The concentrated forces in the roof slab and intermediate wall lead locally to high compressive and tensile forces. Extra attention should be paid to the reinforcement in the roof slab around the intermediate wall and in the intermediate wall itself.

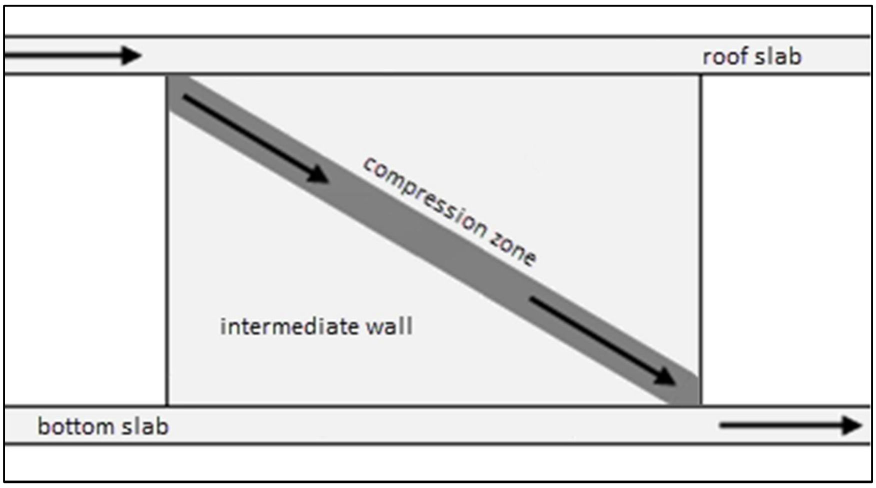

Figure 5-17: Force flow through the intermediate wall

The forces are transferred down via the wall and are balanced in the bottom slab as a concentrated force. The force from the roof slab arriving in the bottom slab via the intermediate wall is a concentrated force, which is distributed over the bottom slab towards the landside of the structure to balance with the friction between the structure and soil again, see Figure 5-18.

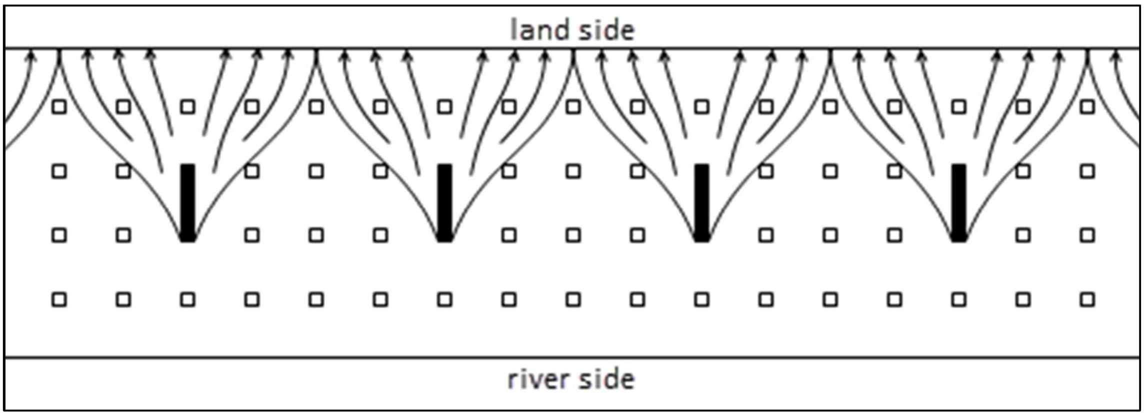

Figure 5-18: Top view of the force distribution in the bottom slab

This analysis of force flow of the horizontal pressure through the structures shows that forces can be concentrated at certain locations. The exact values of the forces are not analysed but when a detailed design of the structures will be made, a three dimensional analysis of the force flow through the structure is required to calculated the peak forces at specific locations. This is not performed in the thesis because it is out of the scope of this thesis.

5.4.7 Connections

The connections are another sensitive part of the flood defence. The connection between a hard structure and a soil body is often the part where the flood defence fails in case of overtopping. Also an increase of the flow capacity along the sides of the structure can cause failure of the flood defence. Due to the increase in flow capacity failure of the revetment, micro or macro instability can occur. To avoid this problem the sheet pile wall can be extended. In that case the wall flow along the structure is obstructed by the sheet pile wall. The sheet pile wall will be extended for the construction of the structure anyway because it requires work space next to the structure for the formwork for casting of the concrete bottom slab and walls.

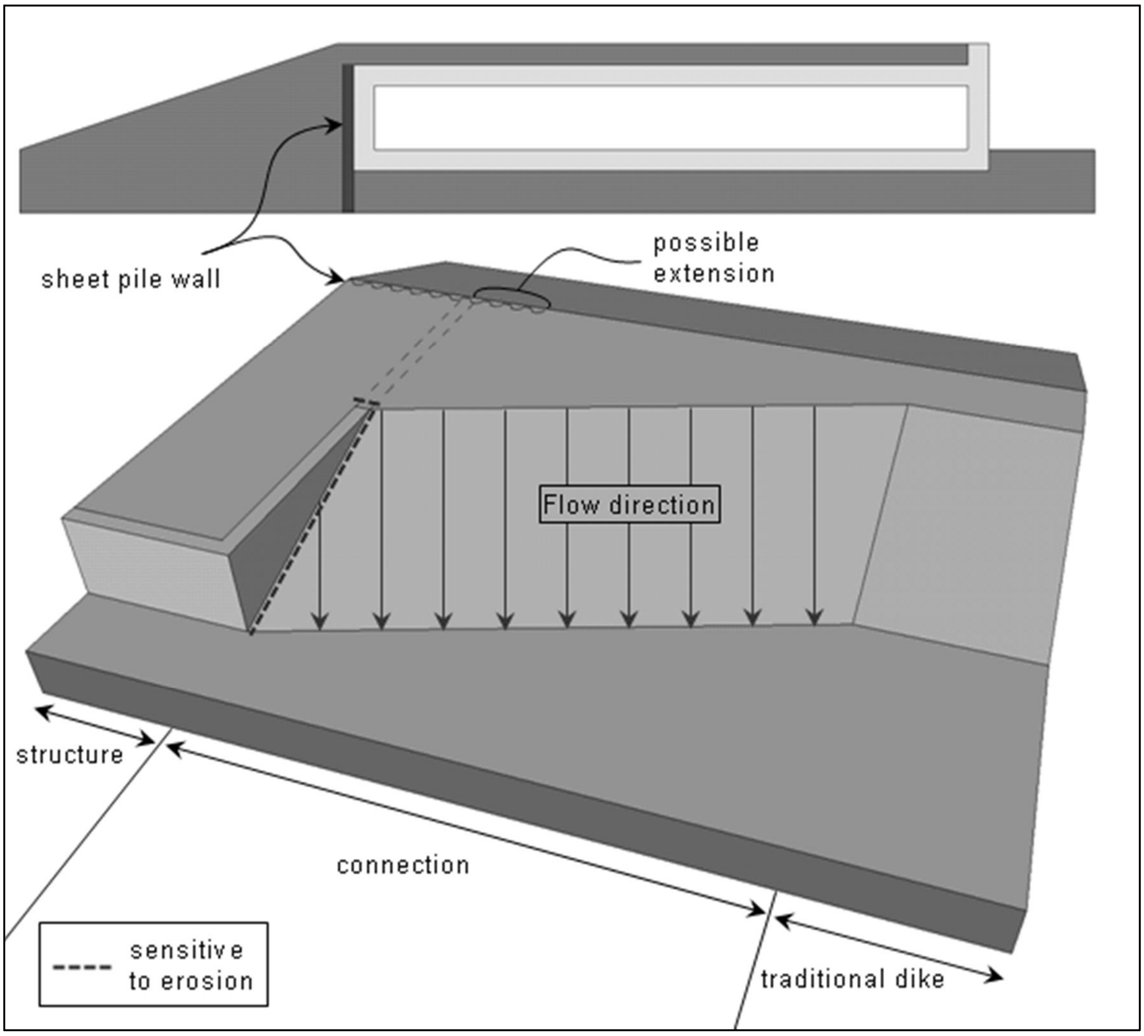

Figure 5-19: Connection between the flood defence with a structure and the traditional dike

In Figure 5-19 the layout of the connection is presented. In order to have a visually attractive transition between the flood defence with a structure and the traditional dike, the inner toe line of the traditional dike is connected to the inner edge of the structure. This could also be necessary to avoid the occurrence of piping partially under the structure, forming a well next to the structure. In addition to the sheet pile wall there are two other phenomena that increase the safety of the connection. The first phenomenon is the width of the crest that becomes larger than the crest width of the traditional dike and the second one is that the slope has an angle with the structure. Due to the larger crest width the flow velocity of overtopping water is reduced, resulting in a lower load on the revetment. The angle between the slope and the structure directs the flow away from the vulnerable interface of the structure and revetment; this reduces the load even more. The concrete edge on top of the structure is slightly higher than the surface level of the soil on top of the structure avoiding overtopping to happen, so that water falling from the top of the flood defence cannot damage the revetment.

5.5 Construction of the design

During construction of the flood defence other loads occur than in the final situation. This results in other forces on the structure and other failure mechanisms of the flood defence. The construction of the multifunctional flood defence is elaborated and the different points of attention are pointed out. This paragraph is divided into two subparagraphs, each containing respectively the construction phases and the construction time.

5.5.1 Construction phases

The construction of the multifunctional flood defence has to be outside the storm season (from the first of October until the first of April) because during this season the most severe storms occur. The flood defences have to be at full strength to be able to withstand the most severe storms during this period, so construction is not allowed. This leaves a construction period of six months to complete the multifunctional flood defence. There are seven different phases distinguished which every part has to go through. Six of the seven phases are presented in Figure 5-20.

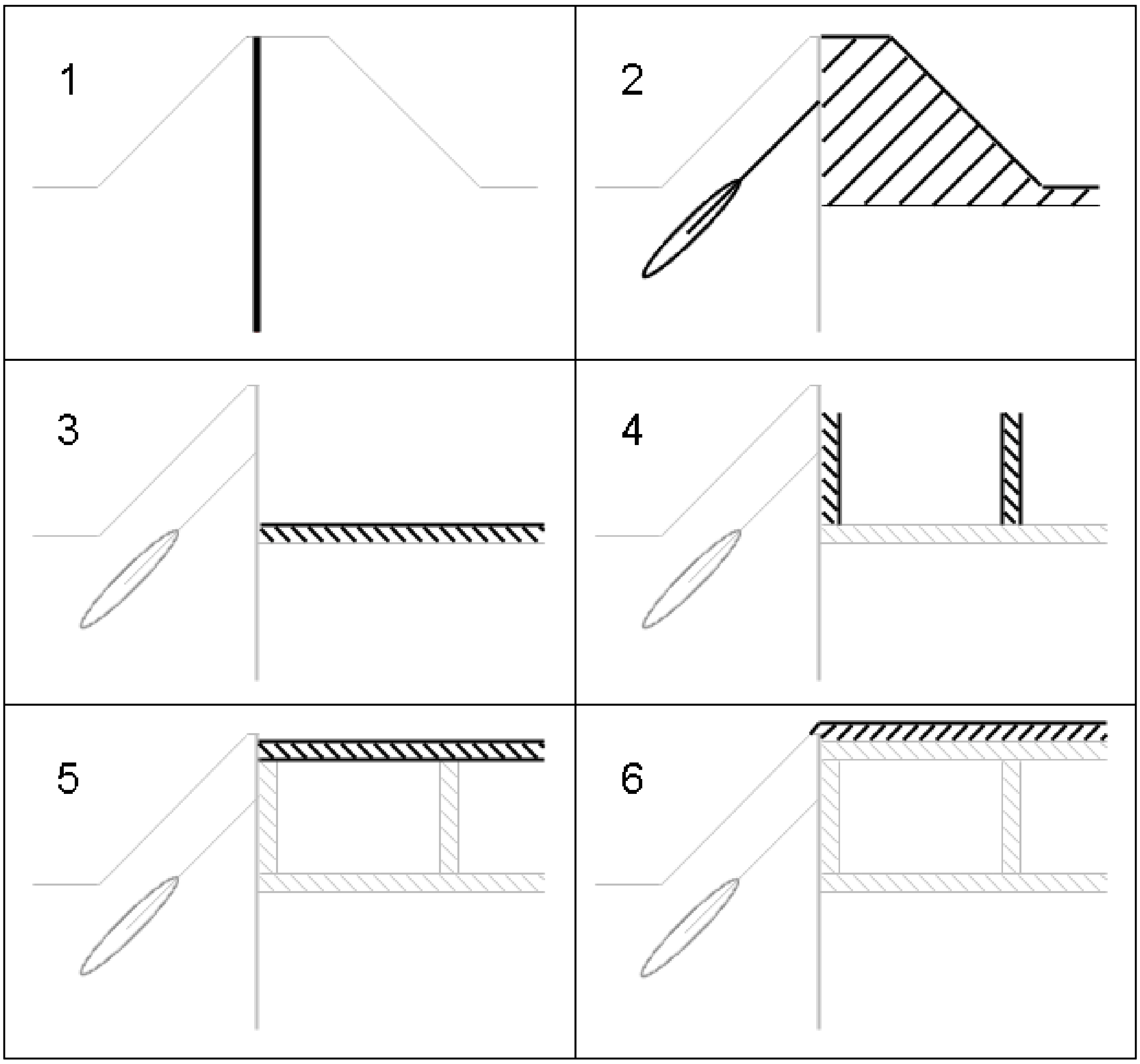

Figure 5-20: Construction phases of the multifunctional flood defence

A brief description of the seven phases and crucial points of each phase are as follows:

- Phase 1:
  The first phase is the installation of the sheet pile wall. The sheet pile wall has two functions (i) temporary flood defence in combination with the remaining soil and (ii) an increase of the seepage length to prevent piping from happening. This phase will start on the first of April, since there is only a six month construction period.
- Phase 2:
  The second phase starts with the excavation of the soil. The sheet pile wall is 5 m high above the excavated surface level, which requires a very stiff sheet pile wall or the application of anchors. The required length of the sheet pile wall below the excavated surface area for the seepage length is smaller than 5 m. The stability of the sheet pile wall requires as a first approximate value (without anchors) twice the height below surface level than above the surface level. This extra required length and stiffness of the sheet pile wall will probably be more expensive than the installation of anchors on a shorter and less stiff sheet pile wall. In this case is therefore chosen for the application of anchors. Phase two is a combination of excavation and installation of anchors because the excavation cannot be done in one go. First the soil is excavated until just below the level at with the anchors are installed, secondly the anchors are installed and after which the soil is further excavated.
- Phase 3:
  The third phase is the making of the bottom slab of the structure. This contains the placing of the formwork and reinforcement steel and after that the casting of the concrete. During this stage is the leakage of water under the sheet pile wall into the building pit an important aspect. Having too much water leaking under the just poured concrete will disrupt the concrete. Also the water pressure under the concrete after hardening of the concrete cannot be too large to prevent lifting of the concrete.
- Phase 4:
  The making of the side walls and columns is the next phase. The same as for the previous phase this holds the placing of the formwork and reinforcement and after that the casting of the concrete.
- Phase 5:
  This phase is the most critical phase of all because this phase will probably take the longest amount of the time. Meaning that all phases starting before this phase will go faster than this phase and all phases starting after this are held up by this phase. Again the same components are obtained; the installation of the formwork and reinforcement and after that the casting of the concrete.
- Phase 6:
  This is the last phase for the completion of the flood defence; backfilling of the soil layer on top of the structure. This phase has to be finished before the start of the storm season on the first of October.
- Phase 7:
  The last phase to complete the structure is the finishing of the structure; this holds the installation of the inside the structure. This can be done after the first of October because it does not affect the water retaining function of the flood defence.

5.5.2 Construction time

With a number of assumptions a time schedule is created in order to show that this can be built in the period outside of the storm season. First some reference projects are analysed and presented in Table 5-12.

Table 5-12: Reference projects (BAM, 2012)

Project name	Floors [-]	Area [m^2]	Time [mo.]	Speed [m^2/mo.]
De Gouden Leeuw, Venray	1	14400	12	1200
Orbis Medical Park, Sittard	2	30000	14	2140
Keizer Karelgarage, Nijmegen	2	18000	18	1000
Multifunctional flood defence, Wageningen	1	9000	6	1500

The reference projects do each have a larger area than the design of the multifunctional flood defence and a longer construction time than the maximum of six months. On the other hand, all of these reference projects are constructed below ground level and have a pile foundation. The multifunctional flood defence is constructed at the surface level of the hinterland and has a shallow foundation which results in a shorter construction time. The comparison with the reference projects is done on the building speed (the area divided by the construction time). The three reference projects have a building speed which is in the same order as is required building speed (1500 m^2 per month) for the multifunctional flood defence. This shows that the building should be able to be constructed outside of the storm season. Phase five, the casting of the roof slab, is the slowest phase. With the assumption that one element of the roof slab can be created in one week is the time schedule of Figure 5-21 established. The casting of the bottom slab, columns and walls is faster than of the roof slab, but in order to keep all the concrete work together along the dike are those slowed down. The first two phases of installation of the sheet pile wall, anchors and the excavating of the soil will be carried out in stages in order not to run too far ahead of the other phases. The same holds for the backfilling of the soil layer, this is a much faster process than the casting of the concrete and therefore split up in three stages. The backfilling of the soil layer is the last phases for the completion of the flood defence and is finished before the start of the storm season. Finishing the inside of the structure can be done in the storm season because it does not affect the water retaining function of the flood defence.

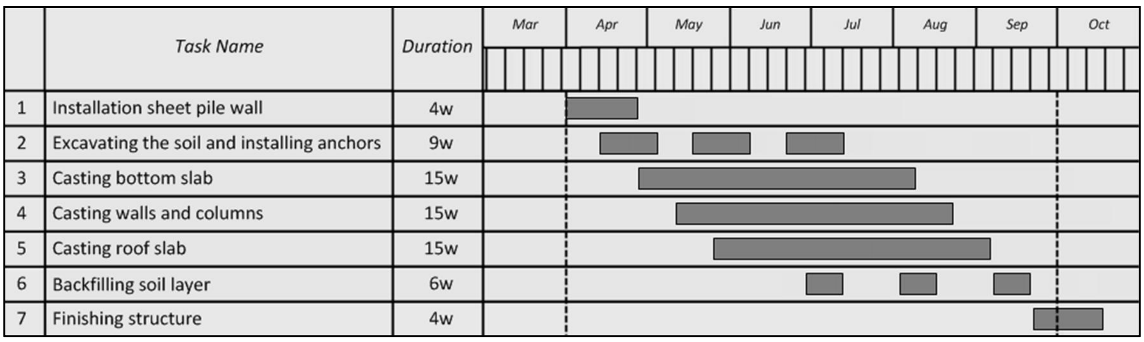

Figure 5-21: Time schedule construction phases

For the construction of the flood defence the structure is divided into elements of 20 m along the dike (15 parts to complete the total length of 300 m). De first step is to start with phase one on element one. The next step is to shift phase one to element the next element and start with phase two on element one. If all phases for all elements have been completed the construction of the multifunctional flood defence is finished. The construction process of the multifunctional flood defence is presented in Figure 5-22. The horizontal axis represents the construction time and the vertical axis represents the elements in which the structure is cut up. The numbers in the graph correspond to the seven distinguished construction phases.

Construction time in weeks

Elements	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
1	1		2		3		4		5					6												7	7	7	7
2	1			2		3		4		5				6												7	7	7	7
3	1			2			3		4		5				6											7	7	7	7
4		1			2			3		4		5			6											7	7	7	7
5		1			2				3		4		5		6											7	7	7	7
6		1						2		3		4		5					6							7	7	7	7
7		1							2		3		4		5				6							7	7	7	7
8			1						2			3		4		5				6						7	7	7	7
9			1							2			3		4		5			6						7	7	7	7
10			1							2				3		4		5		6						7	7	7	7
11			1										2		3		4		5					6		7	7	7	7
12				1										2		3		4		5				6		7	7	7	7
13				1										2			3		4		5				6	7	7	7	7
14				1											2			3		4		5			6	7	7	7	7
15				1											2				3		4		5		6	7	7	7	7

Figure 5-22: Construction process (numbers correspond to the construction phases)

The construction of the flood defence is an important aspect because a delay during construction may result in a crossing of the deadline of October first. Additional research to the construction phases and construction time is required to assure the safety during construction and that the construction time falls within the available time period.

5.6 Changing the dimensions

The dimensions of the structure are varied in order to say something in general about a structure inside dikes. The dimensions can be changed in five different directions; on the landside, on the riverside, in height, in depth and along the alignment of the dike, see Figure 5-23. The design for the Grebbedijk is used as a reference for the comparison with the alternatives with changed dimensions. For each alternative are the advantages, disadvantages and influence on the failure mechanisms described.

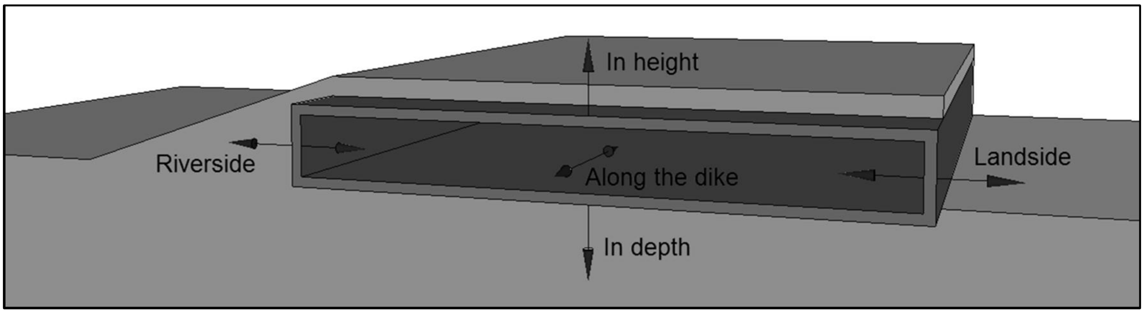

Figure 5-23: Dimensions of the structure which are adjustable

Landside

Lengthening or shortening the structure in the direction of the hinterland has a lot of influence; shortening mostly negative influence, lengthening mostly positive. The width of the structure is highly dependent on the local availability of space and the possibilities to relocate or remove existing buildings. The maximum available amount of space towards the existing building is used for the determination of the width of the structure for the design of the Grebbedijk. In general holds that lengthening the structure towards the hinterland results in a smaller failure probability because: (i) the leakage length is increased, which is positive for the failure mechanism piping and (ii) the horizontal and overturning stability of the structure increase. The effect on the total failure probability depends on the distribution of the failure probability over the failure mechanisms. An effect on a failure mechanism which has a small contribution to the total failure probability is negligible. The contribution of each failure mechanism to the total failure probability is every much depending on the local soil structure, therefore no general conclusions can be stated. Shortening is the opposite of lengthening and has also the opposite effects on the failure mechanisms. The resistance against piping is reduced due to the decreases of seepage length; this can be solved by applying a longer sheet pile. Also the horizontal and overturning stability of the structure are negatively affected by shortening the structure; less shearing resistance and higher soil pressures. This may lead to a pile foundation instead of a shallow foundation as is used in the design for the Grebbedijk. The soil structure plays an important role in the determination of the foundation type. The design is based on a sandy soil structure, which has a relatively high bearing capacity and shearing resistance. Another location has different soil properties which may result in the application of a pile foundation. A pile foundation of the structure results in other problems for the flood defence. The most important difference between a structure with a shallow foundation and a pile foundation is the unequal settlements the pile foundation introduces; the soil around the structure will settle more than the structure itself. This leads to less soil pressure which reduces the resistance against piping. Also the connection between the multifunctional flood defence and the traditional dike experiences the settlement differences. There can be concluded that having a lengthening of the structure in the landward direction has a positive influence on the failure probability of the flood defence. A shortening has a negative influence on the failure probability of the flood defence, additional measures should be done in order to have the same failure probability.

Riverside

The same options are possible on the riverside as on the landside; lengthening and shortening. The same influence on the failure mechanisms can be obtained, so lengthening has a positive effect on the failure probability and shortening a negative effect. In addition to the influence on the failure mechanisms there are two other issues identified. The first issue is the lengthening of the structure on the river side, which holds that the river is narrowed at that part. Especially in the Netherlands relocating the flood defences towards the river is not accepted, flood defence are moved landwards to create more space for the rivers (Ruimte voor der Rivier). The other issue is of a practical nature; shorting the structure on the riverside (i.e. moving the wall on the river side towards the land) starts separating the functions of the flood defence again. Moving the flood defence landwards as a whole does not lead to a different design but during construction the existing flood defence can fulfil the water retaining function while the future flood defence is being built, see Figure 5-24. This ensures the flood safety during construction but it has to be possible in local surroundings.

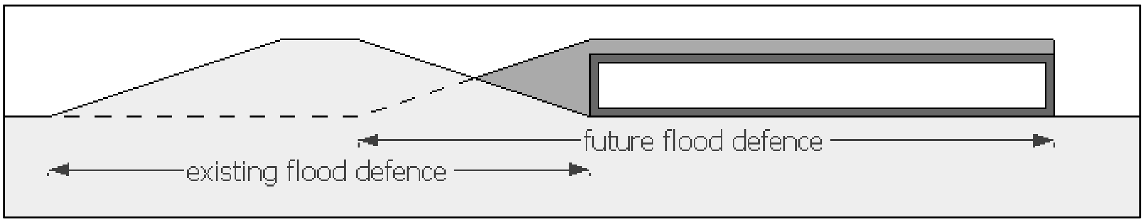

Figure 5-24: Location of the existing and future flood defence

It can be concluded that lengthening or shortening of the structure on the riverside does lead to the same consequences as lengthening or shortening on the landside. Relocating the flood defence from the existing location can be considered but is very much depending on the specific location. Spatial aspects play most times a more important role in that case.

In height

The design of the multifunctional flood defence for the Grebbedijk is completely situated below ground. Increasing the height of the structure leads to the protrusion of the structure out of the dike. The addition of extra floor(s) on the structure leads to higher forces in the structure and on the foundation of the structure, probably leading to a pile foundation instead of a shallow foundation. The effects of having a pile foundation instead of a shallow foundation are described in the previous section and the solution is a larger sheet pile wall. The protrusion of the crest by the structure results in interfaces between concrete and grass cover, which are sensitive to erosion in case of overtopping. The sensitivity to erosion in case of overtopping can be translated to a lower allowable amount of overtopping resulting in a slightly larger required crest height for the flood defence. The structure above the ground level is also subjected to other forces than the water and soil pressure, for instance the wind force, see Figure 5-25. The design of the structure is completely located below ground which makes it roughly only suitable for a garage. The protrusion of the structure out of the dike also allows other functions to be assigned to the structure which makes this a very interesting option.

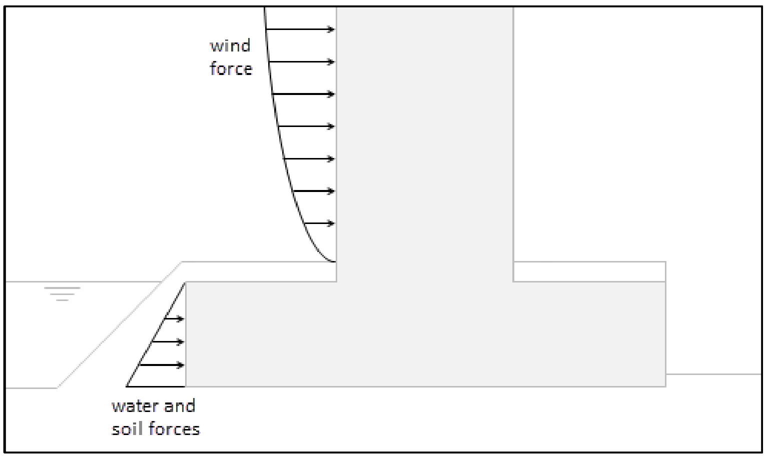

Figure 5-25: Structure on top of the design

In depth

Multiple floors can be added on top of the structure but also below the structure. Instead of having quite a large length of the structure along the dike it is possible to have a shorter structure with multiple layers. The seepage length is increased due to the multiple floors, which is positive for the piping failure mechanism. Also the horizontal and overturning stability are also both influenced in a positive way. A structure that is constructed deeper in the ground experiences more horizontal shearing resistance because of the passive soil pressure and more resistance against overturning by the surrounding soils. The structure experiences larger forces resulting from the water and soil pressure, which can be solved by increasing the dimensions of the elements of the structure. The disadvantage of having a deeper structure is that it probably increases the construction time. On the other hand it may allow the temporary structure of the building pit to fulfil the water retaining function during construction. The bottom of the structure is in the design more or less equal to the ground water level of the hinterland. A deeper structure experiences higher water pressures which might result into leakage problems.

Along of the dike

The length of the structure along the dike is 300 m. The structure is very oblong in order to create the necessary surface area of the garage because there is chosen for a structure with only one floor and the width of the structure is restricted by the surroundings. Significant lengthening of the structure in the direction of the dike results in a construction time longer than the available construction period of six months. The structure cannot be built in one piece; it requires expansion joints every few meters which are sensitive parts of the structure. The expansion joints are at a fixed mutual distance, so lengthening or shortening of the structure does not change the ratio of joints over length and therefore does not affect the failure probability. The length of the flood defence plays also a role in the total failure probability of the flood defence system, the length effect, which is also obtained for traditional flood defences like dikes. The ratio between the failure probability of the adjacent flood defences and of the multifunctional flood defence determines whether it is favourable to have a longer or shorter multifunctional flood defence. If the failure probability of the multifunctional flood defence is smaller than the of the adjacent flood defences it is favourable to have a longer multifunctional flood defence and vice versa.

Conclusion

Changing the dimensions in the riverward and landward direction is very much depending on the availability of space but in general can be stated that a structure with a larger width has a lower probability of failure and a smaller width might require additional measures to ensure the flood safety but does not introduce insurmountable problems. The length of the structure along the dike is not limited in a structural way but only by the available construction period. The most interesting dimensions to change are in depth and in height. Increasing the height of the structure allows other functions for the structures. The main disadvantage of the increasing in height is that the structure probably requires a pile foundation. Having a pile foundation decreases the resistance against piping so extra measures are required to increase the resistance against piping, which can be done by applying a longer sheet pile wall. The construction time is also the constraining aspect for enlarging the structure in depth or height.

5.7 Conclusion

The construction of the design is a critical point in the design of multifunctional flood defence in case of constructing inside the present flood defences; the construction time is limited to the summer period of six months. The design for the multifunctional flood defence is analysed and there has been shown that the construction time is within the available six month period. During construction there has to be paid close attention to the time schedule in order to stay within the available time period.

Changing the dimensions of the structure is in principle possible in all directions. Changing in landward of riverward direction is very much depending on the available space at the specific location. The most interesting dimensions to change are in height or in depth. Increasing the height of the structure allows other functions for the structures because the structure will protrude out of the dike. The construction time is a very important aspect for increasing in height as well as in depth; it has to be within the available construction period of six months.

The purpose of the case study is to identify the issues and differences with the multifunctional flood defences. The main problem is how to deal with the hydraulic loads in the assessment of the overall stability of the structure and the strength of structural elements. Apply large partial factors over design water levels with very small exceedance probabilities seems to conservative. The failure probability has to be calculated to verify the reliability of this design. This is done in the next chapter with using a first order reliability method (FORM) analyses.

6 Reliability analyses

The reliability of the semi probabilistic approach with the use of partial factors is questioned in the previous chapter. The case study of the Grebbedijk showed that the horizontal stability, overturning stability and the strength of the wall are the three failure mechanisms for which there is a lot of uncertainty about which partial factors to be used and the resulting reliability that is reached with those partial factors. In order to analyse the reliability of the structural stability and strength, a first order reliability method (FORM) is carried out. These calculations are performed with the Maple and validated with VaP both are presented in appendix D.

The first paragraph of this chapter describes the three topics addressed in this chapter. The second paragraph presents the FORM analysis used to obtain the results for the three topics. In next three paragraphs are for each failure mechanisms (horizontal stability, overturning stability and strength wall) the three topics treated after which a conclusion is presented in the last paragraph.

6.1 Description of the topics

This chapter addresses the following three topics:

- Design verification with a reliability analyses

The first topic is to verify whether the design made for the Grebbedijk meet the target failure probability and whether using both partial factors and as well as design water levels introduces an excessive amount of reliability into the calculation. The reliability indexes and corresponding failure probabilities are calculated by using a first order reliability method (FORM). This probabilistic approach uses a performance function in which each parameter has a certain distribution instead of a fixed deterministic value. With these distributions the most probable points (design points) and the reliability index are calculated.

- Calibration of the partial factors

The second topic is the calibration of the partial factors. The partial factors can be obtained by comparing this design points of the parameters to the values of those parameters used in the semi probabilistic approach; dividing the two values results in the partial factors.

$$\gamma = \frac{F\left(X_1^*, \ \dots, X_n^*\right)}{F\left(X_1, \ \dots, X_n\right)} \begin{array}{l} \leftarrow \text{design points} \\ \leftarrow \text{semi prob. values} \end{array}$$

This is done for the design points gained from the FORM analysis which is used for the design verification, as well as the design points gained from the FORM analysis with the target reliability. The partial factors used in the semi probabilistic approach are then compared to the ones obtained from the two FORM analyses.

- Reliability based design

The third topic is a look back to the semi probabilistic approach with the results of the FORM analyses. The calibrated partial factors obtained from the previous topic are analysed and there is researched whether it is necessary to use partial factors at all or only on the forces which are not related to the water level. Not using partial factors on the forces related to the water level solves another problem as well. The water force on the structure depends on the height of the structure, the higher the structure the higher the water force. Applying a partial factor on the force of a higher structure would then result in a higher water level used in the calculation than for a structure which is less high, see Figure 6-1

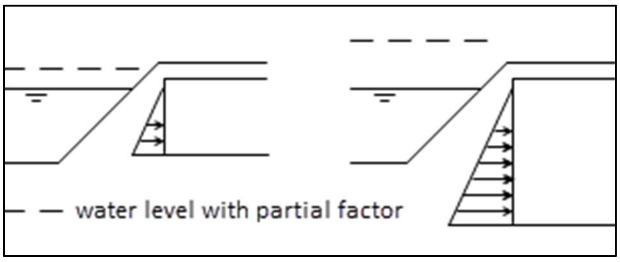

Figure 6-1: Influence of the height of the structure on the applied water level with partial factor

In order to introduce the same amount of reliability into the calculations, the same water level should be used not depending on the height of structure. Which water level has to be used in the calculations is researched within this third topic.

6.2 FORM analysis

The general theory of the FORM analysis is briefly explained in paragraph 4.2.2, but here explained in more detail in order to have a better understanding of the calculations that are performed. The general purpose of the FORM analysis is to obtain the reliability index, which is directly related to the failure probability. The reliability index is calculated by dividing the mean value of the performance function by the standard deviation of the performance function.

$$\beta = \frac{\mu_z}{\sigma_z} \quad \rightarrow \quad P_f = \Phi(-\beta)$$

Where:

β reliability index
P_f failure probability
μ_z mean value of the Z function
σ_z standard deviation of the Z function
$\Phi(..)$ standard normal distribution

The mean value of a non-linear Z function can be approximated by the first two terms of the Taylor-polynomial of the Z function, which is normally distributed according to the central limit theorem. The first term is equal to the value of the performance function in the design point. The second term is the summation of the partial derivatives multiplied by the difference between the mean value and the design value of each parameter.

$$\mu_z = Z(X_1, \dots, X_n) + \sum_{i=1}^{n} \left(\frac{\partial Z(X_1, \dots, X_n)}{\partial X_i} \cdot (\mu_{X_i} - X_i) \right)$$

The standard deviation is than equal to the square root of the summation of the square of the partial derivatives multiplied by the standard deviation of each parameter.

$$\sigma_z = \sqrt{\sum_{i=1}^{n} \left(\frac{\partial Z(X_1, \, \dots \, , X_n)}{\partial X_i} \cdot \sigma_{x_i} \right)^2}$$

For the calculation of the mean value and standard deviation a certain design point is required. This design value is determined by adding or subtracting (depending on the sign of α) a factor times the standard deviation. This factor is the multiplication of the reliability index and the influence factor of each parameter.

$$X_1^* = \mu_i + \alpha_i \cdot \beta \cdot \sigma_i$$

The influence factor is calculated by dividing the multiplication of the partial derivative and the standard deviation of each parameter by the standard deviation of the performance function.

$$\alpha_i = -\frac{\dfrac{\partial Z\left(X_1^*, \, \dots \, , X_n^*\right)}{\partial X_i} \cdot \sigma_{x_i}}{\sqrt{\sum_{i=1}^{n} \left(\dfrac{\partial Z\left(X_1^*, \, \dots \, , X_1^*\right)}{\partial X_i} \cdot \sigma_{x_i} \right)^2}}$$

The calculation of the design point is depending on: (i) the influence factors, (ii) the mean value of the performance function and (iii) the standard deviation of the performance function. Those three parameters are calculated with the use of the design point again, so it is an iterative calculation. The first estimations of the design points are the mean values of the parameters, after a couple of iteration a constant outcome of the calculation is achieved.

The FORM analysis assumes that all parameters have a normal distribution. Of course not all parameters will have a normal distribution. The three considered failure mechanisms have the water level included in the performance function, which has a Gumbel distribution. The Gumbel distribution needs to be transformed to a normal distribution in order to use the FORM analysis. The transformation from a Gumbel distribution to a normal distribution is done in the design point and needs to be calculated for every iteration because the design point is different each iteration (Rackwitz et al., 1977).

$$\sigma = \frac{\varphi\left(\Phi^{-1}\left(F\left(X^*\right)\right)\right)}{f\left(X^*\right)}$$

$$\mu = X^* - \Phi^{-1}\left(F\left(X^*\right)\right) \cdot \sigma$$

Where:

σ standard deviation
μ mean value
Φ^{-1} inverse cumulative standard normal distribution function
φ standard normal probability density function
F arbitrary cumulative distribution function
f arbitrary probability density function
X^* design point

The design points of the parameters with another distribution are calculated in a different way than the parameters with a normal distribution, they need to be transformed back to their original distribution in order to acquire the design point.

$$X^* = F^{-1}\left(\Phi\left(\alpha \cdot \beta\right)\right)$$

Where:

X^* design point
F^{-1} arbitrary inverse cumulative distribution function
Φ standard normal cumulative distribution function
α influence factor
β reliability index

The Gumbel distribution has a location and a scale parameter that can be obtained from two water levels with two different exceedance frequencies. For the Grebbedijk two water levels are given by the water board (WVE, 2012). The Gumbel distribution can be obtained by plotting a linear trend line through the two water levels on a semi logarithmic scale, see Figure 6-2.

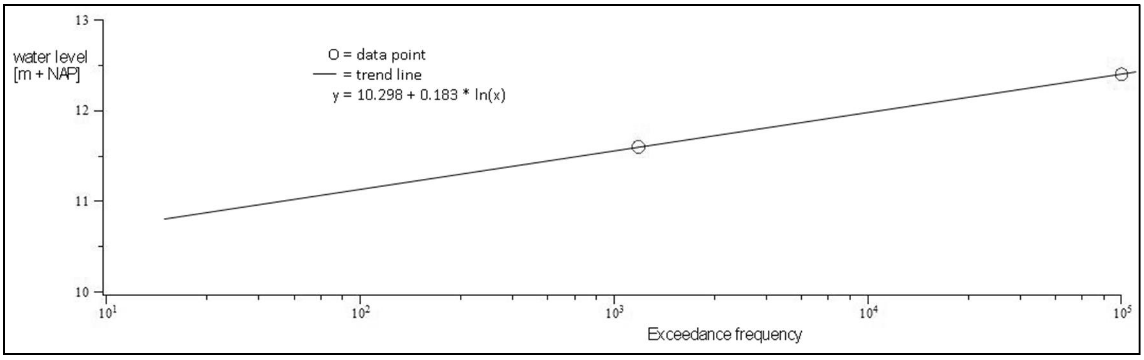

Figure 6-2: Gumbel distribution of the water level

This resulted in the following location and scale parameter:

α = 10.429 location parameter
β = 0.163 scale parameter

The location parameter is a value corresponding to NAP; this should be corrected to the bottom level of the structure to be able to obtain the water height which results in a force on the structure, see Figure 6-3.

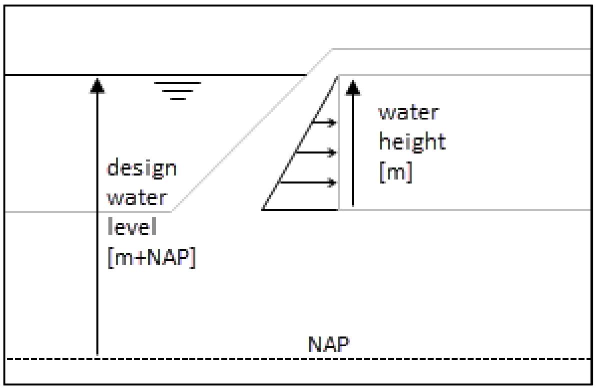

Figure 6-3: Design water level and water height

The bottom of the structure is at 8 m+NAP, resulting in:

α $= 2.429$ location parameter
β $= 0.163$ scale parameter

The distribution of the ground water level inside the dike can be determined, now the distribution of the outer water level is known. In order to simplify the calculations it is assumed that the ground water level inside the dike equals the water level of the river. This is a very conservative assumption because the ground water level will be less extreme than the water level at the river due to time dependent effects and a decrease of the phreatic line inside the dike.

The design point is reached after a couple of iterations. Figure 6-4 shows the Gumbel distribution, the schematised normal distributions and the corresponding design points. As a first estimate of the design point, the mean value of the Gumbel distribution is taken. As can be seen in the figure, the design point goes to the final design point within five iterations. The design point of the water level has a very low probability of occurrence; this is because the water level is the main force driving the horizontal stability.

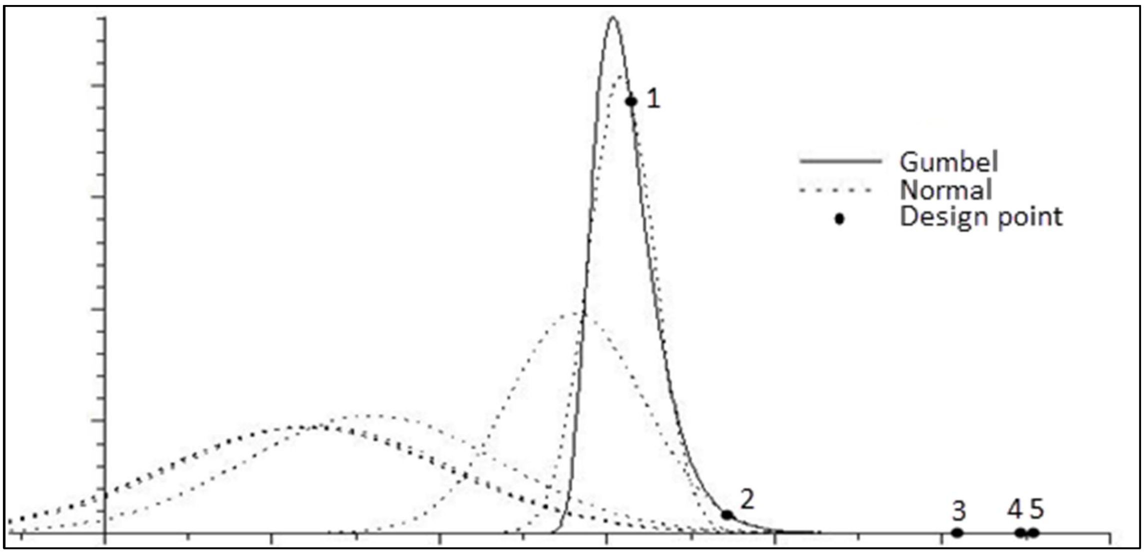

Figure 6-4: Transformation from a Gumbel to a normal distribution

The other parameters have assigned a normal distribution, which can be used directly in the FORM analysis. The mean values and the standard deviations of the normally distributed parameters are presented in the paragraph on the failure mechanisms.

6.3 Horizontal stability

The horizontal stability is the stability against shearing of the entire structure. A performance function is established containing several variables for the FORM analysis. All the variables are then given a certain distribution type and the corresponding distribution parameters. The output of the FORM analysis is the design points and influences factors of the parameters and the reliability index and failure probability of the performance function.

6.3.1 Design verification

Performance function

The performance function consists of a load and strength part. The loads on the structure are the water and soil pressure and the strength part is the friction force between the structure and the soil, which is caused by the sum of the vertical forces, in formula:

$$Z = R - S$$
$$R = f \cdot \Sigma V$$
$$S = \Sigma H$$
$$f = \tan\left(\frac{2}{3} \cdot \varphi\right)$$

Where:

Z	performance function
R	resistance
S	load
f	coefficient of friction
ΣV	sum of the vertical forces
ΣH	sum of the horizontal forces
φ	friction angle

The horizontal forces and vertical forces that are presented in Figure 6-5 are described as follows:

$$F_{h,wat} = 0.5 \cdot \gamma_w \cdot h^2$$
$$F_{h,soil} = 0.5 \cdot (\gamma_{s1} - \gamma_w) \cdot (H + d_s)^2 \cdot K_a$$
$$F_{v,conc} = (2 \cdot d_w \cdot H + d_r \cdot L + d_b \cdot L) \cdot \gamma_c$$
$$F_{v,soil} = L \cdot d_s \cdot \gamma_{s2}$$
$$F_{v,wat} = 0.5 \cdot \gamma_w \cdot h \cdot L$$

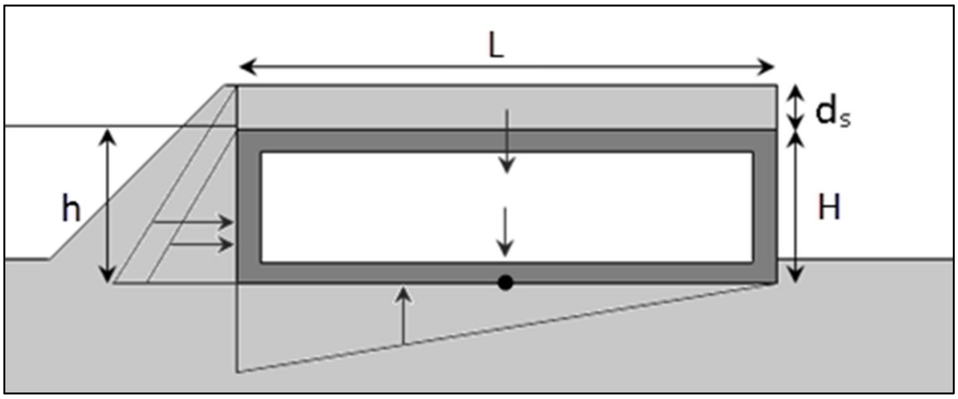

Figure 6-5: Acting forces for the horizontal stability

Results in the following performance function:

$$Z = f \cdot \left(F_{v,conc} + F_{v,soil} - F_{v,wat}\right) - \left(F_{h,wat} + F_{h,soil}\right)$$

Some forces and effects are neglected in order to simplify the calculation in a conservative way, for instants the resistance against shearing of the soil on the right side of the structure. Also the drop in water pressure under the structure due to the sheet pile wall is neglected.

Input variables

The performance function derived in the previous paragraph is a function of eleven parameters with each its own characteristics and corresponding distribution types. Six of the eleven parameters are dimension parameters like the length and width of the structure, three of the parameters are the specific weights of concrete, water and soil, one is the friction angle of the subsoil and one is the water height. All the parameters are described with a normal distribution, except the water height. The mean values and the standard deviations of the normal distributed parameters are presented in Table 6-1. The mean values of six dimension parameters are equal to the dimensions of the design. The standard deviation together with the mean values and standard deviations of the specific weights and the friction angle are found in literature (Mai et al., 2006; Molenaar et al., 2008; NEN-EN 1991, 2011).

Table 6-1: Input parameters for the FORM analysis

Parameter	Sym	Unit	Dist	μ	σ	COV
Length structure	L	m	Normal	30.00	0.03	0.001
Height structure	H	m	Normal	4.00	0.03	0.008
Thickness soil layer	d_s	m	Normal	1.00	0.05	0.050
Thickness wall	d_w	m	Normal	0.40	0.01	0.025
Thickness bottom	d_b	m	Normal	0.50	0.01	0.020
Thickness roof	d_r	m	Normal	0.50	0.01	0.020
Specific weight water	γ_w	kN/m^3	Normal	10.00	0.20	0.020
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	18.00	0.90	0.050
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	20.00	1.00	0.050
Specific weight concrete	γ_c	kN/m^3	Normal	25.00	0.50	0.020
Friction angle	φ	°	Normal	32.50	2.00	0.062

μ mean value
σ standard deviation
COV coefficient of variation

Output

The output of the FORM analysis is the influence factors and the design points of the parameters; see Table 6-2. The influence factors represent the importance of the parameter in relation to the performance function. The table shows that the water level is by far the most important parameter of the failure mechanism; all other parameters are of much less important.

Table 6-2: Output of the FORM analysis

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water height	h	m	Gumbel	0.969	5.010	4.0	1.25
Length structure	L	m	Normal	-0.002	30.000	30.0	1.00
Height structure	H	m	Normal	0.009	4.001	4.0	1.00
Thickness soil layer	d_s	m	Normal	-0.084	0.978	1.0	0.98
Thickness wall	d_w	m	Normal	-0.008	0.400	0.4	1.00
Thickness bottom	d_b	m	Normal	-0.029	0.498	0.5	1.00
Thickness roof	d_r	m	Normal	-0.029	0.498	0.5	1.00
Specific weight water	γ_w	kN/m^3	Normal	0.067	10.071	10.0	1.01
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	-0.103	17.507	18.0	0.97
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	0.090	20.481	20.0	1.02
Specific weight concrete	γ_c	kN/m^3	Normal	-0.065	24.828	25.0	0.99
Friction angle	φ	°	Normal	-0.160	30.798	32.5	0.95

α influence factor
X* design value
X value of the semi probabilistic approach
γ partial factor for the parameter

The FORM analysis converges within a couple of iterations towards the final design point, see Table 6-3. The resulting reliability index and failure probability of the failure mechanism are respectively higher and lower than is required.

Table 6-3: Output performance function of the FORM analysis

Iteration	1	2	3	4	5
μ	382	478	508	510	510
σ	70	90	96	96	96
β	5.49	5.32	5.32	5.32	5.32
P_f	2.04E-8	5.27E-8	5.27E-8	5.27E-8	5.27E-8

μ mean value of the performance function
σ standard deviation of the performance function
β reliability index
P_f failure probability

This calculation shows that the design is safe for horizontal shearing. The second step is to use this probabilistic calculation to reflect back on the semi probabilistic calculations. Those semi probabilistic calculations use partial factors and a design water level to achieve the reliability. The probabilistic approach can now be used to calibrate those partial factors.

6.3.2 Calibration of the partial factors

The partial factors are used to implement reliability in the calculations without doing any probabilistic calculations. Those partial factors need to be calibrated in order to be sure that they represent the required amount of reliability. The partial factors can be determined with the design points and the values of the parameters used in the semi probabilistic approach. By dividing the value of the force calculated with the design point over the value of the force calculated in the semi probabilistic approach, the partial factors are obtained.

$$\gamma = \frac{F\left(X_1^*, \ \dots \ ,X_n^*\right)}{F\left(X_1, \ \dots \ ,X_n\right)}$$

Where:

γ partial factor

F force

X_i^* design point of a parameter

X_i value used in the semi probabilistic approach

The reliability corresponding to those partial factors is the reliability calculated in the previous paragraph. However, the reliability calculated in the previous paragraph is higher than required. The probabilistic calculations need to be adjusted to obtain the design points for the target reliability. This is achieved by not calculating the reliability index in the FORM analysis but fixating it at the target reliability. The target reliability is directly obtained via the failure probability; a required failure probability of 1E-6 corresponds with a reliability of 4.75. The procedure of the FORM analysis is not changed, the only difference is that the reliability is not calculated anymore but fixed at the target reliability. The results of this calculation are presented in Table 6-4.

Table 6-4: Output of the FORM analysis (with fixed target reliability)

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water height	h	m	Gumbel	0.953	4.502	4.0	1.13
Length structure	L	m	Normal	-0.003	30.000	30.0	1.00
Height structure	H	m	Normal	0.011	4.002	4.0	1.00
Thickness soil layer	d_s	m	Normal	-0.097	0.977	1.0	0.98
Thickness wall	d_w	m	Normal	-0.009	0.400	0.4	1.00
Thickness bottom	d_b	m	Normal	-0.034	0.498	0.5	1.00
Thickness roof	d_r	m	Normal	-0.034	0.498	0.5	1.00
Specific weight water	γ_w	kN/m^3	Normal	0.065	10.062	10.0	1.01
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	-0.119	17.489	18.0	0.97
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	-0.106	20.504	20.0	1.03
Specific weight concrete	γ_c	kN/m^3	Normal	-0.075	24.822	25.0	0.99
Friction angle	φ	°	Normal	-0.212	30.486	32.5	0.94

α influence factor

X* design value

X value of the semi probabilistic approach

γ partial factor for the parameter

The influence factors calculated in different design points can vary a lot, resulting in different partial factor for the parameters. The influence factors from the original FORM analysis and the FORM analysis with a fixed reliability index are presented in Figure 6-6. It can be concluded from this figure that the difference is not significant between the influence factors. So the importance is not relocated to other parameters due to the use of another reliability index.

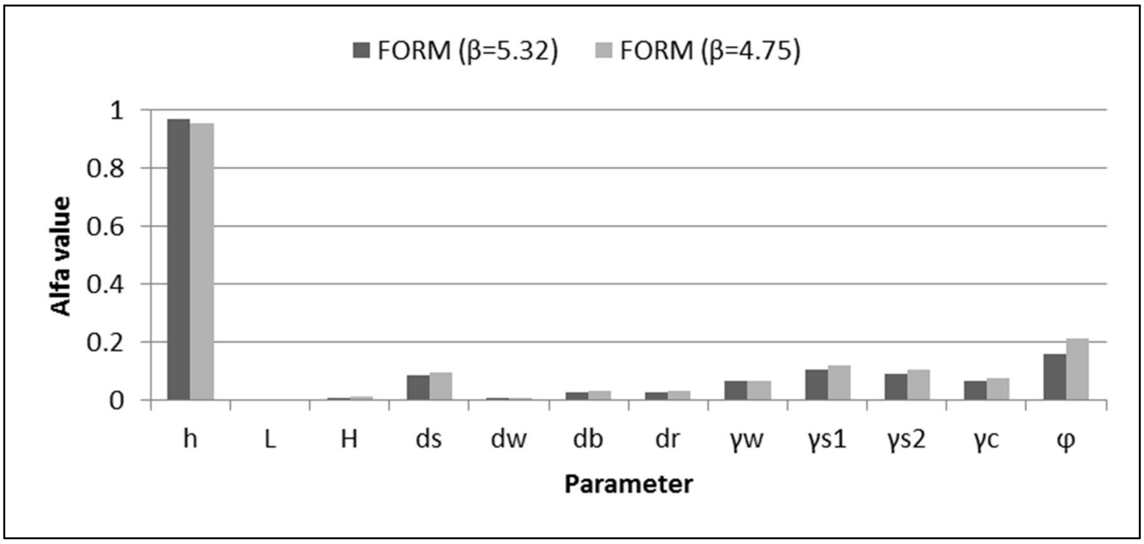

Figure 6-6: Influence factors for the FORM analyses with and without a fixed reliability influence

The partial factors of the parameters can be calculated from the results of the FORM analysis with and without the fixed reliability index. The partial factors for the forces are calculated by dividing the forces calculated with design points over the forces calculated with the characteristic values.

$$\gamma_{F_{h,wat}} = \frac{0.5\cdot\gamma_w^*\cdot h^{*2}}{0.5\cdot\gamma_w\cdot h^2}$$

$$\gamma_{F_{h,soil}} = \frac{0.5\cdot\left(\gamma_{s1}^*-\gamma_w^*\right)\cdot\left(H^*+d_s^*\right)^2\cdot K_a}{0.5\cdot\left(\gamma_{s1}-\gamma_w\right)\cdot\left(H+d_s\right)^2\cdot K_a}$$

$$\gamma_{F_{v,conc}} = \frac{\tan\left(\frac{2}{3}\cdot\varphi^*\right)\cdot\left(2\cdot d_w^*\cdot H^*+d_r^*\cdot L^*+d_b^*\cdot L^*\right)\cdot\gamma_c^*}{\tan\left(\frac{2}{3}\cdot\varphi\right)\cdot\left(2\cdot d_w\cdot H+d_r\cdot L+d_b\cdot L\right)\cdot\gamma_c}$$

$$\gamma_{F_{v,soil}} = \frac{\tan\left(\frac{2}{3}\cdot\varphi^*\right)\cdot L^*\cdot d_s^*\cdot\gamma_{s2}^*}{\tan\left(\frac{2}{3}\cdot\varphi\right)\cdot L\cdot d_s\cdot\gamma_{s2}}$$

$$\gamma_{F_{v,wat}} = \frac{\tan\left(\frac{2}{3}\cdot\varphi^*\right)\cdot 0.5^*\cdot\gamma_w^*\cdot h^*\cdot L^*}{\tan\left(\frac{2}{3}\cdot\varphi\right)\cdot 0.5\cdot\gamma_w\cdot h\cdot L}$$

The partial factors calculated with both FORM analyses and the ones used in the semi probabilistic approach are presented in Table 6-5 together with the failure probabilities.

Table 6-5: Partial factors for the three considered cases

Parameter	Sym	Case 1	Case 2	Case 3
Horizontal water pressure	$F_{h,wat}$	1.27	1.58	1.27
Horizontal soil pressure	$F_{h,soil}$	1.20	1.03	1.04
Vertical water pressure	$F_{v,wat}$	1.27	1.19	1.06
Weight of the structure	$F_{v,conc}$	0.90	0.90	0.89
Weight of the soil	$F_{v,soil}$	0.90	0.93	0.92
Failure probability	P_f	<1.0E-6	5.3E-8	1.0E-6

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

The partial factors from the FORM analyses for the forces generating resistance correspond with the partial factors used in the semi probabilistic approach. The difference is obtained in the partial factors for the action forces. The partial factor for the horizontal soil pressure is almost equal to one. The partial factor for the water pressure under the structure is lower than the partial factor used in the semi probabilistic approach. This relates to the fact that the water pressure is not so much an action force, but a force reducing the resistance; the influence of the friction angle reduces the partial factor. The horizontal water pressure has the largest partial factor in all cases, meaning that it is the most important force for the failure mechanism horizontal shearing. Three of the partial factors from the third case (with the fixed reliability index) have more or less the same value as the partial factors used in the semi probabilistic approach, the other two partial factors have lower values than in the semi probabilistic approach. So it can be concluded that the semi probabilistic approach has a failure probability which is smaller than the failure probability of the FORM analysis with a fixed reliability index, so smaller than 5.3E-8.

6.3.3 Reliability based design

The design water level is further investigated since it is so important in the calculation of the reliability index and corresponding failure probability. The design point of the water height corresponds to a design water level referred to NAP with an exceedance probability, see Table 6-6. Also the failure probability for the two cases done with the FORM analysis is presented in the same table.

Table 6-6: Partial factors for the three considered cases

Parameter	Case 1	Case 2	Case 3	Unit
Design point water height	4.0	5.0	4.5	m
Design water level	12.0	13.0	12.5	m+NAP
Exceedance probability	1.0E-4	1.3E-7	3.0E-6	1/year
Failure probability	<1.0E-6	5.3E-8	1.0E-6	1/year

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

The failure probability of this failure mechanism is for case 2 and 3 in the order of the exceedance probability of the water level but slightly smaller. So applying a design water level with an exceedance probability equal to the target failure probability will result in an actual failure probability slightly smaller than the exceedance probability. From the FORM analysis is already found that the design water level is the most important parameter in the calculation of the reliability of the failure mechanism. Using a design water level with an exceedance probability equal to the target failure probability does not require the use of partial factors for the forces resulting from water pressure. This holds only for horizontal stability, the next paragraph researches if this is also the case for overturning stability.

6.4 Overturning stability

The overturning stability of the structure is the prevention of tumbling of the structure as a result of forces generating moments with respect to a certain rotation point. This paragraph treats the FORM analysis on the overturning stability in the same way the horizontal stability is treated in the previous paragraph.

6.4.1 Design verification

Performance function

The total force following from the forces acting on the structure has to be within a certain distance from the centre point under the structure. That distance of that force with respect to the centre point can be calculated by dividing the summation of the moments acting on the structure by the summation of the vertical forces acting on the structure. The same forces as for the horizontal stability are obtained. Two of the forces (weight of the structure and weight of the soil) have their work line crossing the centre point, i.e. no generation of moment from those forces. The other forces do not cross the centre point and are generating a moment.

$$\frac{\Sigma M}{\Sigma V} < \frac{1}{6} \cdot L$$

$$Z = R - S$$

$$R = \frac{1}{6} \cdot L$$

$$S = \frac{\Sigma M}{\Sigma V}$$

Where:

Z	performance function
R	resistance
S	load
ΣM	sum of the moments
ΣV	sum of the vertical forces
L	length of the structure

As said before the forces are equal to the forces used in the horizontal stability, see Figure 6-7. The moments generated by those forces are all in the same direction, i.e. no forces generating counteracting moments.

$$M_{h,wat} = \frac{1}{3} \cdot h \cdot F_{h,wat}$$

$$M_{h,soil} = \frac{1}{3} \cdot H \cdot F_{h,soil}$$

$$M_{v,wat} = \frac{1}{6} \cdot L \cdot F_{v,wat}$$

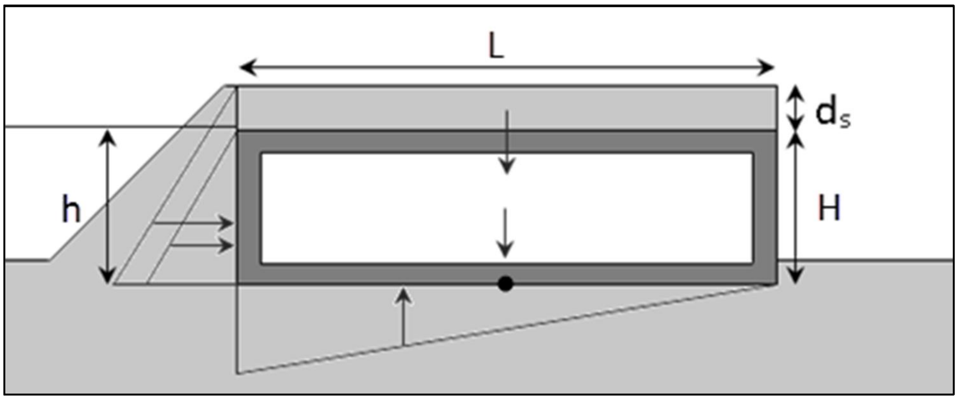

Figure 6-7: Acting forces for the overturning stability

Input variables

The parameters are the same as for the horizontal stability, except the friction angle is not present in the performance function. The chosen distributions with the corresponding distribution parameters are the same as for the horizontal stability, but are presented in Table 6-7 for completeness.

Table 6-7: Input parameters for the FORM analysis

Parameter	Sym	Unit	Dist	μ	σ	COV
Length structure	L	m	Normal	30.00	0.03	0.001
Height structure	H	m	Normal	4.00	0.03	0.008
Thickness soil layer	d_s	m	Normal	1.00	0.05	0.050
Thickness wall	d_w	m	Normal	0.40	0.01	0.025
Thickness bottom	d_b	m	Normal	0.50	0.01	0.020
Thickness roof	d_r	m	Normal	0.50	0.01	0.020
Specific weight water	γ_w	kN/m^3	Normal	10.00	0.20	0.020
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	18.00	0.90	0.050
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	20.00	1.00	0.050
Specific weight concrete	γ_c	kN/m^3	Normal	25.00	0.50	0.020

μ mean value
σ standard deviation
COV coefficient of variation

Output

The output of the FORM analysis is presented in the same way as for the horizontal stability, see Table 6-2. The same effect is identified for the overturning stability as for the horizontal stability; the water height is the most important parameter.

Table 6-8: Output of the FORM analysis

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water height	h	m	Gumbel	0.983	4.708	4.0	1.18
Length structure	L	m	Normal	-0.000	30.000	30.0	1.00
Height structure	H	m	Normal	-0.000	4.000	4.0	1.00
Thickness soil layer	d_s	m	Normal	-0.095	0.977	1.0	0.98
Thickness wall	d_w	m	Normal	-0.007	0.400	0.4	1.00
Thickness bottom	d_b	m	Normal	-0.028	0.499	0.5	1.00
Thickness roof	d_r	m	Normal	-0.028	0.499	0.5	1.00
Specific weight water	γ_w	kN/m^3	Normal	0.095	10.092	10.0	1.01
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	-0.098	17.569	18.0	0.98
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	0.011	20.052	20.0	1.00
Specific weight concrete	γ_c	kN/m^3	Normal	-0.062	24.850	25.0	0.99

α influence factor
X* design value
X value of the semi probabilistic approach
γ partial factor for the parameter

The resulting reliability and corresponding failure probability are respectively higher and lower than required. The required failure probability is equal to 1E-6 and the calculated failure probability is in the order of 1E-7.

Table 6-9: Output performance function of the FORM analysis

Iteration	1	2	3	4	5
μ	5.41	12.502	9.66	9.33	9.33
σ	1.01	2.54	1.98	1.92	1.91
β	5.34	4.93	4.87	4.87	4.87
P_f	4.72E-8	4.04E-7	5.50E-7	5.53E-7	5.53E-7

μ mean value of the performance function
σ standard deviation of the performance function
β reliability index
P_f failure probability

This calculation shows that the design is safe for overturning stability. The second step is to use this probabilistic calculation to reflect back on the semi probabilistic calculations. Those semi probabilistic calculations use partial factors and a design water level to achieve the reliability. The probabilistic approach can now be used to calibrate those partial factors.

6.4.2 Calibration of the partial factors

In the same way as is done for the horizontal stability, the partial factors are calculated for the overturning stability. The failure probability of this failure mechanism is also adjusted to the target failure probability in order to obtain the partial factors which need to be used to achieve that target failure probability. The results of the adjusted FORM analysis are presented in Table 6-10.

Table 6-10: Output of the FORM analysis (with fixed target reliability)

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water height	h	m	Gumbel	0.983	4.614	4.0	1.15
Length structure	L	m	Normal	-0.000	30.000	30.0	1.00
Height structure	H	m	Normal	-0.000	4.000	4.0	1.00
Thickness soil layer	d_s	m	Normal	-0.095	0.977	1.0	0.98
Thickness wall	d_w	m	Normal	-0.007	0.400	0.4	1.00
Thickness bottom	d_b	m	Normal	-0.028	0.499	0.5	1.00
Thickness roof	d_r	m	Normal	-0.028	0.499	0.5	1.00
Specific weight water	γ_w	kN/m^3	Normal	0.095	10.090	10.0	1.01
Specific weight soil (clay)	γ_{s1}	kN/m^3	Normal	-0.099	17.579	18.0	0.98
Specific weight soil (sand)	γ_{s2}	kN/m^3	Normal	0.011	20.053	20.0	1.00
Specific weight concrete	γ_c	kN/m^3	Normal	-0.062	24.853	25.0	0.99

α influence factor
X* design value
X value of the semi probabilistic approach
γ partial factor for the parameter

The influence factors of both FORM analyses are presented in Figure 6-6, from which can be seen that the influence factors barely change.

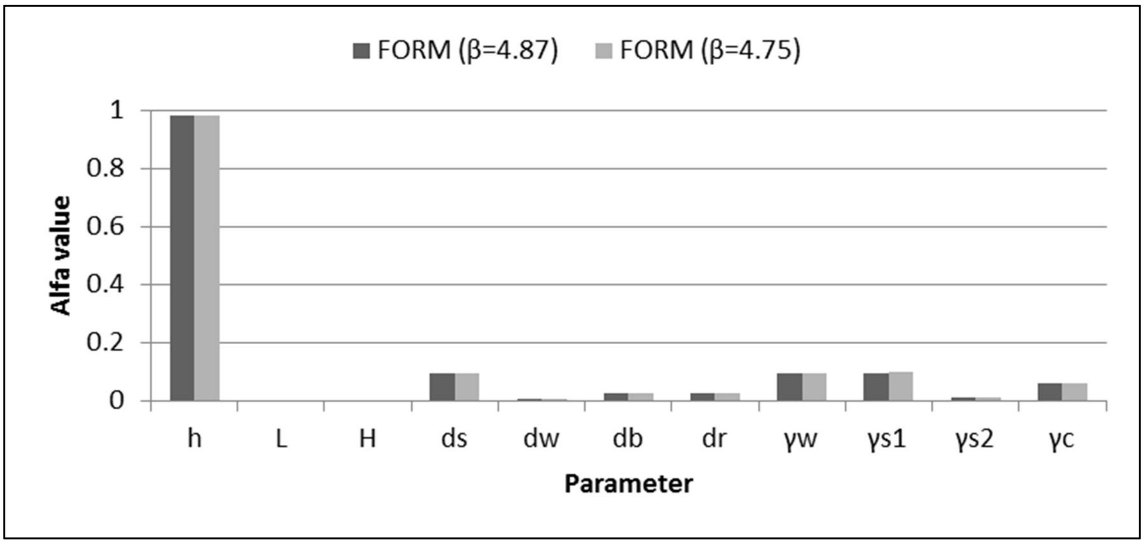

Figure 6-8: Influence factors for the FORM analyses with and without a fixed reliability influence

The calculation of the partial factors follows the same procedure as for the horizontal stability. The magnitude of the force calculated with the design points is divided by the magnitude of the force calculated with the characteristic values to obtain the partial factors. The partial factors and failure probabilities are presented in Table 6-11.

Table 6-11: Partial factors for the three considered cases

Parameter	Sym	Case 1	Case 2	Case 3
Horizontal water pressure	$M_{h,wat}$	1.27	1.65	1.55
Horizontal soil pressure	$M_{h,soil}$	1.20	0.98	0.98
Vertical water pressure	$M_{v,wat}$	1.27	1.19	1.16
Vertical water pressure	$F_{v,wat}$	1.27	1.19	1.16
Weight of the structure	$F_{v,conc}$	0.90	0.99	0.99
Weight of the soil	$F_{v,soil}$	0.90	0.95	0.96
Failure probability	P_f	<1.0E-6	5.5E-7	1.0E-6

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

In the semi probabilistic approach a partial factor is used for the horizontal soil pressure which is higher than one because the force is unfavourable for the overturning stability. The partial factors calculated with the FORM analyses are slightly below one, meaning a reduction of the force. This is caused by two parameters: the height of the structure and the thickness of the soil layer. Both parameters occur as well on the loading side as on the resistance side of the equation. Since both parameters are proportional to the third power of the parameters in the calculation of the horizontal soil pressure it reduces the partial factor below one. It is remarkable that the partial factor for the horizontal water pressure is much higher than all other partial factors. The same phenomenon is seen for the horizontal stability, but much smaller. In both cases is the water height the most important parameter in the FORM analysis, resulting in high partial factors. On the other hand, the vertical water pressure is linearly proportional to the water height resulting in a much lower partial factor. The water height is further researched in the next paragraph since it is so important in this calculation.

6.4.3 Reliability based design

The design water level is further researched in this paragraph to get more insight in the exceedance probability and failure probability. There are three cases compared: (i) the semi probabilistic approach used in chapter 5, (ii) the FORM analysis performed in this chapter and (iii) the FORM analysis with a fixed reliability index also performed in this chapter. The design points calculated are transformed back to the design water levels and the exceedance probabilities of those water levels are calculated and compared to the failure probabilities.

Table 6-12: Partial factors for the three considered cases

Parameter	Case 1	Case 2	Case 3	Unit
Design point water height	4.0	4.7	4.6	m
Design water level	12.0	12.7	12.6	m+NAP
Exceedance probability	1.0E-4	8.3E-7	1.6E-6	1/year
Failure probability	<1.0E-6	5.5E-7	1.0E-6	1/year

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

The exceedance probabilities of the water levels are close to the failure probabilities of the failure mechanism; overturning stability. Using the design water level with an exceedance probability equal to the target failure probability will introduce enough reliability in the calculation, since the exceedance probability of the design water level is larger than the target failure probability. The same is obtained for the horizontal stability, so for both cases the use of a design water level would introduce enough reliability into the design. The strength of structural elements is treated in the next paragraph in order to see whether the same can be applied for the assessment of the strength of structural elements.

6.5 Strength wall

The consideration of the FORM analyses for the stability of the structure led to the insight that applying a design water level with a certain exceedance probability introduces the required amount of reliability into the calculations. The structural strength is researched in a similar way to find out if the same can be applied. The element that experiences the water pressure is the wall.

6.5.1 Design verification

Performance function

Failure of the wall is caused by bending moments acting on the wall exceeding resistance of the wall. The bending moment resistance is generated by the reinforcement steel in the wall; failure of this reinforcement steel means failure of the wall. The moments acting on the wall are the water and soil pressure, see Figure 6-9.

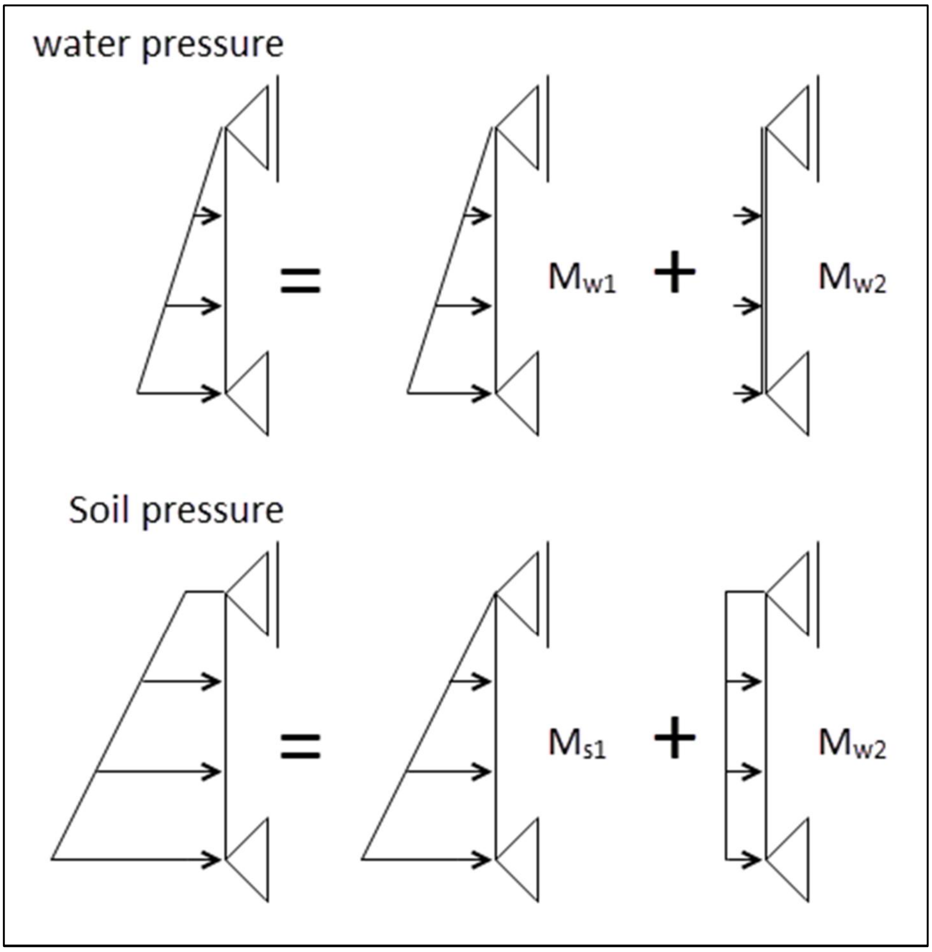

Figure 6-9: Soil and water pressure acting on the wall

$$Z = R - S$$
$$R = A_s \cdot f_s \cdot z$$
$$z = 0.75 \cdot d_w$$
$$S = \Sigma M = M_{w1} + M_{w2} + M_{s1} + M_{s2}$$
$$M_{w1} = 0.064 \cdot h \cdot \gamma_w \cdot H^2$$
$$M_{w2} = \frac{1}{8} \cdot (h - H) \cdot \gamma_w \cdot H^2$$
$$M_{s1} = 0.064 \cdot H \cdot (\gamma_s - \gamma_w) \cdot H^2 \cdot K_a$$
$$M_{s2} = \frac{1}{8} \cdot d_s \cdot (\gamma_s - \gamma_w) \cdot H^2 \cdot K_a$$
$$Z = 0.75 \cdot A_s \cdot f_s \cdot d_w - (M_{w1} + M_{w2} + M_{s1} + M_{s2})$$

Where:

Z	performance function
R	resistance
S	load
A_s	reinforcement area
f_s	yield strength of steel
z	internal lever arm
d_w	wall thickness
ΣM	sum of the moments

In the semi probabilistic calculations, the water force consists only out of the first part since the water level is equal to the top of the structure. In the FORM analysis it is possible to have a larger water level than the top of the structure, resulting in an extra force from the water pressure. The second force generated by the water pressure takes that into account, being zero when the water level is equal to the height of the structure and larger than one if the water level is larger than the height of the structure.

Input variables

The parameters used in the calculations are similar to the ones used in the previous calculations, except there are two new parameters introduced: the yield strength and the reinforcement area of the reinforcement steel. The reinforcement area has a normal distribution and the yield strength a lognormal distribution, see Table 6-13.

Table 6-13: Input parameters for the FORM analysis

Parameter	Sym	Unit	Dist	μ	σ	COV
Height structure	H	m	Normal	4.00	0.03	0.008
Thickness soil layer	d_s	m	Normal	1.00	0.05	0.050
Thickness wall	d_w	m	Normal	0.40	0.01	0.025
Specific weight water	γ_w	kN/m^3	Normal	10.00	0.20	0.020
Specific weight soil (sand)	γ_s	kN/m^3	Normal	20.00	1.00	0.050
Yield strength steel	f_s	kN/m^2	Lognormal	500E3	32.5E3	0.053
Reinforcement area	A_s	m^2	Normal	0.9E-3	1E-5	0.011

μ mean value
σ standard deviation
COV coefficient of variation

The lognormal distribution is differently treated than a normal distribution in the FORM analysis. Like for the Gumbel distribution, transformations have to be made. Every iteration starts with the calculation of the mean and standard deviation of the schematised normal distribution in the same way as is done for the Gumbel distribution (Rackwitz & Fiessler, 1977). The lognormal distribution has also two input parameters: a location parameter and a scale parameter. They can be calculated from the mean and standard deviation of the normal distribution (Holicky et al., 2008).

$$\sigma_{LN} = \sqrt{1 + \frac{\sigma_N^2}{\mu_N^2}}$$

$$\mu_{LN} = \ln\left(\mu_N - 0.5 \cdot \sigma_{LN}^2\right)$$

Where:

σ_N standard deviation normal distribution
μ_N mean normal distribution
σ_{LN} standard deviation lognormal distribution
μ_{LN} mean lognormal distribution

Output

The influence factors, design point and partial factors resulting from the FORM analysis are presented in Table 6-14. Again it can be seen that the water height is the most important parameter in the calculations.

Table 6-14: Output of the FORM analysis

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water height	h	m	Gumbel	0.935	5.015	4.0	1.25
Height structure	H	m	Normal	0.050	4.008	4.0	1.00
Thickness soil layer	d_s	m	Normal	0.027	1.007	1.0	1.01
Thickness wall	d_w	m	Normal	-0.106	0.394	0.4	0.99
Specific weight water	γ_w	kN/m^3	Normal	0.020	10.022	10.0	1.00
Specific weight soil (sand)	γ_s	kN/m^3	Normal	0.152	20.839	20.0	1.04
Yield strength steel	f_s	kN/m^2	Lognormal	-0.293	447E3	500E3	0.89
Reinforcement area	A_s	m^2	Normal	-0.047	0.897E-3	0.9E-3	1.00

α influence factor
X* design value
X value of the semi probabilistic approach
γ partial factor for the parameter

The resulting reliability and corresponding failure probability are respectively higher and lower than required, see Table 6-15.

Table 6-15: Output performance function of the FORM analysis

Iteration	1	2	3	4	5
μ	125	153	156	156	156
σ	21.8	27.8	28.3	28.3	28.3
β	5.72	5.51	5.52	5.52	5.52
P_f	5.5E-9	1.8E-8	1.7E-8	1.7E-8	1.7E-8

μ mean value of the performance function
σ standard deviation of the performance function
β reliability index
P_f failure probability

This FORM analysis demonstrates that the strength of the wall is sufficient to withstand the force on the wall. The other issue presented in the introduction of this chapter is the calibration of the partial factors used in the semi probabilistic approach, which is researched in the next paragraph.

6.5.2 Calibration of the partial factors

The partial factors used for the strength of the wall are different than for the stability of the structure. The resistance part in the stability calculations are forces and the resistance in the strength calculation are the material properties of the wall. This introduced two new material parameters: the yield strength and the reinforcement area of the steel. Both introduce extra uncertainties into the calculations. To obtain the partial factors, the reliability is fixed at the target reliability in the same way as it is done for the stability calculations. The results of the FORM analysis with a fixed reliability index are presented in Table 6-16.

Table 6-16: Output of the FORM analysis

Parameter	Sym	Unit	Dist	α	X*	X	γ
Water level	h	m	Gumbel	0.914	4.362	4.0	1.09
Height structure	H	m	Normal	0.012	4.006	4.0	1.00
Thickness soil layer	d_s	m	Normal	0.031	1.007	1.0	1.01
Thickness wall	d_w	m	Normal	-0.122	0.394	0.4	0.99
Specific weight water	γ_w	kN/m^3	Normal	0.007	10.007	10.0	1.00
Specific weight soil (sand)	γ_s	kN/m^3	Normal	0.174	20.829	20.0	1.04
Yield strength steel	f_s	kN/m^2	Lognormal	-0.337	447E3	500E3	0.89
Reinforcement area	A_s	m^2	Normal	-0.054	0.897E-3	0.9E-3	1.00

α influence factor
X* design value
X value of the semi probabilistic approach
γ partial factor for the parameter

The main difference between the calculation without and with a fixed reliability index is the water level. Fixing the reliability index at a lower value also results in a lower partial factor for the water level and consequently in a lower partial factor for the moment generated by the water pressure. The comparison of the influence factors is presented in Figure 6-10. From this figure can be concluded that also in this case the influence factors do not change significantly due to the fixed reliability index.

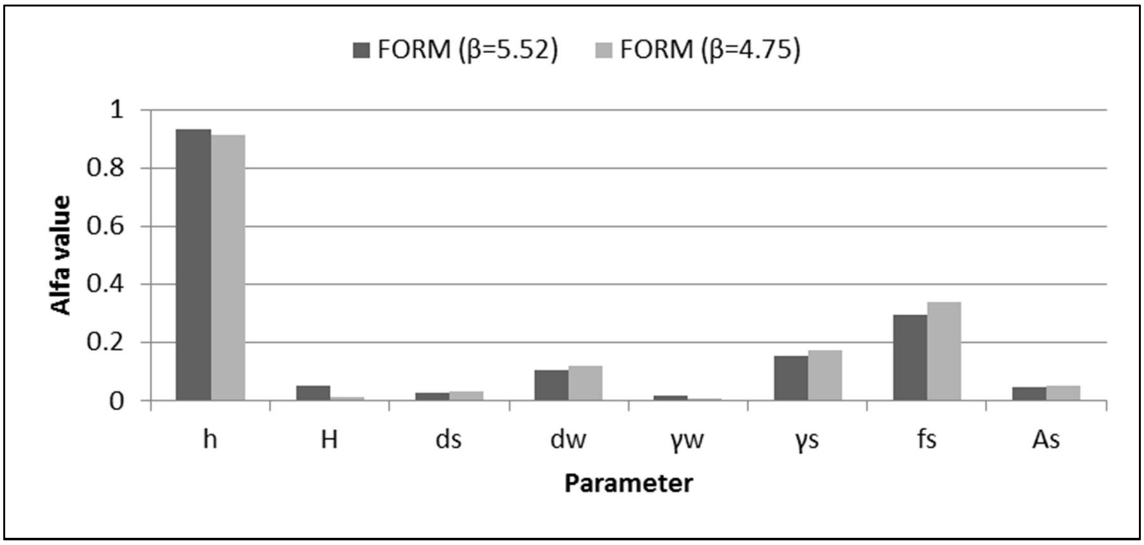

Figure 6-10: Influence factors for the FORM analyses with and without a fixed reliability influence

The partial factors for the water and soil pressure are calculated for the total moment generated by either the water or soil pressure, not for the decomposed forces.

$$\underbrace{\gamma_R \cdot M_R}_{\text{resistance}} - \underbrace{\gamma_w \cdot (M_{w1} + M_{w2})}_{\text{water forces}} - \underbrace{\gamma_s \cdot (M_{s1} + M_{s2})}_{\text{soil forces}}$$

The partial factors for the three cases are presented in Table 6-17 together with the corresponding failure probabilities.

Table 6-17: Partial factors for the three considered cases

Parameter	Sym	Case 1	Case 2	Case 3
Bending moment resistance	M_r	0.87	0.88	0.88
Water pressure	M_w	1.30	1.76	1.27
Soil pressure	M_s	1.30	1.09	1.09
Failure probability	P_f	<1.0E-6	1.7E-8	1.0E-6

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

The partial factor used in the semi probabilistic approach for the bending moment resistance of the wall correspond with the partial factors calculated with the FORM analyses. The difference between case 2 and 3 is not noticeable for the resistance partial factor, meaning that the difference in reliability does not come from the resistances part. The same is found for the soil pressure, only in this case the partial factor is much lower than the partial factor used in the probabilistic approach, meaning that the variability in the soil pressure is overestimated in the semi probabilistic approach. The partial factor for the water pressure differs quite a lot for the three considered cases, which is not remarkable given the fact that the water height is the most important parameter in the FORM analysis. Case number three presents the partial factors which need to be used to achieve the required amount of reliability. The partial factors for the water pressure and resistance are close to the ones used in the semi probabilistic approach. The partial factor for the soil pressure is much higher in case of the semi probabilistic approach, resulting in an unnecessary amount of reliability introduced into the design. The water height is just like for the horizontal and overturning stability the most important parameter. In the next paragraph is researched whether the same principle of applying only a design water level is suitable for the strength calculation as well.

6.5.3 Reliability based design

The water height is related to the design water level with a certain exceedance probability. The failure probability needs to correspond with the exceedance probability to be able to apply the same principle (using a design water level without a partial factor) to the strength of the wall. In Table 6-18 are the results presented of the three considered cases.

Table 6-18: Partial factors for the three considered cases

Parameter	Case 1	Case 2	Case 3	Unit
Design point water height	4.0	5.0	4.4	m
Design water level	12.0	13.0	12.4	m+NAP
Exceedance probability	1.0E-4	1.3E-7	7.0E-6	1/year
Failure probability	<1.0E-6	1.7E-8	1.0E-6	1/year

Case 1 Semi probabilistic approach
Case 2 FORM analysis
Case 3 FORM analysis (with fixed β)

From the table can be concluded that the failure probabilities correspond with the exceedance probability of the design water level. The failure probability is in both cases smaller than the exceedance probability. Applying a design water level with an exceedance probability equal to the target failure probability will result in an actual failure probability slightly lower than the target failure probability. So researching the horizontal stability, overturning stability and strength of the wall all resulted into the applicability of the principle of design water levels without the use of partial factors.

6.6 Conclusion

This chapter has treated three topics which are described in the introduction. For each topic a conclusion is presented in this paragraph.

Design verification

The first topic is the verification of the design presented in the previous chapter on three failure mechanisms (horizontal stability, overturning stability, strength wall). The failure probabilities of the three failure mechanisms are all smaller than the required failure probabilities, see Table 6-19. This means that the design is safe but the use of partial factors and a design water level introduces an excessive (factor 10 to 100) amount of reliability in the calculations.

Table 6-19: Reliability indices and failure probabilities of the failure mechanisms

Failure mechanism	Reliability index	Failure Probability
Horizontal stability	5.32	5.3E-8
Overturning stability	4.87	5.5E-7
Strength wall	5.52	1.7E-8
Required	4.75	1.0E-6

Calibration of the partial factors

The second topic is the calibration of the partial factors which are used in the semi probabilistic approach. The FORM analysis used to calculate the reliability indices and failure probabilities is adjusted in order to obtain partial factors with the target reliability, see Table 6-20.

Table 6-20: Partial factors for the three failure mechanisms

Failure mechanism	Force	Sym	Semi prob. approach	Calibrated value
Horizontal stability	Horizontal water pressure	$F_{h,wat}$	1.27	1.27
	Horizontal soil pressure	$F_{h,soil}$	1.20	1.04
	Vertical water pressure	$F_{v,wat}$	1.27	1.06
	Weight of the structure	$F_{v,conc}$	0.90	0.90
	Weight of the soil	$F_{v,soil}$	0.90	0.92
Overturning stability	Horizontal water pressure	$M_{h,wat}$	1.27	1.55
	Horizontal soil pressure	$M_{h,soil}$	1.20	0.98
	Vertical water pressure	$M_{v,wat}$	1.27	1.16
	Vertical water pressure	$F_{v,wat}$	1.27	1.16
	Weight of the structure	$F_{v,conc}$	0.90	0.99
	Weight of the soil	$F_{v,soil}$	0.90	0.96
Strength wall	Bending moment resistance	M_r	0.87	0.88
	Water pressure	M_w	1.30	1.27
	Soil pressure	M_s	1.30	1.09

Not all partial factors used in the semi probabilistic approach correspond with the calibrated values. Especially the partial factors for the forces resulting from the water level differ a lot from the partial factors used in the semi probabilistic approach, which is the result of the proportionality to the water height. For instance, the moment resulting from the horizontal water pressure in overturning stability is proportional to the third power of the water height and the moment of the vertical water pressure is linearly proportional to the water height. The calibrated partial factors for forces not related to water pressure are all closer to one than the ones used in the semi probabilistic approach.

Reliability based design

The partial factors calibrated for the force resulting from the water height vary between 1.06 and 1.55, so applying one partial factor on all forces is not adequate. The design points of the water level resulting from the FORM analyses are much more constant, in contrast to the partial factors. More or less the same water level is obtained for all forces in all three failure mechanisms resulting in the required amount of reliability, see Table 6-21. The table shows that the water heights, the corresponding design water levels and the exceedance probabilities are quite close to each other for the different failure mechanisms.

Table 6-21: Water levels for all failure mechanisms to obtain the target failure probability

Failure mechanism	Water height [m]	Design water level [m+NAP]	Exceedance probability [1/year]
Horizontal stability	4.5	12.5	3.0E-6
Overturning stability	4.6	12.6	1.6E-6
Strength wall	4.4	12.4	7.0E-6
Proposed one to use	4.7	12.7	1.0E-6

Additionally, the exceedance probability of the water level is also of the same order as the target failure probability of 1E-6. So applying a design water level with an exceedance probability equal to the target failure probability will result in an actual failure probability slightly lower than the target failure probability. The use of partial factors on the forces related to the water level is not required anymore in that case, i.e. the partial factor is equal to one. The highest partial factor for the loading forces (other than the ones related to the water level) is equal to 1.09. Applying a partial factor of 1.1 on all those forces is a conservative value. The same is obtained for the resistance forces, all calibrated partial factors are larger than 0.9, so applying a partial factor of 0.9 on all force is a conservative assumption. In Table 6-22 are the proposed values for the partial factors shown together with the ones used in the semi probabilistic approach as well the calibrated partial factors.

Table 6-22: Proposed partial factors to be used

Failure mechanism	Force	Sym	Semi prob. approach	Calibrated value	Proposed value
Horizontal stability	Horizontal water pressure	$F_{h,wat}$	1.27	1.27	1.0
	Horizontal soil pressure	$F_{h,soil}$	1.20	1.04	1.1
	Vertical water pressure	$F_{v,wat}$	1.27	1.06	1.0
	Weight of the structure	$F_{v,conc}$	0.90	0.90	0.9
	Weight of the soil	$F_{v,soil}$	0.90	0.92	0.9
Overturning stability	Horizontal water pressure	$M_{h,wat}$	1.27	1.55	1.0
	Horizontal soil pressure	$M_{h,soil}$	1.20	0.98	1.1
	Vertical water pressure	$M_{v,wat}$	1.27	1.16	1.0
	Vertical water pressure	$F_{v,wat}$	1.27	1.16	1.0
	Weight of the structure	$F_{v,conc}$	0.90	0.99	0.9
	Weight of the soil	$F_{v,soil}$	0.90	0.96	0.9
Strength wall	Bending moment resistance	M_r	0.87*	0.88	0.87
	Water pressure	M_w	1.30	1.27	1.0
	Soil pressure	M_s	1.30	1.09	1.1

*yield strength reduction according to the Eurocode: 1/1.15=0.87

The partial factor for the bending moment resistance is the only partial factor below 0.9. The overall stability of the structure is calculated only with the use of forces. The resistance in the strength calculations is generated by the material properties of the concrete. Partial factors in the semi probabilistic approach for the resistance of a concrete wall are applied on the material properties, so not on the resisting force. These material factors can continue to be used since the calibrated factor corresponds with the ones used in the semi probabilistic approach.

Applicability

The results from the three topics presented above are applicable for the design made in the previous chapter. In order to make proposed partial factors general applicable a lot more design configurations have to be regarded. The water height is the most important parameter in the calculations performed for this thesis. The partial factors for the other forces might become larger than the ones obtained, in case the water height is not the most important parameter.

The water height will be the dominant parameter when building below ground, because in that case the soil pressure and water pressure are the main forces acting on the structure. Placing a structure on top of this structure will introduce large force flowing down through the structure. The water level might become less important than the forces acting on the tall building. In that case applying the proposed partial factors of 1.1 on unfavourable forces and 0.9 on favourable force might not be introducing a sufficient amount of reliability. Further research is required to demonstrate to which extend these partial factor are applicable.

7 Evaluation

The master thesis is evaluated in this last chapter. This is done by answering the research questions and elaborating on the objective of this thesis in the first paragraph. The second paragraph lists a number of recommendations for further research on the outcome of this thesis.

7.1 Conclusions

In the introduction of the thesis are three research question presented to achieve the main objective of this these. First the research questions are answered after which is elaborated on the objective.

What aspects of the design, safety assessment and the risk analysis of dikes and hydraulic structures can be used for a dike with a structure inside?

Multifunctional flood defences do not only combine two or more functions to the same structure but it also results in coming together on a lot of other disciplines. On the financial, spatial and governmental level it leads to the coming together of different parties with different interests and purposes. On the technical disciplines, multifunctional use leads to a conflict between the Dutch guidelines for flood defences and the Eurocode for buildings. Especially dealing with the hydraulic loads on the structure leads to uncertainties for the reliability.

The assessment of the multifunctional flood defence has a lot of resemblance with the assessment of hydraulic structures and dikes. The difference with hydraulic structures is that the multifunctional flood defence is permanently closed, resulting in the absence of the failure mechanism reliability closure. The failure mechanisms that require the most attention are piping, overtopping, stability, strength and the connections.

There is an increasing use of probabilistic analyses for the design and assessment of the flood defence system. The multifunctional flood defence is also part of the flood defence system and is therefore analysed with a semi and fully probabilistic approach. The use of partial factors on forces resulting from the water level with a certain exceedance probability introduces twice an amount of reliability into the calculations, probably over dimensioning the flood defence.

Based on the case study: What are the attention points for the design and safety assessment of the dike with a structure inside?

For the case study, the location of the Grebbedijk is chosen because of both a demand for spatial development as well as flood safety. The design is assessed on the most critical failure mechanisms: overtopping, piping, stability, strength and connections. The transitions between the concrete and the soil are vulnerable to erosion in case of overtopping. The large crest width will decrease the flow velocities, reducing the load on the revetment of the flood defence. The occurrence of piping on the interface of the concrete and subsoil is resolved by the use of a sheet pile wall. The outcome of the calculations on the stability and the strength fulfil the requirement but this involves the use of partial factors together with the design water level. The connections between the multifunctional flood defence and the adjacent flood defences are sensitive to erosion in the same way as the transitions between concrete and soil.

The construction of the design is a critical point in the design of multifunctional flood defence in case of constructing inside the present flood defences; the construction time is limited to the summer period of six months. The design for the multifunctional flood defence is analysed and there has been shown that the construction time is within the available six month period. During construction there has to be paid close attention to the time schedule in order to stay within the available time period.

Changing the dimensions of the structure is in principle possible in all directions. Changing in landward of riverward direction is very much depending on the available space at the specific location. The most interesting dimensions to change are in height or in depth. Increasing the height of the structure allows other functions for the structures because the structure will protrude out of the dike. The construction time is a very important aspect for increasing in height as well as in depth; it has to be within the available construction period of six months.

The purpose of the case study is to identify the issues and differences with the multifunctional flood defences. The main problem is how to deal with the hydraulic loads in the assessment of the overall stability of the structure and the strength of structural elements. Apply large partial factors over design water levels with very small exceedance probabilities seems to conservative.

Based on the case study: What are the failure probabilities of the failure mechanisms and do the partial factors used in the semi probabilistic approach correspond with the ones obtained from the reliability analyses?

The three failure mechanisms, in which the hydraulic load is the dominant parameter, are the horizontal stability, overturning stability and strength of the wall. For all three failure mechanisms are reliability analyses carried out resulting in failure probability much smaller than the required failure probability. Designing a multifunctional flood defence with partial factors on the force related to the water level is a too conservative approach.

The calibrated partial factors are too much depending on the proportionality of the force to the water level. From this can be concluded that the use of partial factors is not a convenient way to deal with reliability in these calculations. The FORM analyses showed that for all three failure mechanisms more or less the same design water level is required to meet the target reliability. The exceedance probability of the design water level is in the same order as the failure probability for all three failure mechanisms. So applying a design water level with an exceedance probability equal to the target failure probability introduces the required amount of reliability into the calculations. This only holds for the forces related to the water level. The calibrated partial factors for the loading forces (not related to the water level) are all smaller than 1.1 and the calibrated partial factors for the resisting forces are all larger than 0.9. So using a design water level and no partial factors (i.e. factor of one) for the water forces, together with a partial factor of 1.1 for loading forces and partial factor of 0.9 for resistance forces, results in the required amount of reliability.

The results of the FORM analyses are applicable for the design made in this thesis. In order to make proposed partial factors general applicable a lot more design configurations have to be regarded. The water height is the most important parameter in the calculations performed for this thesis. The partial factors for the other forces might become larger than the ones obtained, in case the water height is not the most important parameter.

Finding a way to design and assess a dike with a structure inside in a reliable manner.

The objective of this thesis is to find a way to design and assess a dike with a structure inside in a reliable manner. The current guidelines and codes present partial factors which introduce an extensive amount of reliability into the design. Additionally, forces related to the design water level with already a very small exceedance probability do also get partial factors and those partial factors are not related the proportionality of the design water level to the forces. For the design presented in this thesis is demonstrated that the target reliability can be obtained by using a design water level with an exceedance probability equal to the target failure probability. The partial factors for the forces related to the design water level are equal to 1.0, other unfavourable forces equal to 1.1 and other favourable forces equal to 0.9.

7.2 Recommendations

In the research of this master thesis are some assumptions and simplifications made in order to obtain the results. This paragraph lists a number of recommendations for further research and some ideas to improve or extent the obtained results.

- This thesis focussed only on the technical discipline for multifunctional use. On other disciplines (financial, spatial and governmental) are also some impediments which require investigation before multifunctional flood defences can be realised. This requires the collaboration of all involved parties and the willingness and positive attitude towards multifunctional use of the flood defences.
- The existing guidelines do not prohibit multifunctional use of the flood defences but do certainly not embrace it. Creating a guideline or framework for multifunctional use of flood defences on how to deal with certain technical difficulties would give involved parties more confidence to start the discussion on other non-technical impediments.
- The case study shows that the construction phases are an important aspect of multifunctional flood defences, because of the limited amount of time to construct the multifunctional flood defences. The design made in this thesis is likely to be constructed within this time window, but further investigation is required to guarantee that the construction time is shorter than the time window.
- Transitions between different types of revetment are in general a weak spot in the flood defences. The connections between the multifunctional flood defence and the adjacent flood defences require more research in order guarantee the safety of the connections.
- Changing the dimensions of the structure is possible because all problems can be solved due to constructive measures. However, the reliability analysis is done for the design presented in this thesis. Changing the dimensions of the structure may lead to other results of the reliability analysis. As long as the water force is dominant over the other forces the partial factors proposed in this thesis will be applicable. If, for instance, a tall building is placed on top of this structure the wind force becomes more dominant than the water force. In that case are the partial factors not applicable anymore. Further

research is required to investigate the applicability of the partial factors for different design configurations.

- In the semi probabilistic calculations as well as in the FORM analysis, the conservative assumption is made that the ground water level is equal to the river water level. The ground water level will have smaller values than the river water level in reality, because of the limited permeability of the soil. Research to the actual ground water level will result in smaller loads on the structure.
- When designing and assessing a dike with a structure inside it is recommended to use a fully probabilistic approach instead of a semi probabilistic approach because the partial factors prescribed by the current guidelines and codes result in the introduction of an extensive amount of reliability.

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Appendices

A Examples of multifunctional flood defences

This appendix describes three examples in relation to the two chosen kinds of multifunctional use of flood defences. A lot more examples of multifunctional use can be found in the literature (Otter, 2003; Voorendt, 2012b). The example describes in this chapter are:

- Katwijk, dike in dune with parking garage
- Schevingen, boulevard
- Rotterdam, Roof Park

A.1 Katwijk, dike in dune with parking garage

The safety assessment of 2006 showed that Katwijk cannot resist a storm with an exceedance probability of 1/10,000 per year. If this storm occurs, large parts of Noord-Holland, Zuid-Holland and Utrecht will be flooded. The flood safety aspect together with the need of the municipality for more parking spaces resulted in a flood defence that is a combination of a dune and dike with a parking garage inside, see Figure A-1.

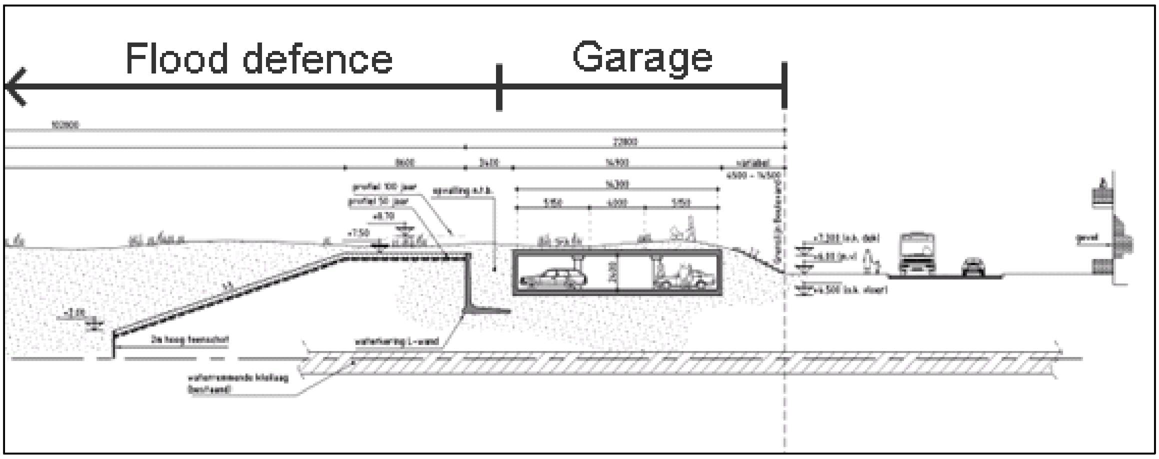


Figure A-1: Cross section dike in dune with parking garage

The dike inside of the dune ensures the flood safety, because it reduces the dune erosion. The requirements for the parking garage are the following (Rijnland, 2012):

- The design of the flood defence is leading.
- The garage in completely separated from the flood defence.
- The garage should not lead to loss of stability or induce unforeseen settlement.
- The garage must be located outside the core area of the flood defence.

From these requirements it can be obtained that the functions flood safety and parking space must be separated. Eventually, a solution with a parking space outside the flood defence is chosen, because of the lack of trust and confidence about flood safety aspects from the water board.

A.2 Scheveningen, boulevard

The boulevard of Scheveningen is one of the weak links in the flood defence system. Urban development and flood safety are combined in the design for the new boulevard. The dike consists of a sand body with a stone revetment on top of it. Over this dike the boulevard and the beach are created, so the dike is actually hidden under the boulevard, see Figure A-2.

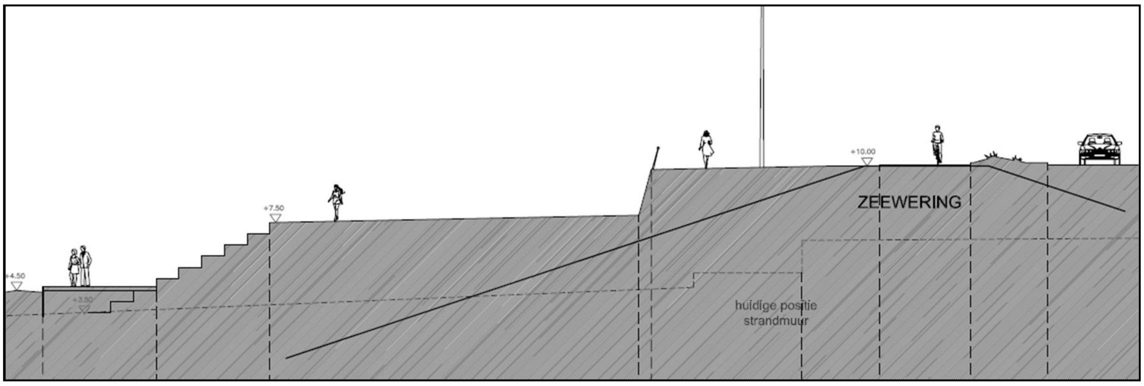


Figure A-2: Schevingen, dike in boulevard

The foreshore is raised with sand supplementation to reduce the wave impact on the boulevard and dike. The crest height can be lower due to the reduced wave impact. A lower crest height leads to less visual blocking, which is a wish from the residents of Schevingen. The design of the new boulevard is made by a renowned Spanish architect, see Figure A-3.

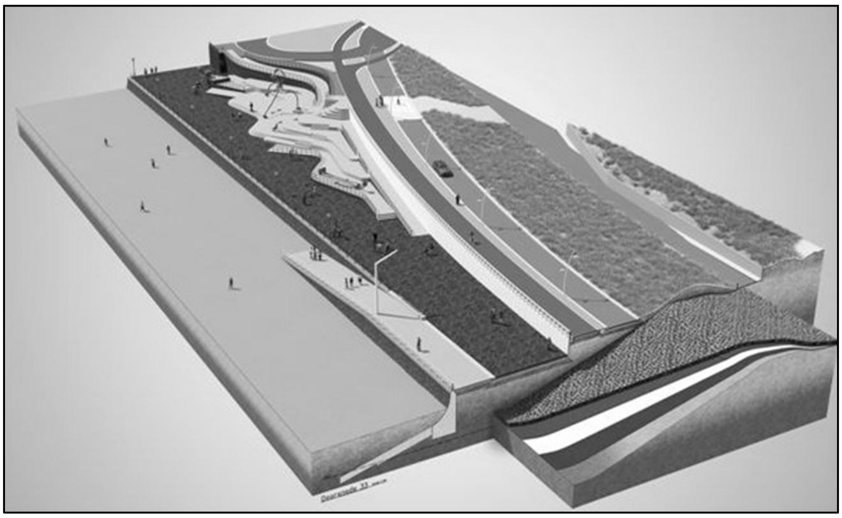


Figure A-3: Artist impression Scheveningen boulevard

A.3 Rotterdam, Roof Park

This multifunctional building is located next to a flood defence. The building itself is a prefect example of multifunctional use of space. The building combines a shopping centre with a park and underneath the park a parking garage is located. The water retaining function of the flood defence is performed by the dike and remains intact, the Roof Park is therefore not fully integrated, see Figure A-4.

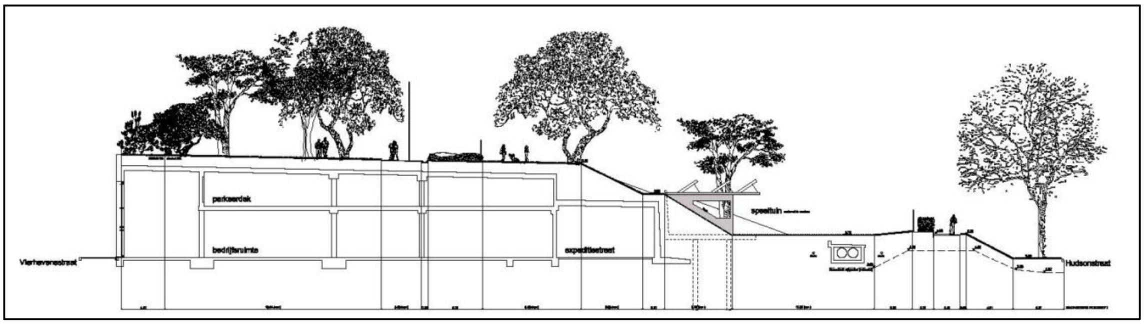


Figure A-4: Roof Park in Rotterdam

Initially the water board was strongly opposed to the construction of the roof park because it would give a lot of trouble with future adjustments to the flood defence. The municipality of Rotterdam has promised to the water board to pay for the extra costs of future adjustments to the flood defence, so the water board would give the construction permit to the municipality for the construction.

B Introduction of the case study

B.1 Choice of the location for the case studie

For the location of the case study is looked into four different locations (Nijmegen, Arnhem, Tiel and Wageningen, see Figure B-1) all located along the main rivers in the Netherlands. All locations have, in greater or lesser extent, problems with flood safety and a demand for special development.

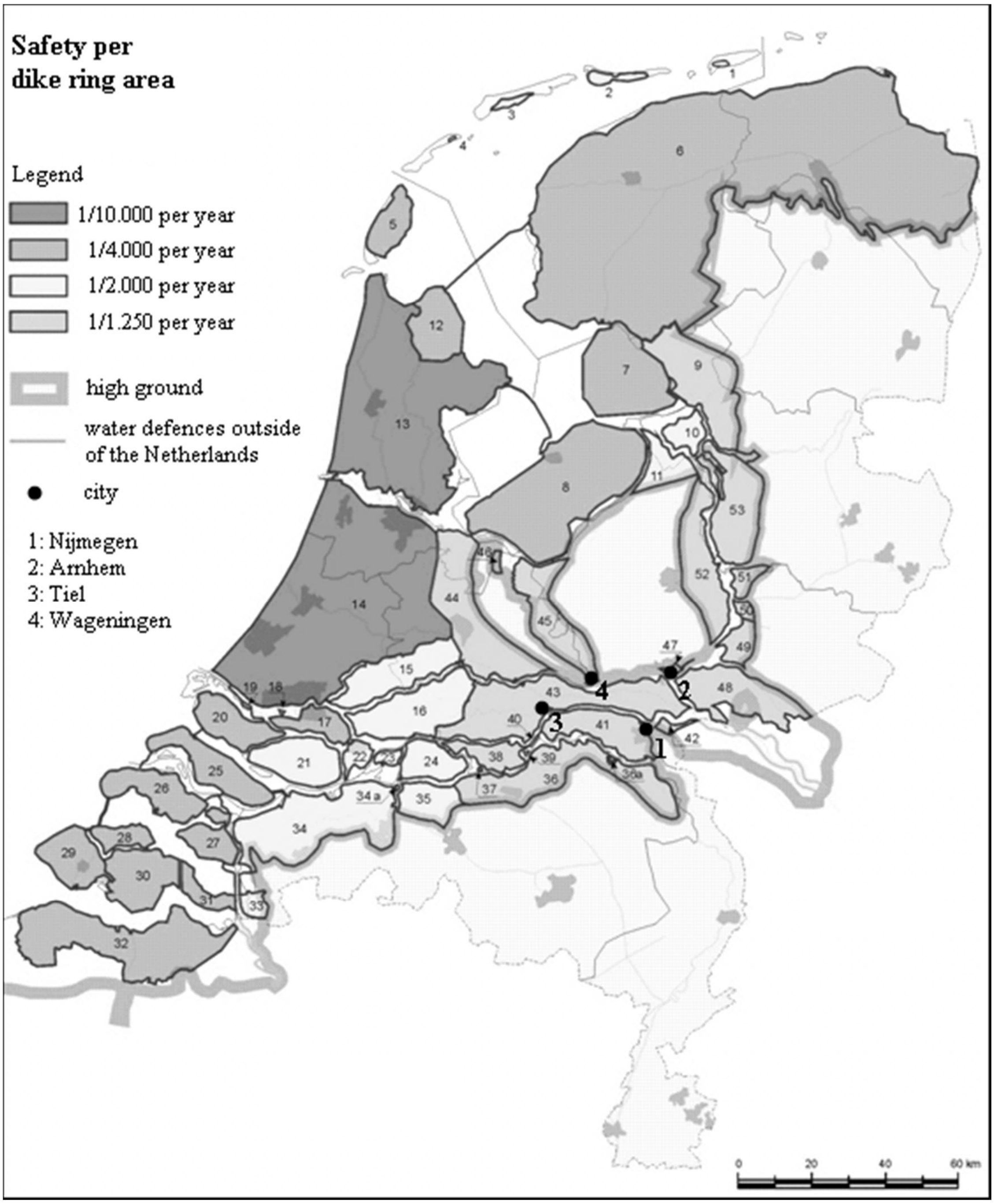


Figure B-1: The potential locations considered for the case study

Nijmegen

There are currently two projects running at Nijmegen, Waalfront 1.2 and Dike relocation Lent. Dike relocation Lent is located on the north side of the river Waal where a side channel is established. The dike is moved landward for the construction of the side channel resulting in an island between the main and side channel. On the north side of the side channel a new part dike and quay is created and the island is raised. For the design of the new dike was chosen for a green solution. The project Waalfront 1.2 is located on the south side of the river Waal where an old industrial area is converted to residential area. This project consists of a number of subprojects whereby in each subproject houses are built according to the demand for residential properties and economic resources.

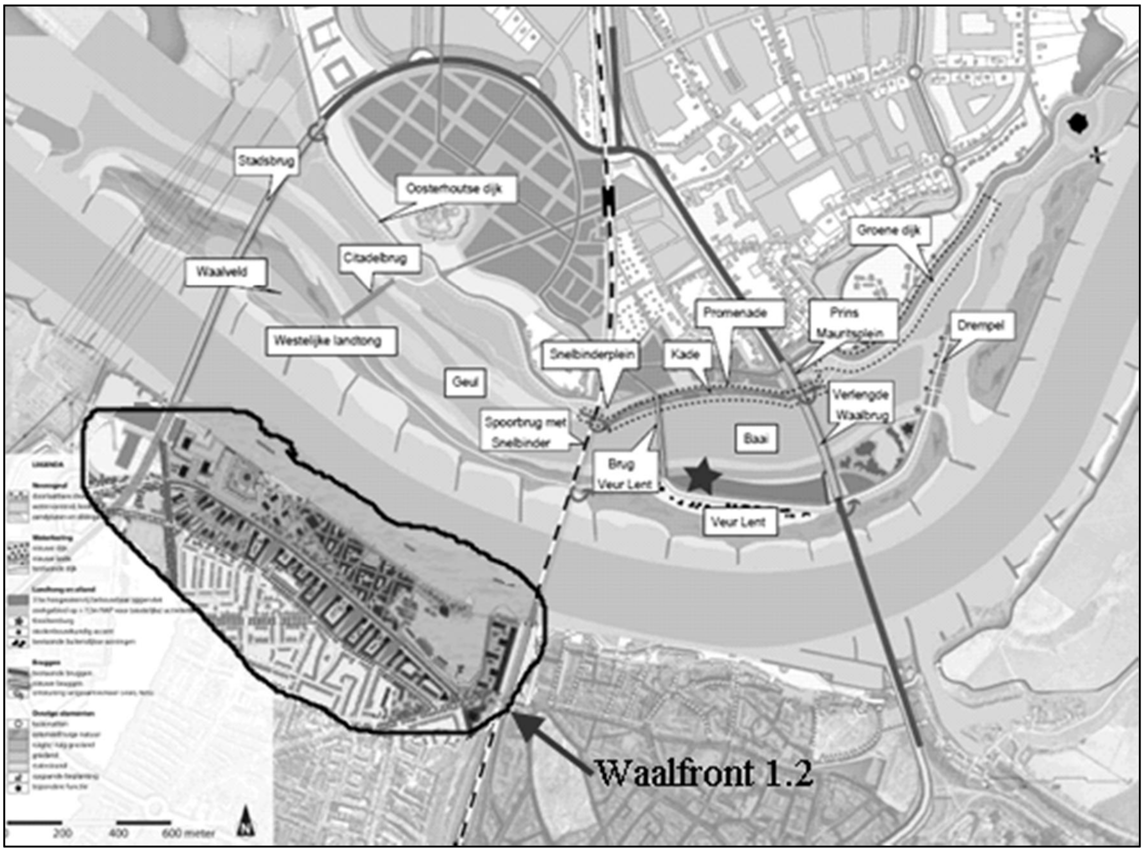


Figure B-2: Projects around Nijmegen

Arnhem

Arnhem is located at the junction of the Pannerdensch Channel in the Lower Rhine and the IJssel. On the north side of the river is much industry and infrastructure and the primary barriers are mostly quays. On the south side of the rivers the flood defence are mostly dikes and there is plenty of space for recreation and nature. To my knowledge there are no running projects or initiatives that have any relation to structures in dikes.

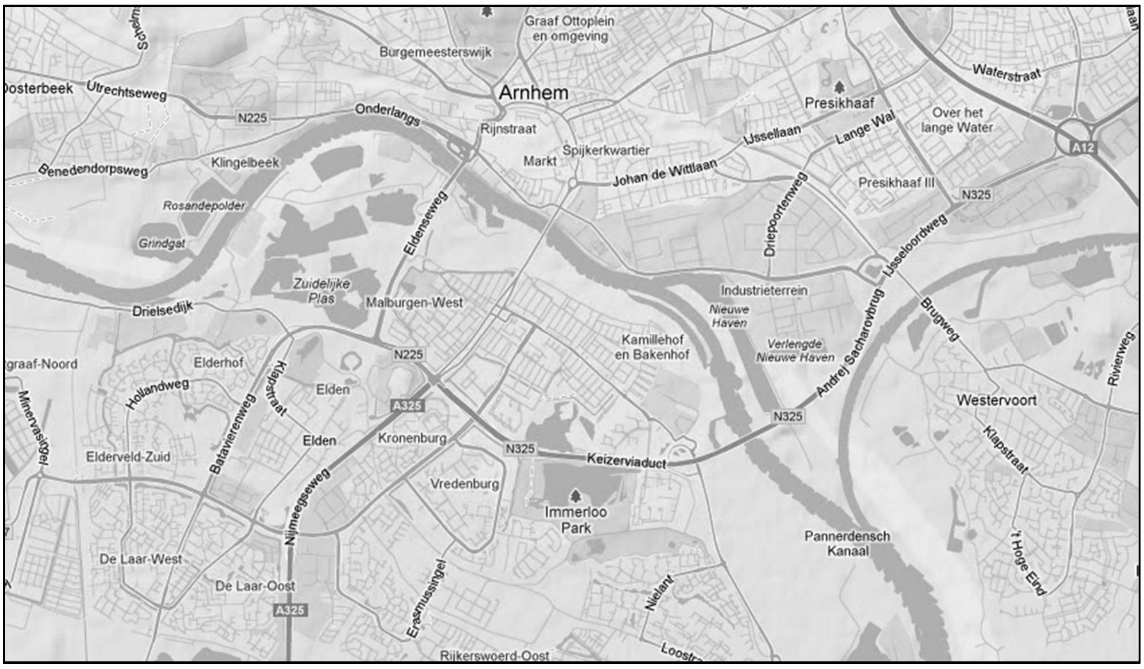


Figure B-3: Area around Arnhem

Tiel East

Tiel East is struggling with seepage problems caused by the absence of a sealing layer in the soil. There is chosen for an integrated approach; besides solving the water issues there is also attention for stimulating the development of housing, nature and recreation. For this project is already a preliminary design made, there is chosen for a green embankment for the functions recreation and nature. In this design, the fill up of the port excluded, but there are already some sketches made in the master plan . The damping of the port and urban development would be a suitable location for a dike with a construction therein.

Figure B-4: Masterplan development Tiel

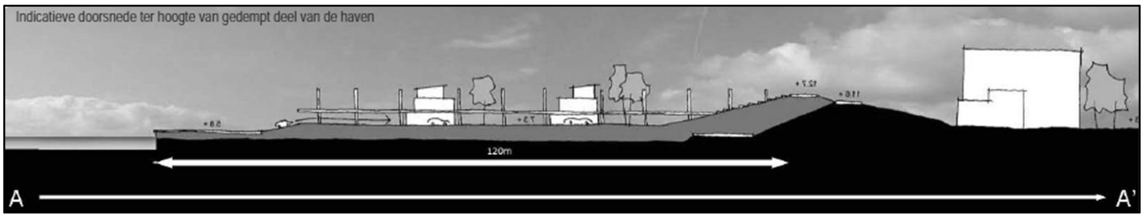


Figure B-5: Cross section of filled up port

Wageningen

Between Wageningen and Rhenen is the 5.5 km long Grebbedijk situated. This dike is a candidate to be redesigned soon because it protects an area with a lot of people and a dike breach will lead to huge economic losses. The idea is to design the dike in such a way that it is almost impossible to breach, a so-called super dike. In a research of Deltares to delta dikes (same meaning as super dike = unfailable dike) is the Grebbedijk included. This study focused on locations where delta dikes reduce the casualty risk most effectively. The study showed that the Grebbedijk is one of the locations where a delta dike is most cost effective. LA4sale (La4sale, 2010) is a landscape architectural firm which was commissioned by the province of Utrecht to do a field analysis and develop design strategies for the Grebbedijk. Hereby are also a number of sketches made containing a structure inside of the flood defence, but still with separate parts for the flood defence and the structure (see Figure B-6 and Figure B-7).

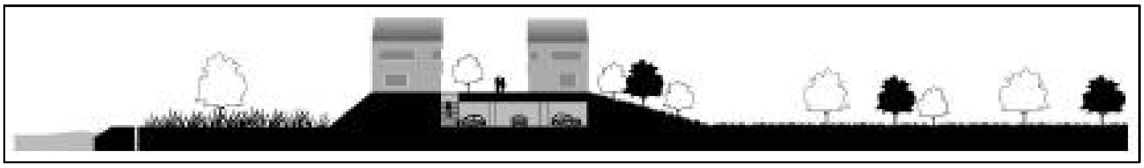


Figure B-6: Cross section with infrastructure and car parking inside the dike and housing on the dike

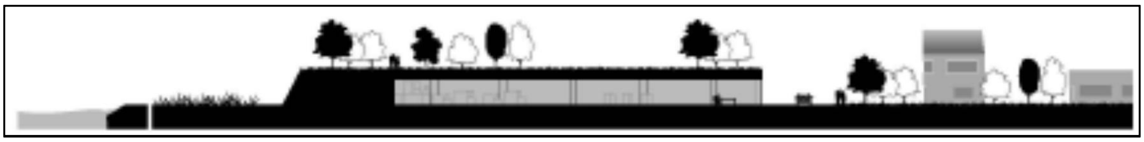


Figure B-7: Cross section with office area inside the dike and recreation and nature on the dike

Case selection

For the choice of the location for the case study are a couple of criteria of importance:

1 Demand for flood safety
A high demand for flood safety is positive and a low demand is negative.
2 Demand for spatial development
A high demand for spatial development is positive and a low demand is negative.
3 Running projects
Locations where the flood defence is recently or is currently being built or reinforcement are negative because adjustments or new projects are not likely to occur, otherwise this is positive.
4 Type of flood defence
The type of that is looked for is a dike with enough space in it surrounding area to develop.

The four location described in the previous paragraphs are evaluated on the criteria. The results are presented in Figure B-1.

Table B-1: Criteria for the chosen location (N=Nijmegen, A=Arnhem, T=Tiel, W=Wageningen)

Criterion	N	A	T	W
1: Demand for flood safety	+	+	++	++
2: Demand for spatial development	++	-	+	+
3: Already running projects	-	+	-	+
4: Type of flood defence	-	+	+	+

- Nijmegen
  - There is a demand for flood safety because the river is a bottleneck at this location, so a widening the river means relocation of the flood defences.
  - Nijmegen has a high demand for spatial development on housing, infrastructure and recreational areas.
  - At the north as well as the south side of the river projects are already started.
  - The flood defences on the south side are meanly quay walls.
- Arnhem
  - There is a little amount of demand for flood safety; Arnhem is often mentioned as a candidate for an unfailable dike.
  - In Arnhem there is currently no noticeable amount of demand for safety development.
  - There are no running projects.
  - The flood defence system consists out of dikes.
- Tiel
  - There is a lot of trouble with seepage water in this city, so a high demand for special development.
  - There are plans for a redesign of the southeast part of Tiel.
  - No projects in execution but there are plans in an advanced stage.
  - It is a relatively small part of the dike that is available for multifunctional use.
- Wageningen

- An excellent location since a very small dike in length protects a large area.
- In the city centre there is a demand for parking space.
- There are currently no running projects.
- The flood defence system consists out of dikes.

From this can be concluded that the location Wageningen is the best location to perform a case study.

B.2 Area and stakeholder analysis

B.2.1 Area analysis

The Netherlands is divided into dike ring areas, which are each protecting a piece of land. Every dike ring area has a safety standard, depending on the threat, the size and the importance of the area. The Grebbedijk is part of the dike ring area 45, Gelderse Vallei. This dike ring area has an exceedance probability of 1/1250 per year, so this area is protected against water levels that occur on average once every 1250 years. Figure B-8 shows the dike ring areas of the Netherlands and the exceedance probabilities for every dike ring area.

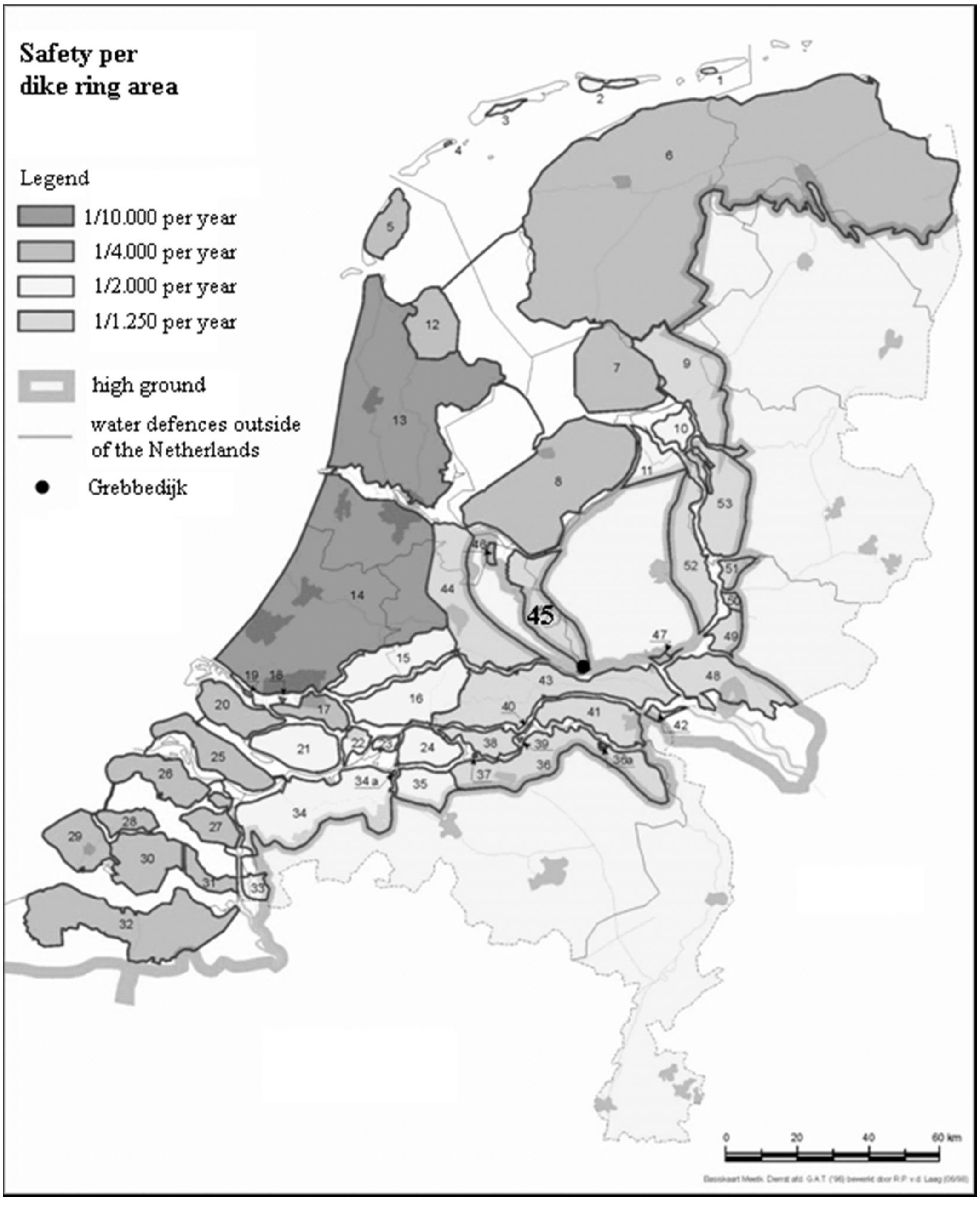


Figure B-8: Exceedance probability dike ring areas

Dike ring area 45 is adjacent to the Lower Rhine in the south part of the area and to the Eem, Eenmeer and the Nijkerkernauw in the north part, see Figure B-9. The dike ring area located between two high grounds, in the east the Veluwe and in the west the Utrechtse Heuvelrug. The Grebbedijk in the south part of this area protects the area against the water from the Lower Rhine. The provincial border between Utrecht and Gelderland runs through this area. A breach of this dike causes disruption for 250,000 people and an economic loss of 10 billion euro (Wijnacker, 2013).

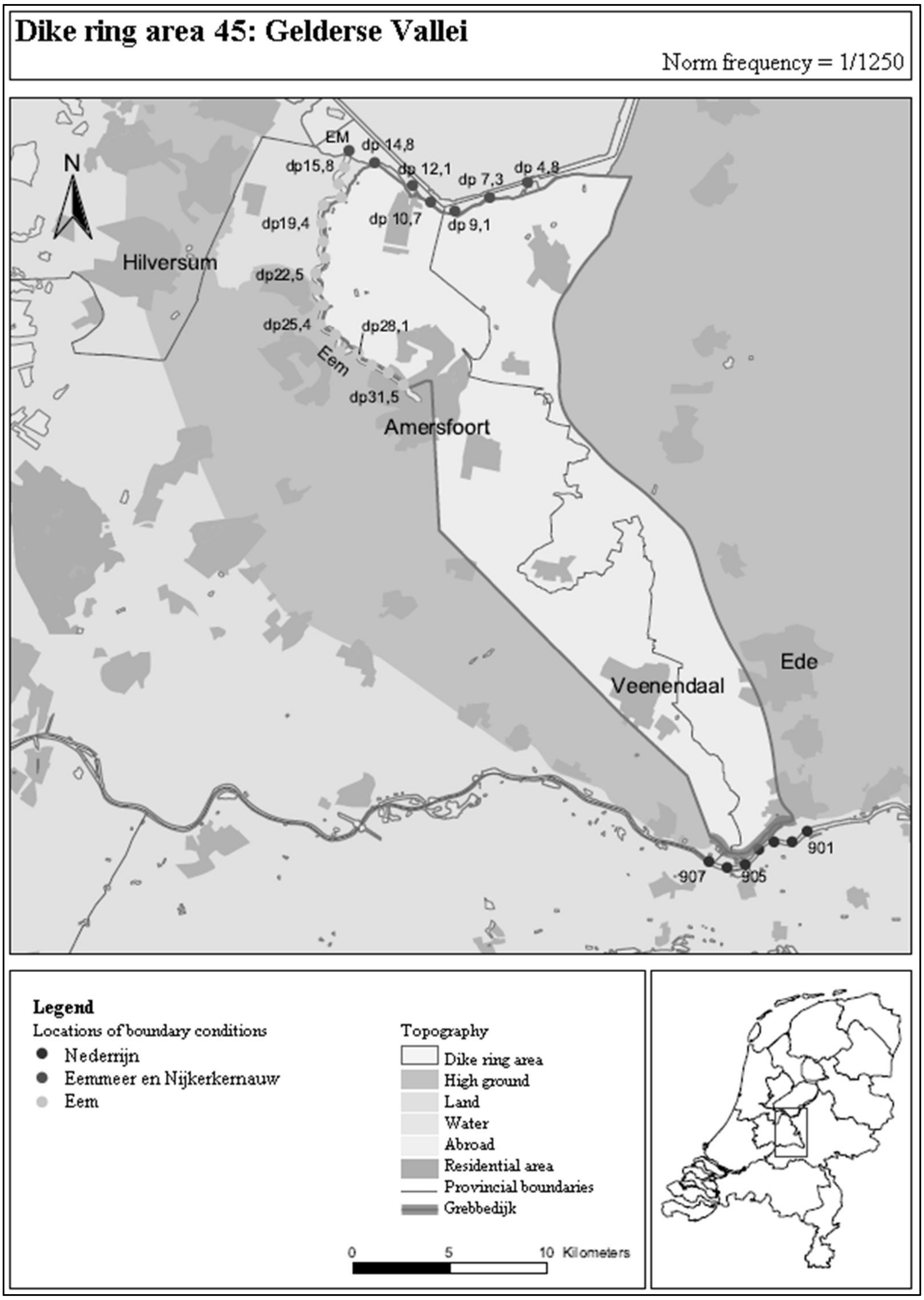


Figure B-9: Dike ring area 45: Gelderse Vallei (HR, 2006)

Further zooming in on the Grebbedijk shows clearly how the Grebbedijk connects two high grounds with each other, see Figure B-10. On the east side, the Grebbedijk connects to the Veluwe and on the west side to the Utrechtse Heuvelrug. The Grebbelijk has a length 5.5 km.

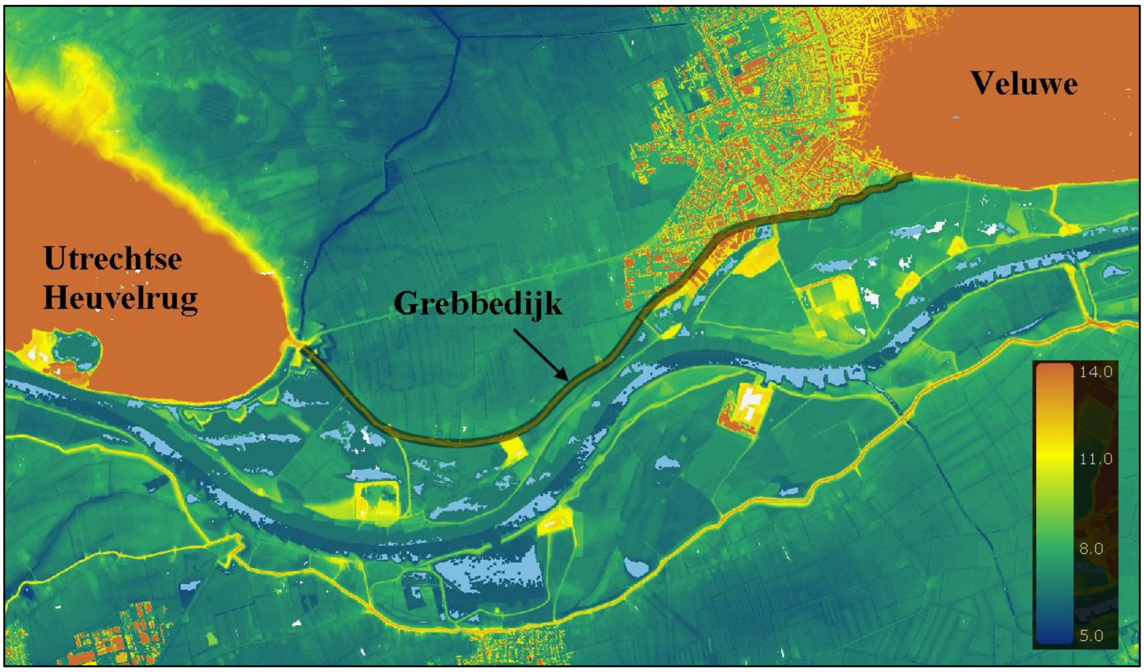


Figure B-10: The Grebbedijk is the connection between high grounds (AHN)

Figure B-11: Area analysis (Google Maps)

When having a closer look at the surrounding area (Figure B-11) of the Grebbedijk, four different types of surroundings can be distinguished. Behind the most eastern part of the Grebbedijk is the old city centre of Wageningen located, this part is the residential area. A small harbour surrounded by an industrial area is located west of the area. Further to the west is a small marina located with the related facilities. The largest and most western part of the Grebbedijk is located in rural area. It is not possible to expand the Grebbedijk landward in the residential area because of the old city centre and the just built Rustenburcht neighbourhood. There is also a part of the stronghold of Wageningen located along the Grebbedijk. In the industrial area the Grebbedijk is a wide dike with buildings on top of the dike. From the marina area westward the available area around the dike for the application of a wider dike becomes larger. Also another alignment of the dike could be possible for the western part of the Grebbedijk, see Figure B-12.

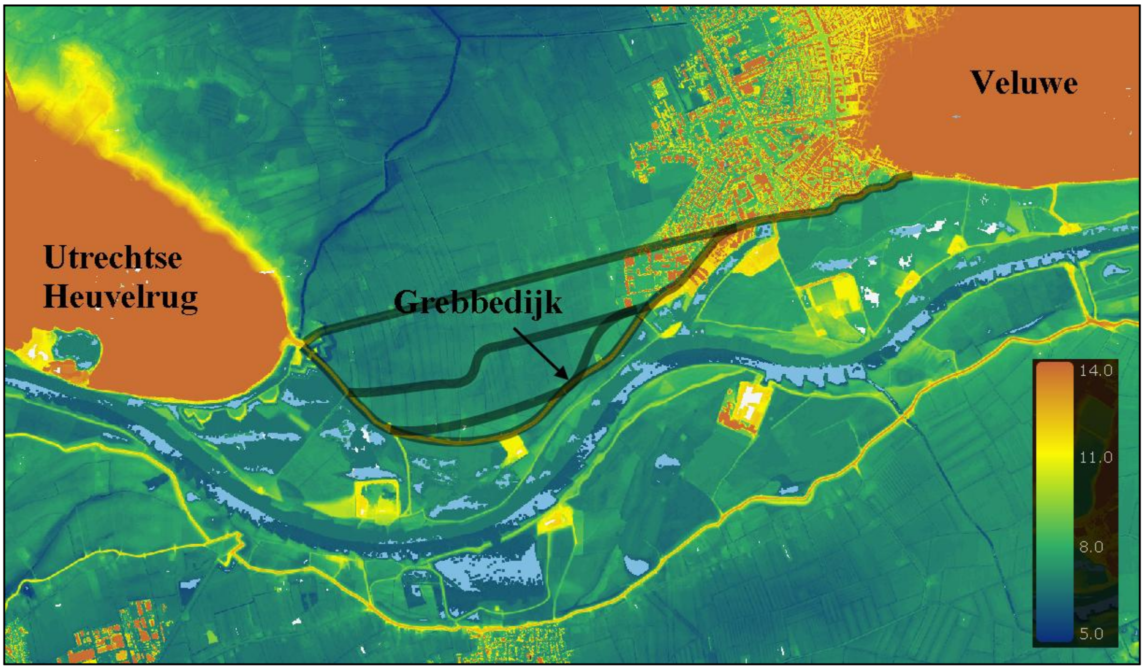


Figure B-12: Possibilities for adjusting the alignment of western part of the Grebbedijk (AHN)

Figure B-13: Location of the Hoornwerk and the Witte Sluis (Google Maps)

On the Grebbedijk itself are two historically structures identified, the Hoornwerk at the west end and the Witte Sluis on near the old city centre, Figure B-13. The Hoornwerk is part of the Grebbelinie and defended the Grebbeberg and the sluice which controlled the inlet of water in the Lower Rhine, see Figure B-14 and Figure B-15. The Witte Sluis was built because of a dike section relocation which closed of a stream, but is not used anymore.

Figure B-14: Hoornwerk (Google Maps)

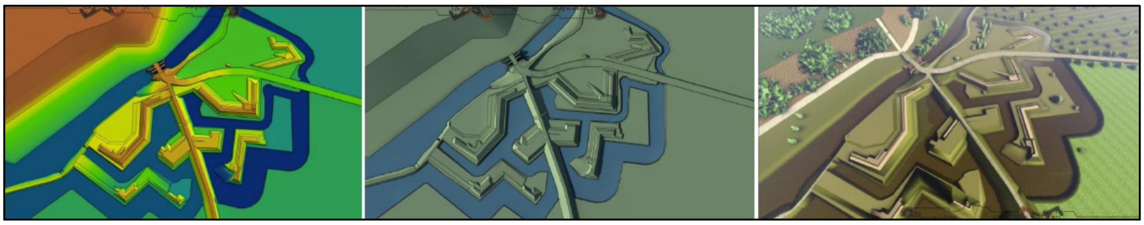


Figure B-15: Impression of the Hoornwerk at the east end of the Grebbedijk (Bruinsma, 2010)

B.2.2 Stakeholder analysis

The biggest challenge with multifunctional use of the flood defences is the amount of stakeholders involved. Every stakeholder has its own wishes, interests and influences and they are likely to conflict with each other. The water board is responsible for the management of the flood defences and normally has minimal interference from other parties. In case of multifunctional use the amount of stakeholders increases, which complicates the processes on different aspects. For the Grebbedijk the following stakeholders are identified:

- Province

The province determines whether cities can expand, where industrial and office parks may be built, where infrastructure is located and where agricultural, natural and recreational areas come. All this is put in a structure plan, which the municipality fulfils with zoning plans. The province is also responsible for the preservation and creating of nature and the observance of environmental laws for air, soil and water quality. It is also responsible of the supervision of the municipalities and the water boards.

For the Grebbedijk the provinces Utrecht en Gelderland are involved. Both provinces have expressed that they want to anticipate to the possible changes is safety standards, the assumption of an unfailable dike. They also want to invest in the special quality of this area, specific the infrastructure and recreational areas preserving the cultural and historical value of this area.

- Municipality

The municipality is responsible for the zoning plans that fulfil the structure plans drawn up by province. The location of houses, nature and companies are defined in the zoning plans. The municipality also oversees the housing construction.

The municipalities involved with the Grebbedijk are Wageningen and Rhenen. The municipalities mentioned together with other municipalities are united in several foundations like Projectbureau SVGV. This foundation together with its partners works on seven themes; nature, scenery and cultural history, environment, water, agriculture, recreation and liveability.

- Water board

The water board is responsible for the management of the water system. They make sure that we have enough clean water and then we are protected against flooding. In the field of flood safety they take into account changes in design water levels and safety philosophies.

The Grebbedijk is part of dike ring area 45, which fall under the management of the Vallei en Veluwe water board since 01-01-2013. The water board cooperation with the province of Utrecht has commissioned a research (Ter Maat, 2009) whether there should be invested in compartmentalisation of the area or designing an unfailable dike. The outcome of the research is that is an unfailable dike is preferred and this conclusion is taken over by the water board.

- Nature organizations

Several organizations are involved in the surrounding area of the Grebbedijk, like Staatsbosbeheer and Natuurmomumenten. These organizations attempt to preserve or intensify natural and recreational areas, wildlife and cultural historical buildings and structures. The provinces, municipalities as well as the water board have expressed that the design of a new flood defences has to have a rural feel and that existing natural and recreational areas at least have to be conserved.

- Companies and individuals

With a new design of the Grebbedijk there are also a lot of companies and individuals involved, especially if the alignment of the dike is changed. By informing and involving them in an early stage of the design process, the companies and individuals can express their concerns and problems and those can be incorporated in the further design process.

The companies involved can roughly be divided into two groups, the companies around the harbour and the companies in the rural area. The companies around the harbour are mostly handling raw materials and the companies in the rural area are mostly farmers. The two groups of companies are not likely to cause any problems if they are not hindered in their doings. Individuals are more complicated, but involving them and compensating them for the losses will likely ensure the resolving of most of their problems.

- Marina

The reason that the marina plays a significant role in the new design of the Grebbedijk is that the marina forms an obstacle for wildlife to pass from the Utrechtse Heuvelrug to the Veluwe. Moving the marina to another location makes it possible to have an ecological link between Utrechtse Heuvelrug and the Veluwe. The entrance of the harbour can than be crossed by larger species. The new location of the marina

B.2.3 Site visit

To get a better feeling of the area, I visited the area and had a look myself at the dike and its surroundings. In Figure B-16 are the locations mentioned of the photos taken during the site visit.

Figure B-16: Locations of the photo taken (Google Maps)

Through the site visit is determined that the rural area consists of two parts; the western part is an area with a higher historical and natural value than the eastern part. Also in the residential area a division can be made into three parts; the western part behind which the old city centre is located, the middle part behind which the stronghold is located and the eastern part behind which the new build neighbourhood Rustenburcht is located.

Figure B-17: Photo 1, West end connection to high grounds

Figure B-18: Photo 2, New residential area

Figure B-19: Phote 3, Stronghold of Wageningen

Figure B-20: Phote 4, Old city centre of Wageningen

Figure B-21: Phote 5, Entrance of the harbour of Wageningen

Figure B-22: Phote 6, Harbour of Wageningen

Figure B-23: Phote 7, Entrance of the marina of Wageningen

Figure B-24: Phote 8, Marina of Wageningen

Figure B-25: Phote 9, Rural area of the Grebbedijk

Figure B-26: Phote 10, East end of the Grebbedijk, Hoornwerk

B.3 General information case study Grebbedijk

B.3.1 Cross sections Grebbedijk

The area for the

Figure B-27: Top view locations cross sections 1 of 3 (WVE, 2012)

Figure B-28: Top view locations cross sections 2 of 3 (WVE, 2012)

Figure B-29: Top view locations cross sections 3 of 3 (WVE, 2012)

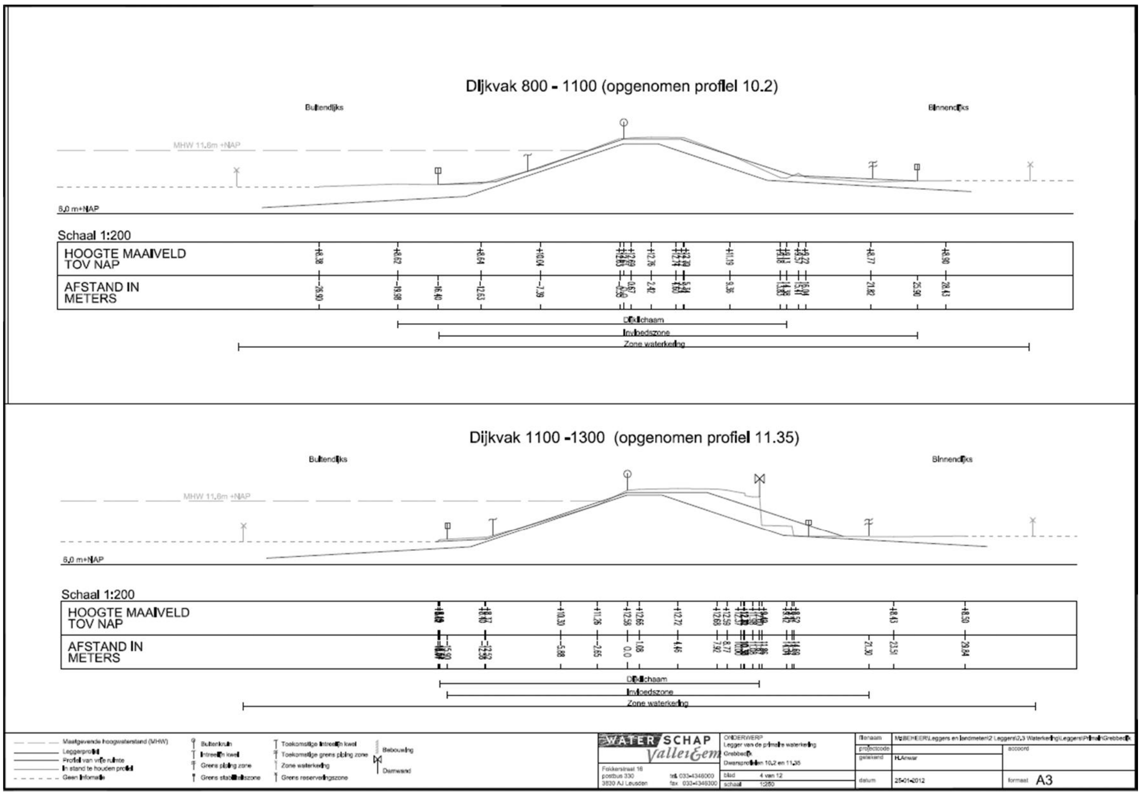


Figure B-30: Cross sections at 10.2 and 11.35 (WVE, 2012)

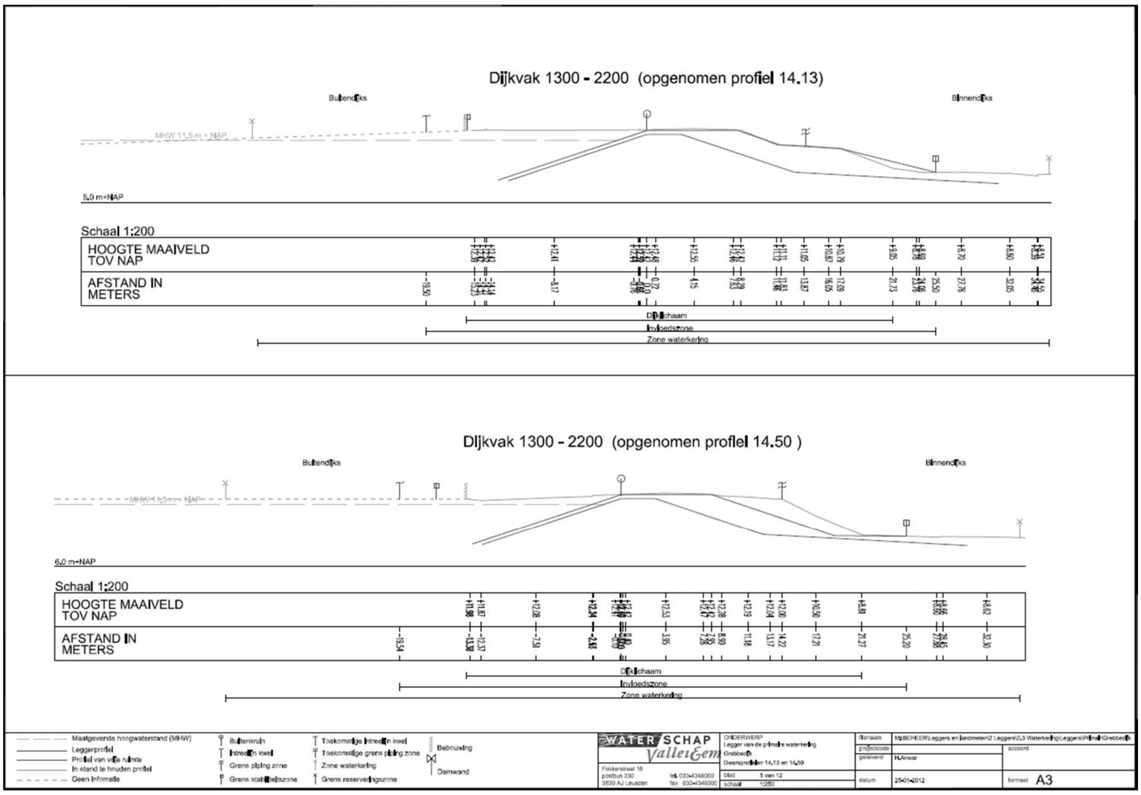


Figure B-31: Cross sections at 14.13 and 14.50 (WVE, 2012)

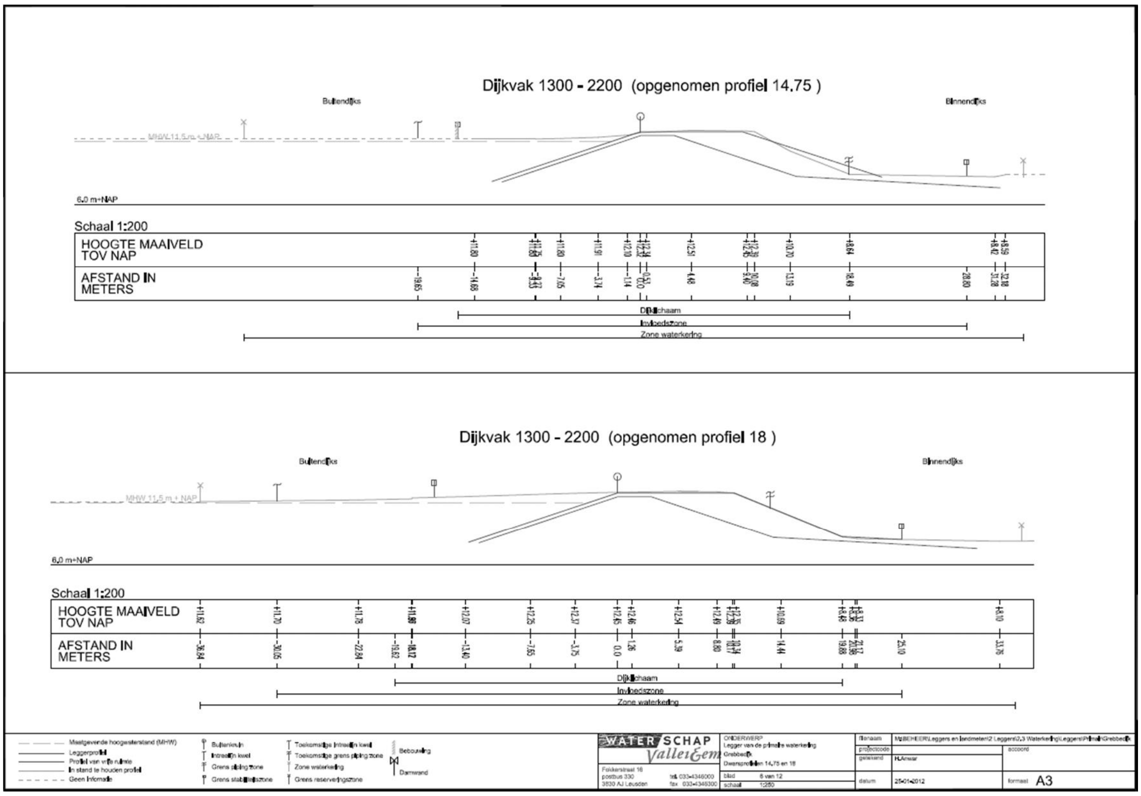


Figure B-32: Cross sections at 14.75 and 18 (WVE, 2012)

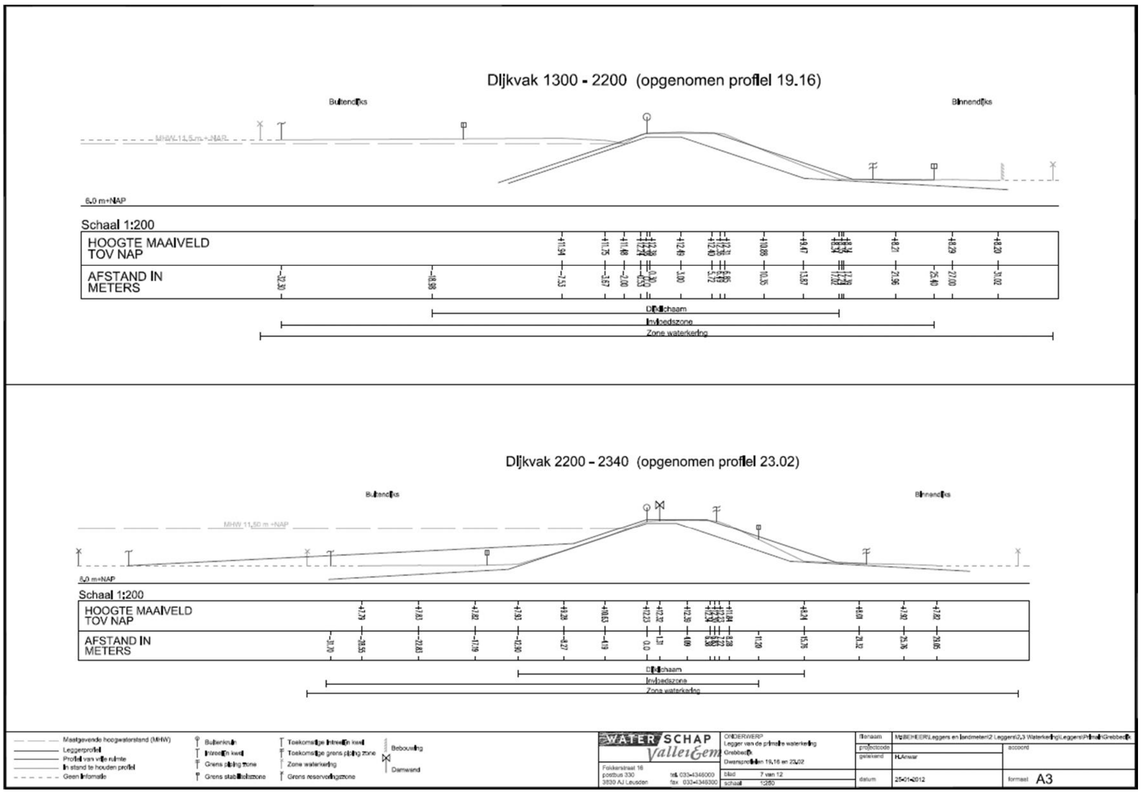


Figure B-33: Cross sections at 19.16 and 23.02 (WVE, 2012)

B.3.2 Soil information

The soil information that is available at the DINOloket from the surrounding area of the Grebbedijk is shown in Figure B-34.

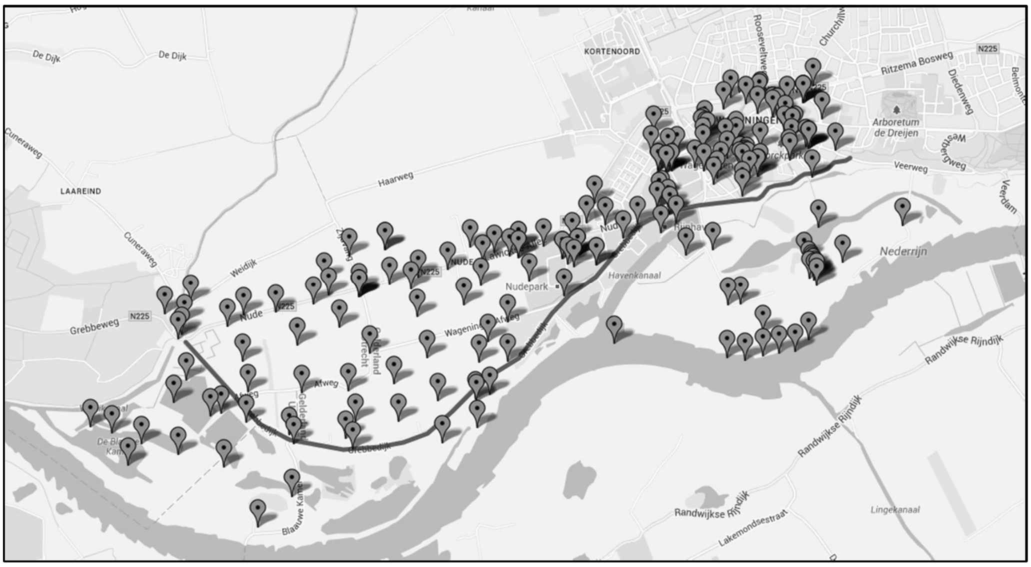


Figure B-34: Locations of soil information around the Grebbedijk

For the soil structure of the surrounding area of the marina en harbour is looked into the locations shown in Figure B-35.

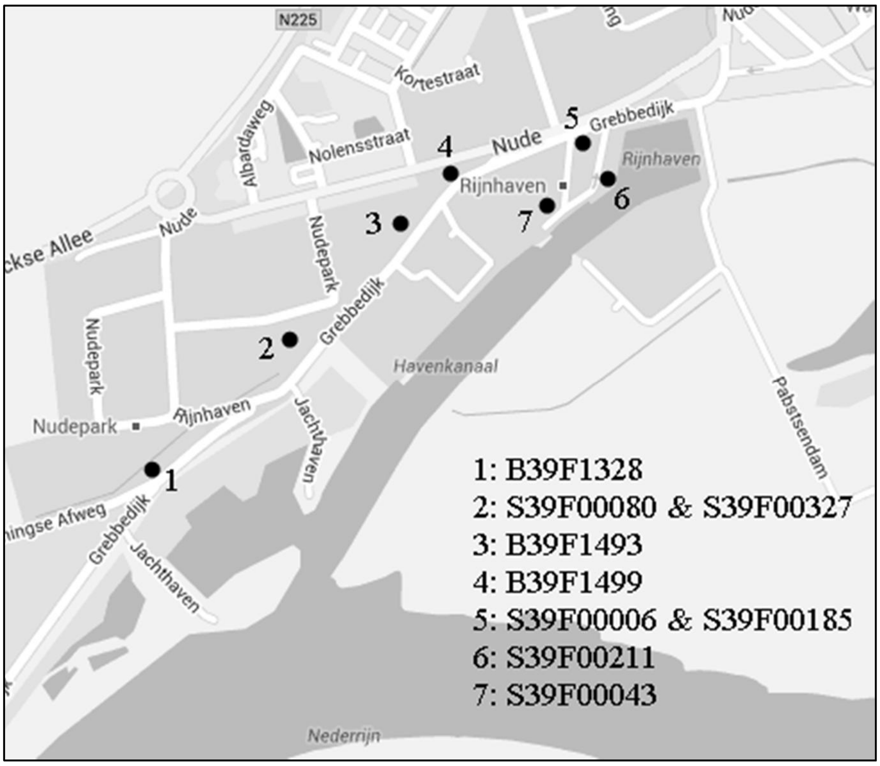


Figure B-35: Bore and probe numbers around the harbour and marina

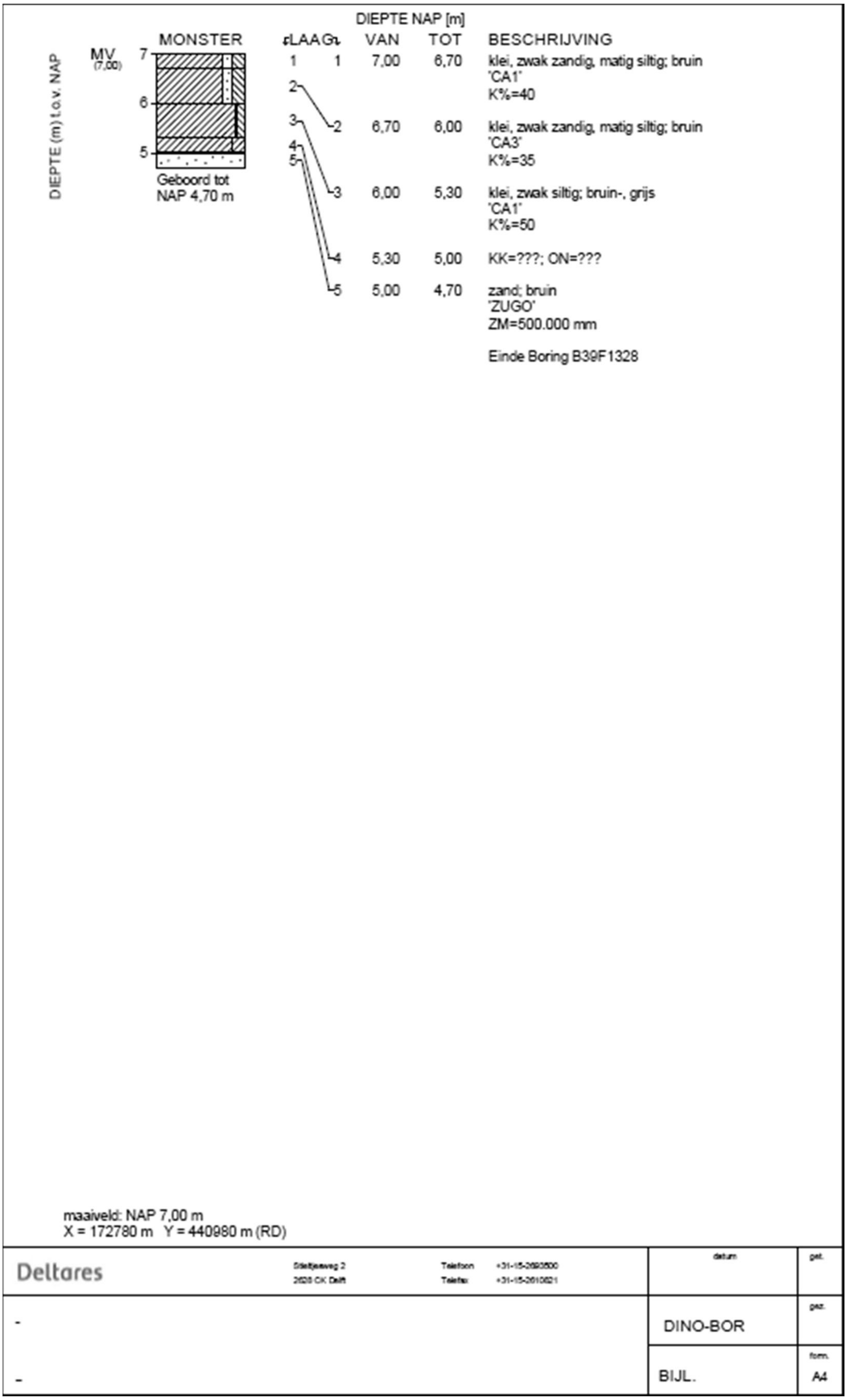
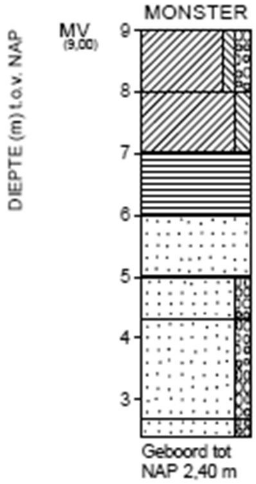


Figure B-36: Bore number B39F1328



DIEPTE NAP [m]

LAAG		VAN	TOT	BESCHRIJVING
1	1	9,00	8,00	klei, zwak siltig, grind; bruin 'GG2' K%=40
2	2	8,00	7,00	klei, matig siltig; bruin 'CA2' K%=45
3	3	7,00	6,00	veen; zwart
4	4	6,00	5,00	zand; grijs 'ZMGO' ZM=300.000 mm
5	5	5,00	4,30	ZG=???; grijs 'ZZGO' ZM=500.000 mm
6	6	4,30	2,70	ZG=???; grijs 'ZMGO' ZM=350.000 mm
7	7	2,70	2,40	ZG=???; ON=??? 'ZZFO' ZM=350.000 mm

Einde Boring B39F1493

maaiveld: NAP 9,00 m
X = 173170 m Y = 441360 m (RD)

Deltares Stieltjesweg 2 2628 CK Delft Telefoon +31-15-2693500 Telefax +31-15-2610821 | datum | get.

- | DINO-BOR | gez.

- | BIJL. | form. A4

Figure B-37: Bore number B39F1493

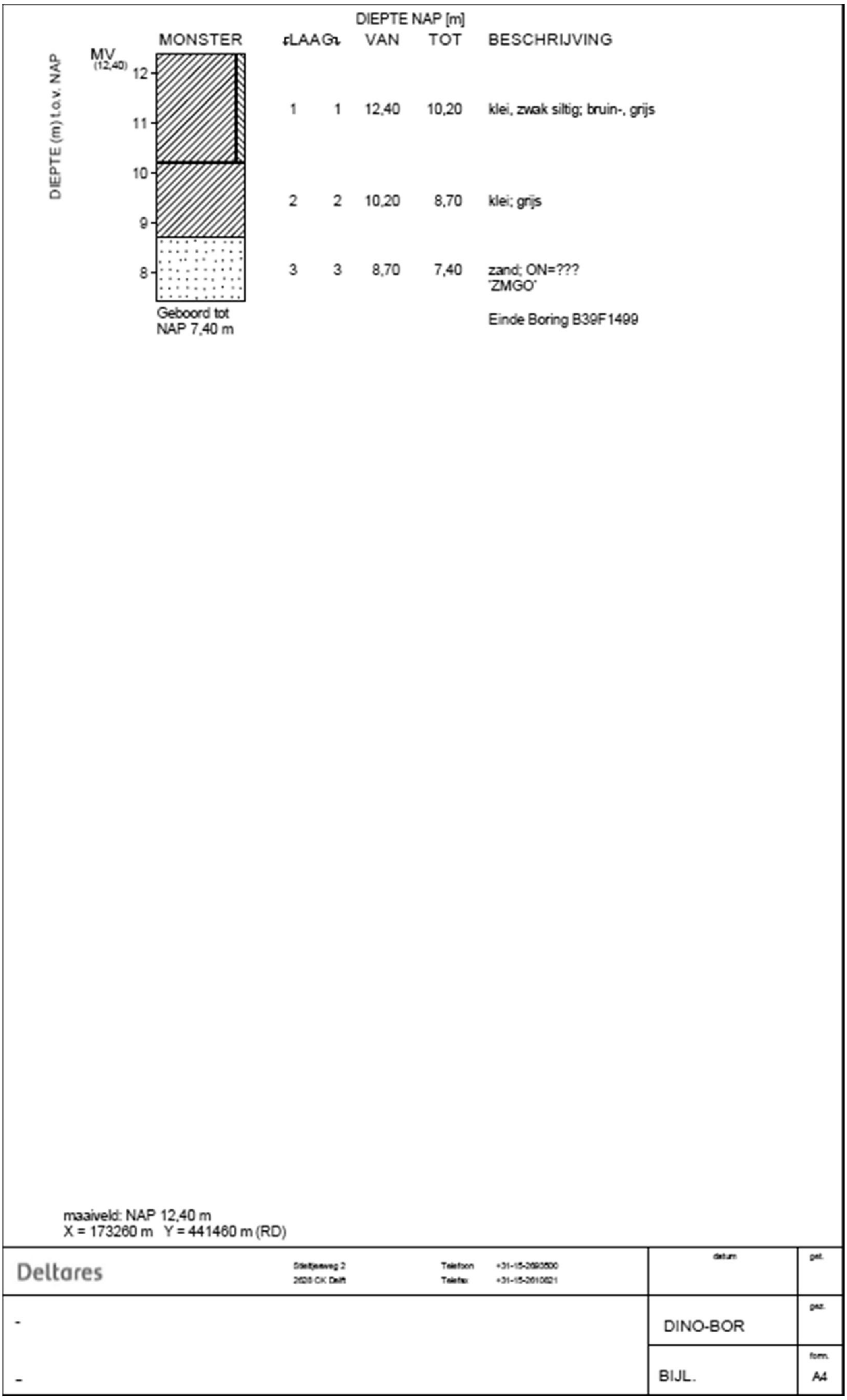


Figure B-38: Bore number B39F1499

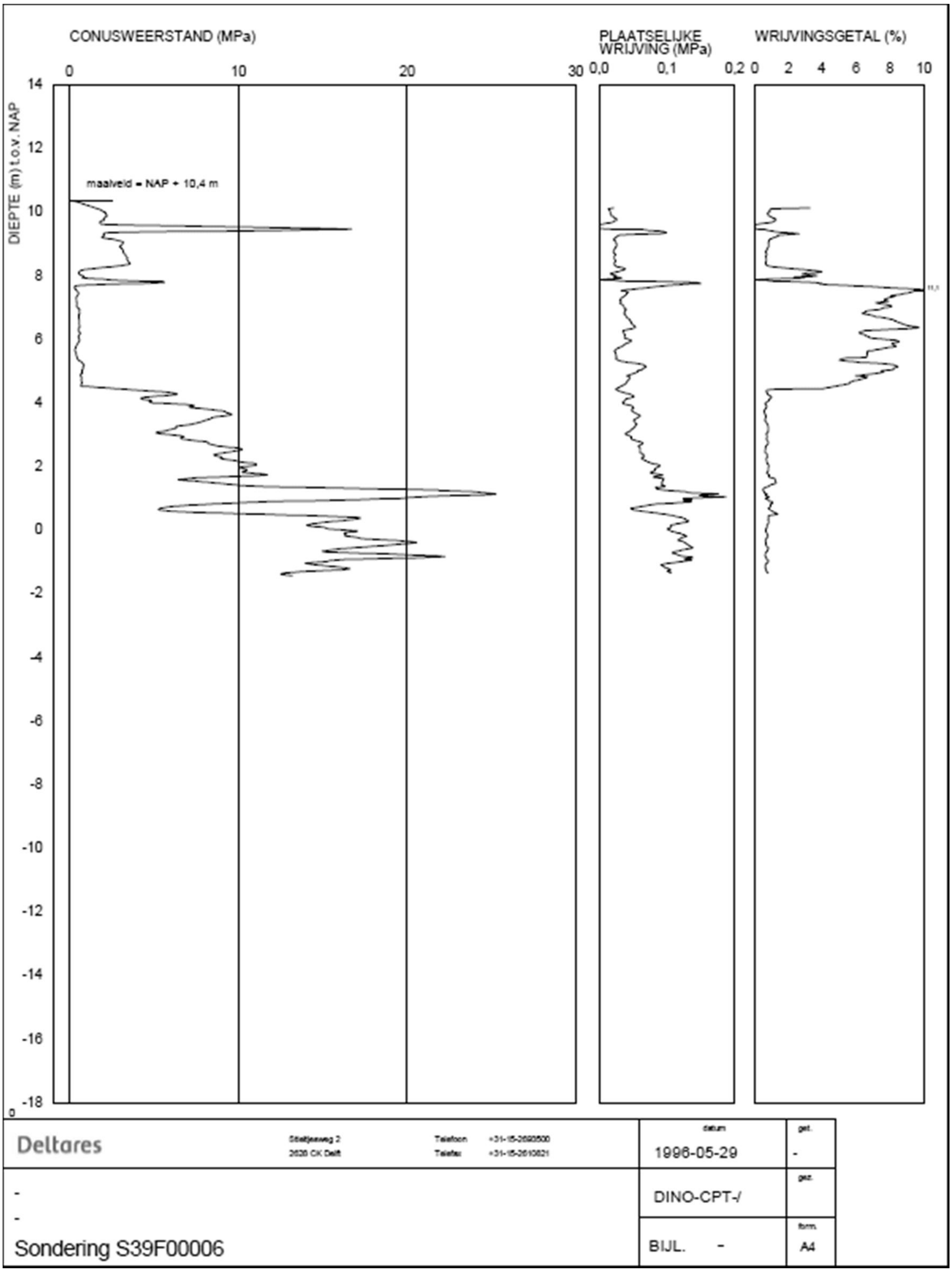


Figure B-39: Probe number S39F00006

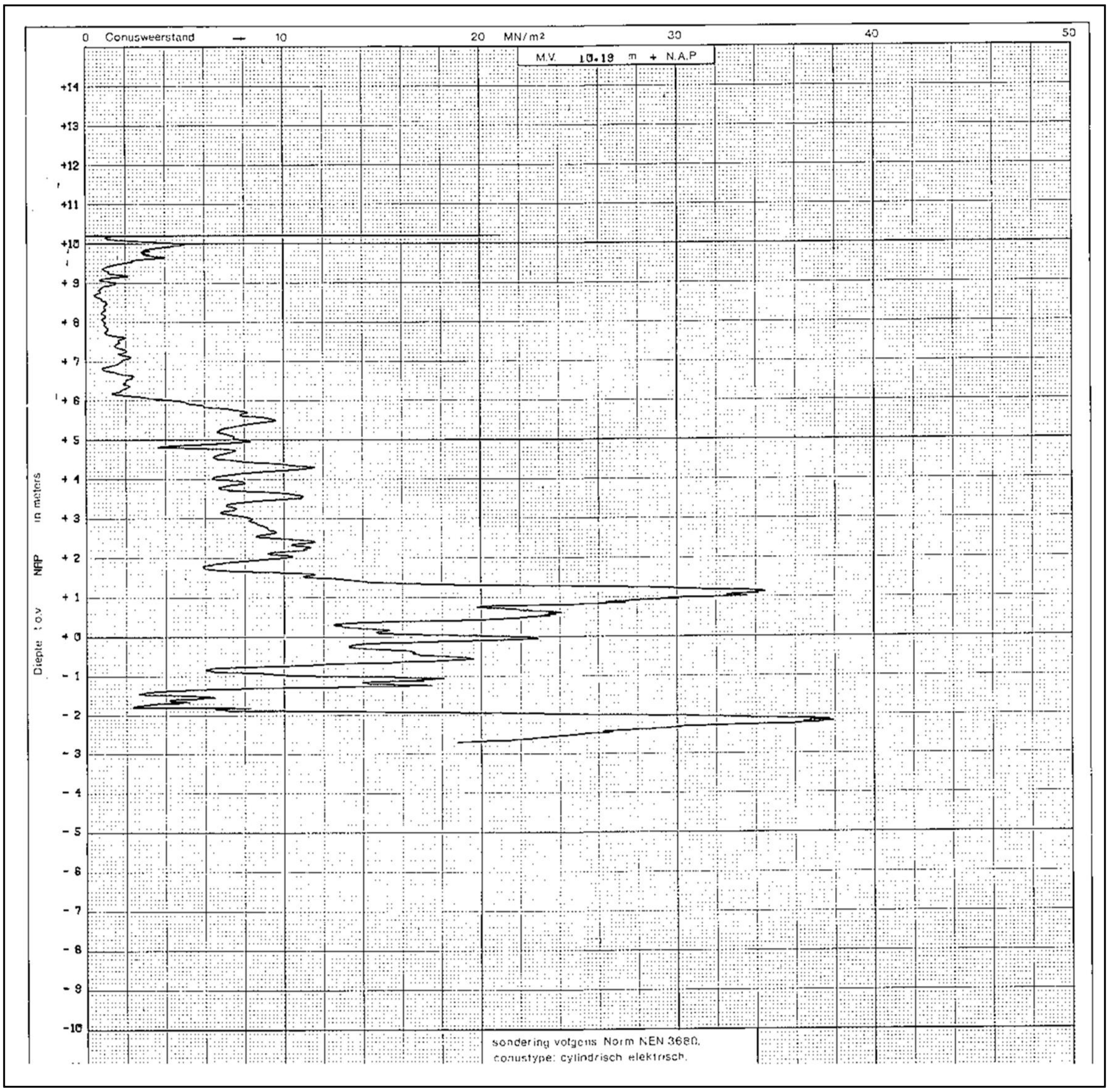


Figure B-40: Probe number S39F00043

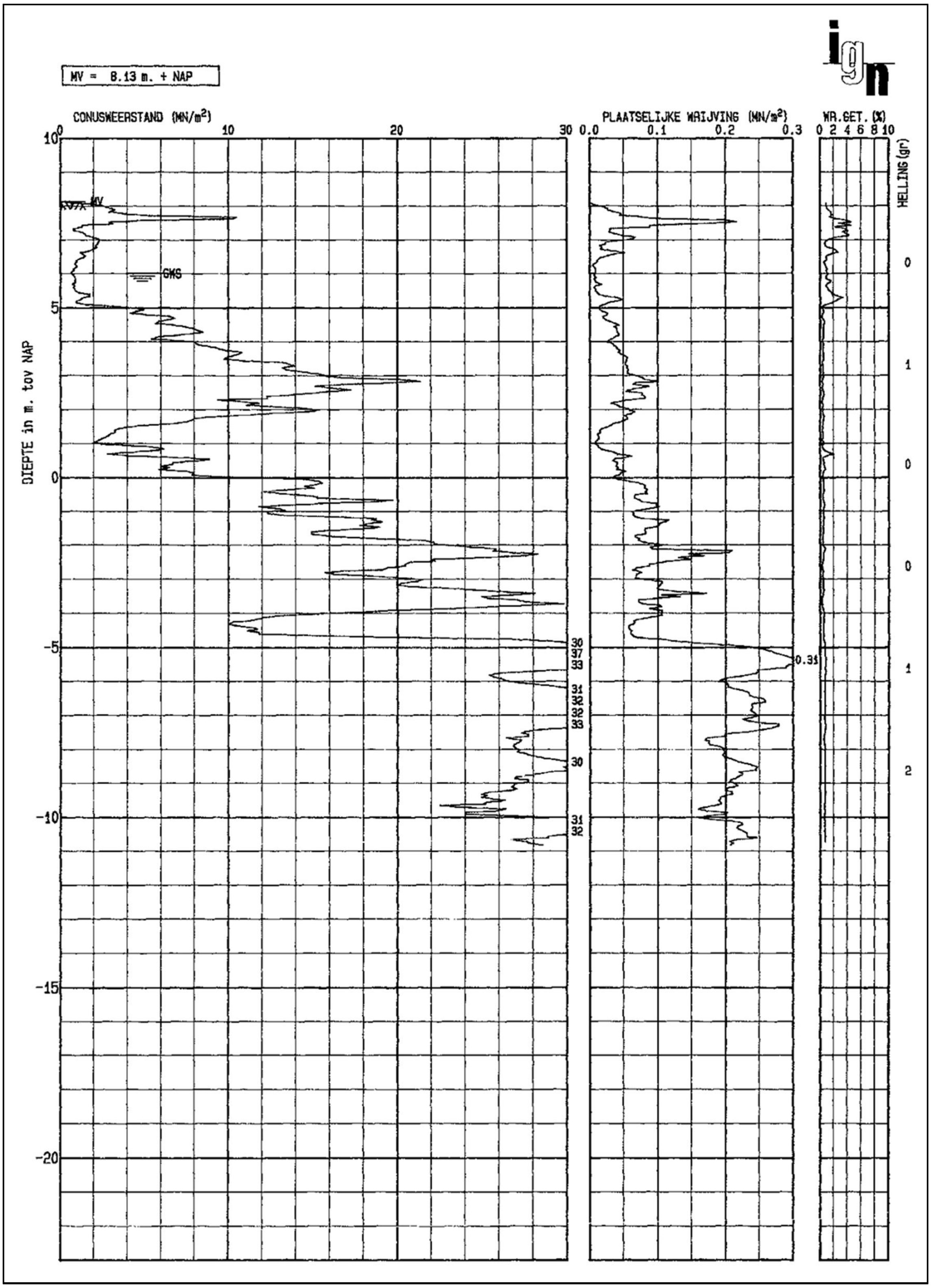


Figure B-41: Probe number S39F00080

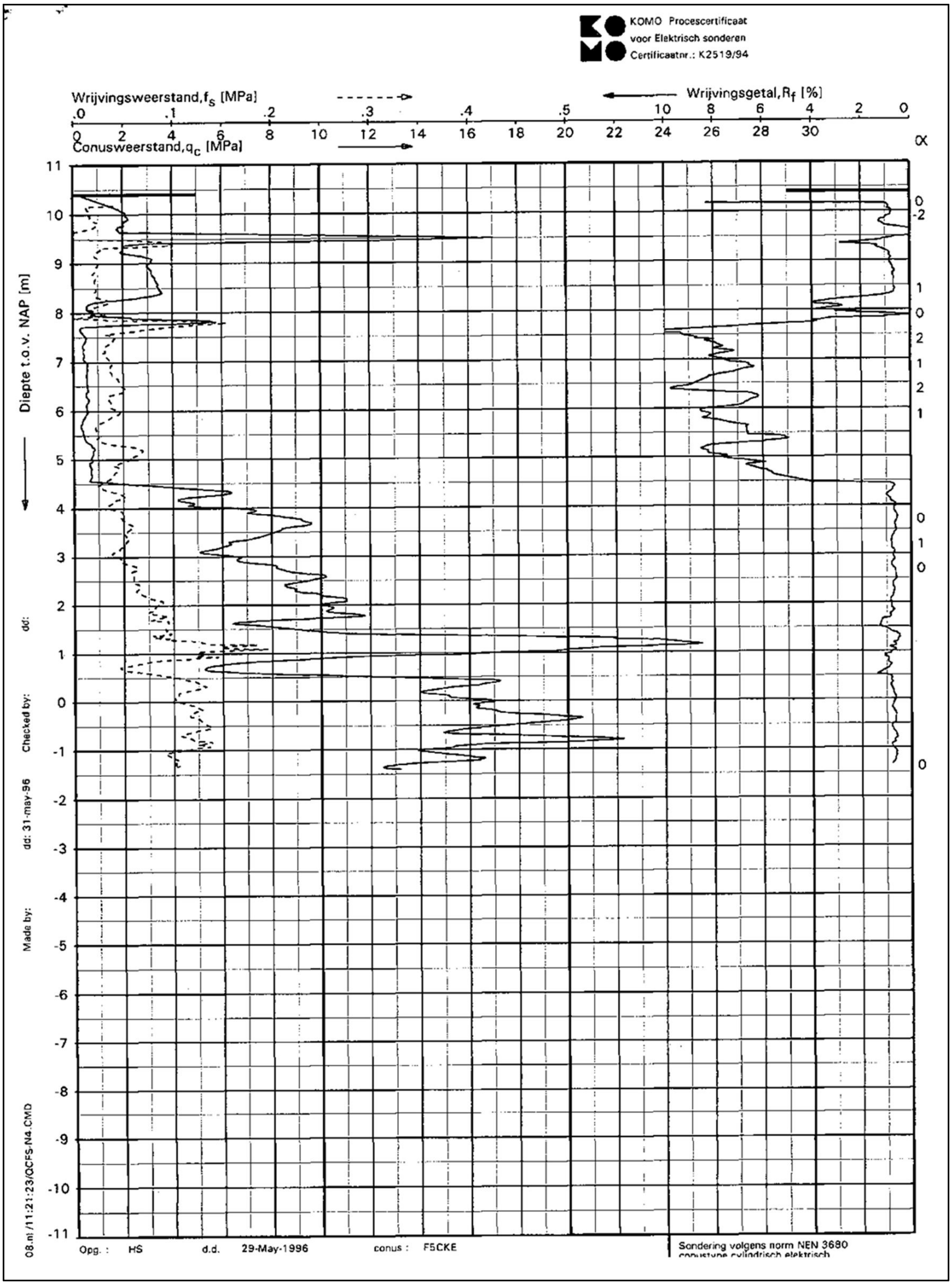


Figure B-42: Probe number S39F00185

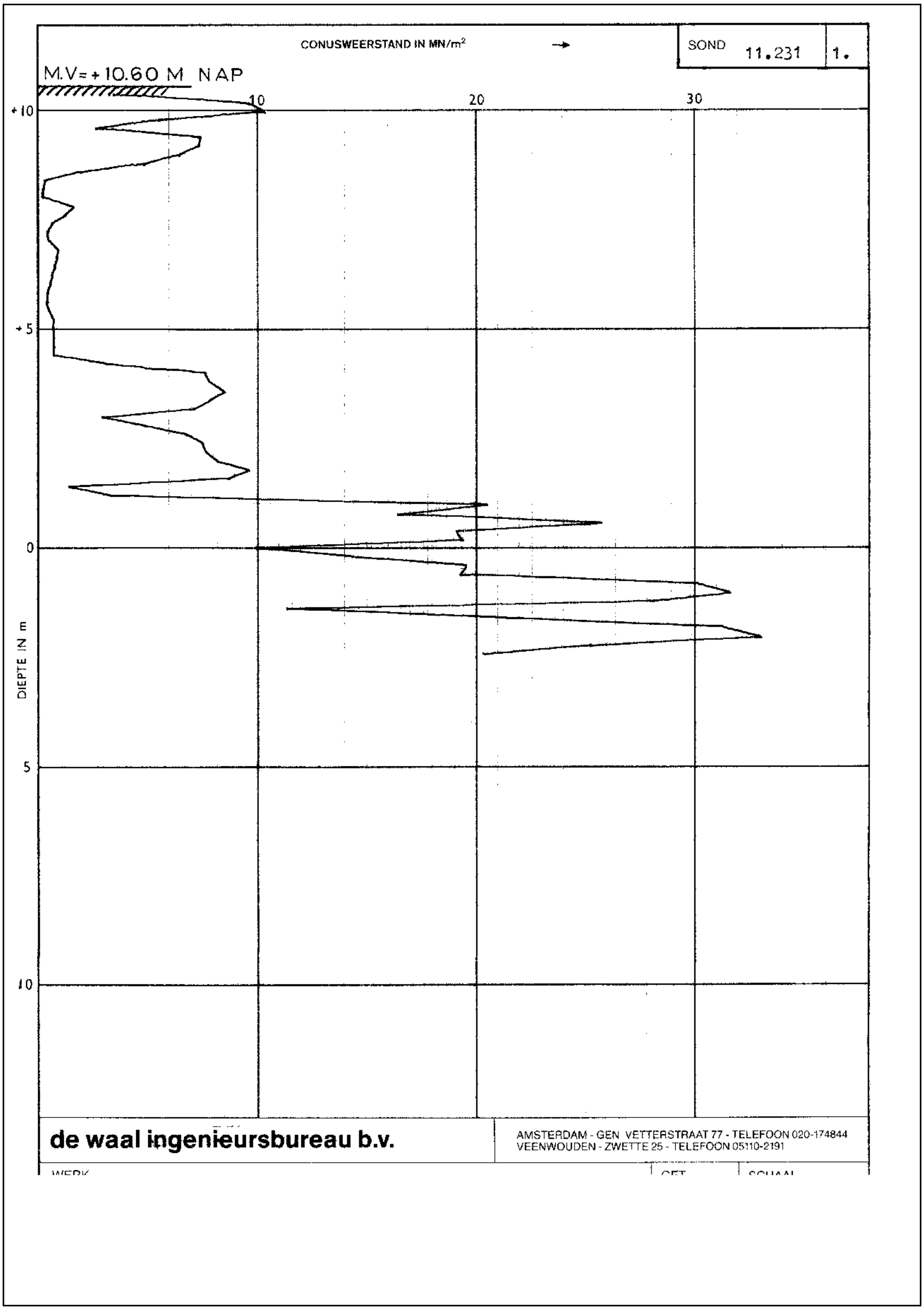


Figure B-43: Probe number S39F00211

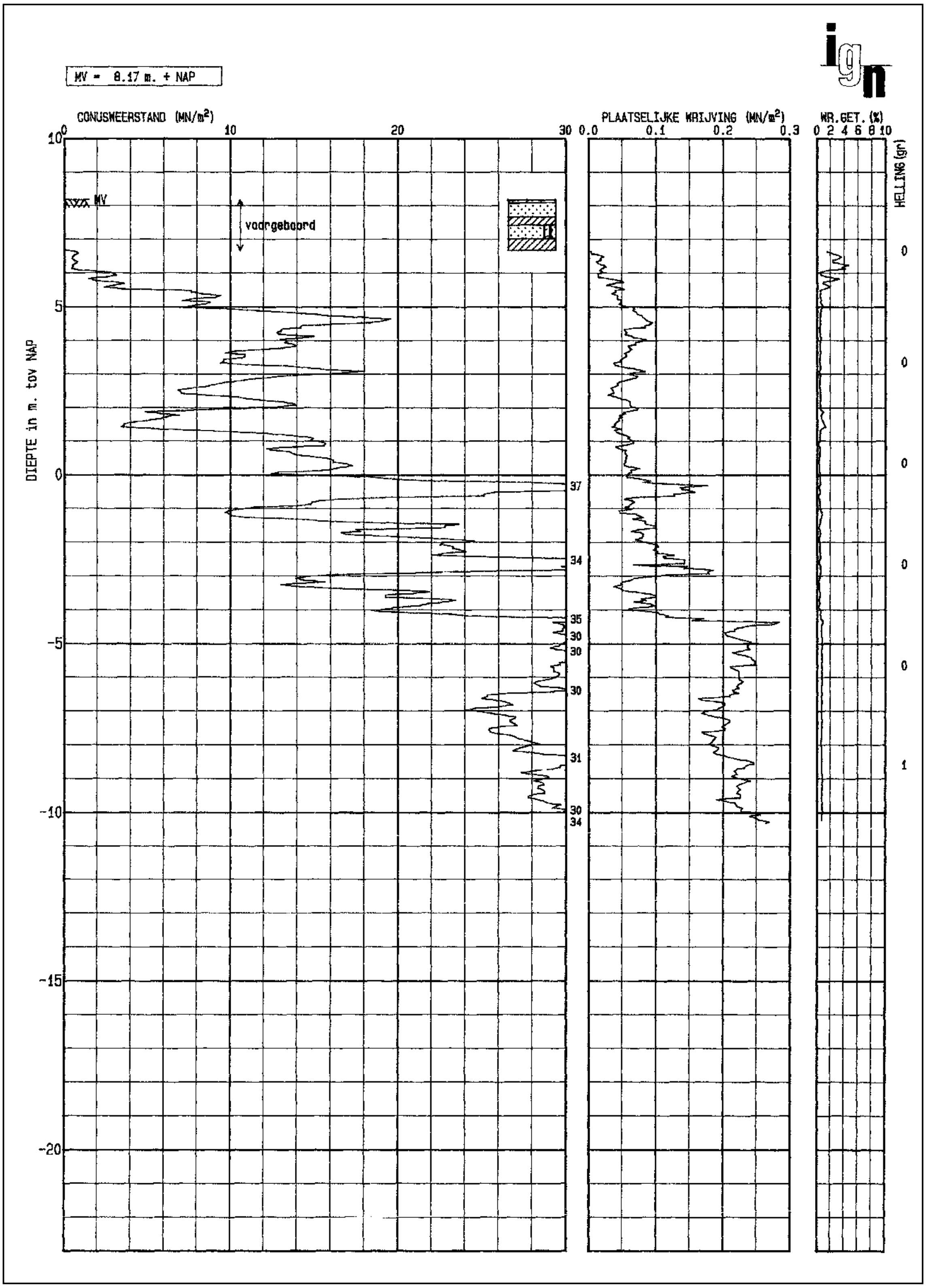


Figure B-44: Probe number S39F00327

The soil structure shown in Figure B-45 obtained from the bores and probes. The soil structure is a conservative assumption. For this first preliminary design this is sufficient but for the final design the soil structure requires a more accurate soil structure, also the differences along the length of the flood defence need to be taken into account.

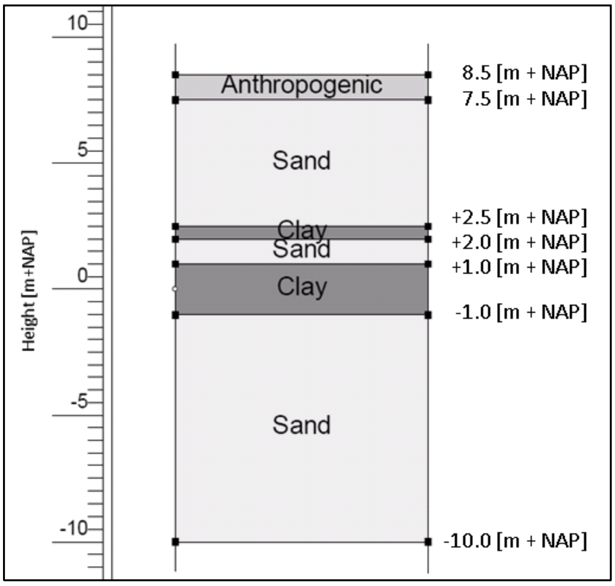


Figure B-45: Obtained soil structure

B.3.3 Ground water level

There was a groundwater level measuring instrument that measured from 1951 to 2000. The installation was located as presented in Figure B-46 and the data is shown in Figure B-47. The average groundwater level is 6.5 m+NAP.

Figure B-46: Location groundwater level measuring instrument

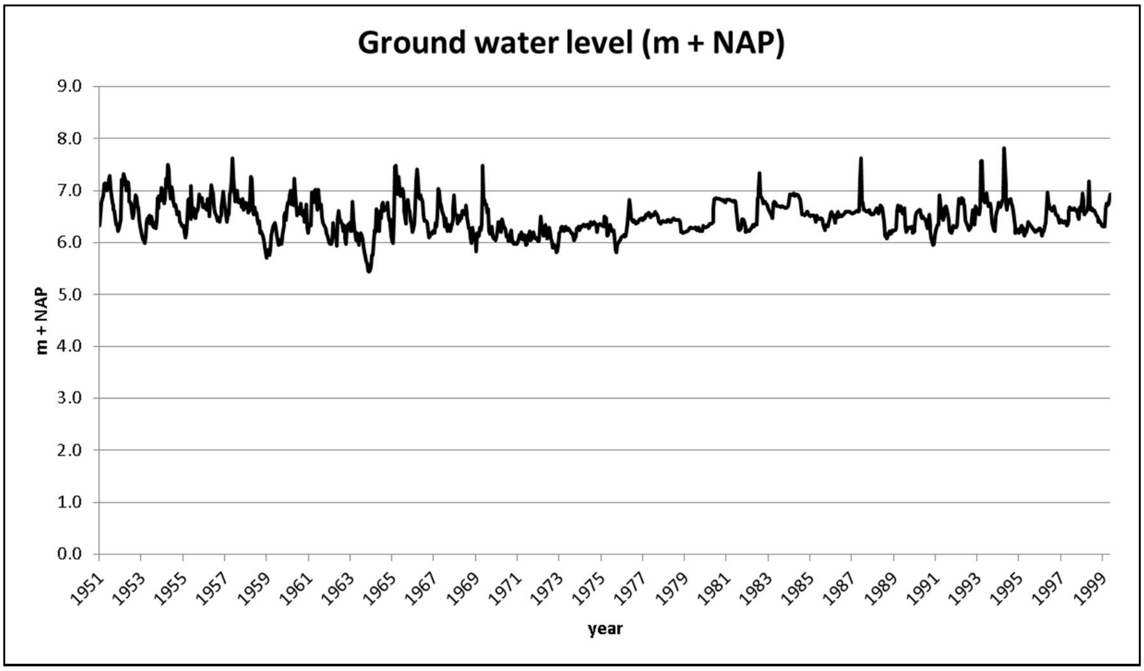


Figure B-47: Groundwater level from 1951 to 2000

C Calculations for the case study

This appendix holds the calculations for case study of the Grebbedijk. The calculations are divided in four paragraphs which hold the following calculations:

- Overtopping
- Piping
- Stability of the structure
- Strength of the structural elements

C.1 Overtopping

A couple of calculations are done for overtopping:

- Hand calculation
- PCOverslag
- Comparison with precipitation
- Storage on the crest

After the calculations a conclusion is presented.

C.1.1 Hand calculation

First a relatively simple hand calculation is done according to the guideline for wave run-up and wave overtopping (TAW, 2002). In this guideline is the following formula presented for the calculation of the amount of overtopping:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan(\alpha)}} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(-4.3 \cdot \frac{h_k}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$$

With:

$$\xi_0 = \frac{\tan(\alpha)}{\sqrt{s_0}}$$

$$s_0 = \frac{2 \cdot \pi \cdot H_{m0}}{g \cdot T_{m-1.0}^2}$$

Where:

q	overtopping discharge	$m^3/m/s$
g	gravitational acceleration	m/s^2
H_{m0}	significant wave height	m
ξ_0	breaker parameter	$-$
s_0	wave steepness	$-$
$T_{m-1.0}$	spectral wave period	s
$\tan(\alpha)$	angle of the slope	$-$
h_k	free crest height above the still water level	m
γ_b	influence factors for a berm	$-$
γ_f	influence factors for a roughness	$-$
γ_β	influence factors for a angle of wave attack	$-$
γ_v	influence factors for a vertical wall on slope	$-$

Using the following values for the parameters in the formula:

g	9.81	m/s^2
H_{m0}	0.5	m
ξ_0	1.30	$-$
s_0	0.07	$-$
$T_{m-1.0}$	2.2	s
$\tan(\alpha)$	0.33	$-$
h_k	1	m
γ_b	1	$-$
γ_f	1	$-$
γ_β	1	$-$
γ_v	1	$-$

Results in:

$$q \quad 0.92 \cdot 10^{-3} \quad m^3/m/s^2 \quad \approx \quad 0.92 \quad l/m/s^2$$

The amount of overtopping is in the order of the allowable amount of overtopping of 1 l/m/s and is even slightly smaller. In order to verify the hand calculation is the program PCOverslag used to calculate the amount of overtopping. The program PCOverslag is recommended by the guideline for wave run-up and wave overtopping (TAW, 2002) and is based on the same principles.

C.1.2 PCOverslag

The crest height of the flood defence is by a large extent determined by the overtopping criterion. Overtopping results in problems with the amount of water behind the dike and erosion of the inner slope. To do calculations on the overtopping criterion the software PCOverslag is used with the input data stated in Table C-1.

Table C-1: Input data PCOverslag

Parameter	Value	Unit
Crest height	13.0	m+NAP
Water level	12.0	m+NAP
Angle of slope	1:3	-
Wave height	0.5	m
Wave period	2.2	s
Storm duration	8	h

The results of the calculation are summarized in Table C-2 and Figure C-1, the full report of the calculation in shown in Figure C-2. The calculated crest height of 2% wave run-up is higher than the assumed crest height of the dike in the input data of the program.

Table C-2: Results of the PCOverslag calculation

Parameter	Value	Unit
Percentage overtopping	3.26	%
2% wave run-up	1.07	m
Average overtopping discharge	0.11	l/s/m
Crest height 2% wave run-up	13.07	m+NAP
Crest height (0.1 l/m/s overtopping)	13.01	m+NAP
Crest height (1 l/m/s overtopping)	12.70	m+NAP
Crest height (10 l/m/s overtopping)	12.38	m+NAP

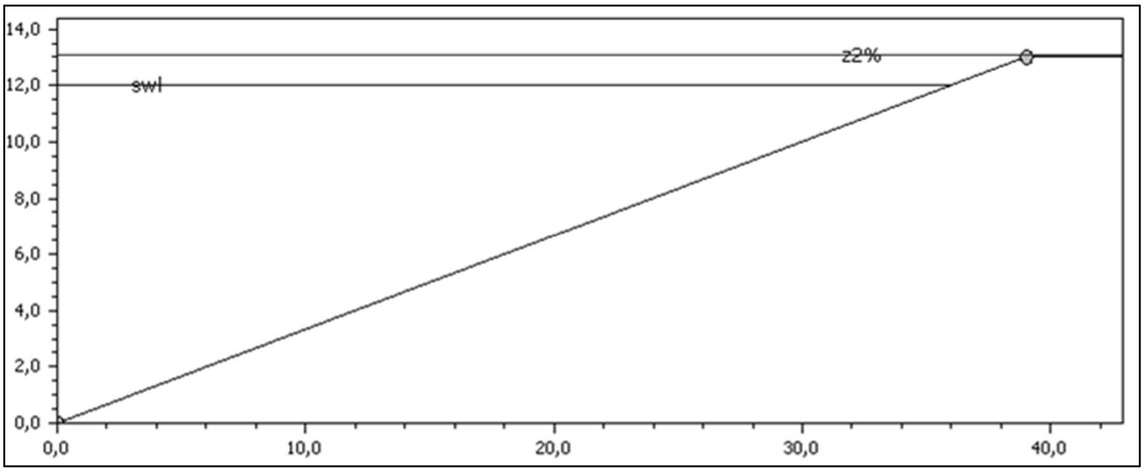


Figure C-1: Result of the PCOverslag calculation

However, the software does not include the width of the crest in the calculation, which influences the overtopping of the dike. The large width ensures that the wave energy dissipates while flowing over the dike. Part of the volume of the wave will infiltrated in the soil and part of it will flow back towards the river side, especially if the crest has a small slope towards the river side.

Hydraulische Randvoorwaarden

Significante Golfhoogte	Hm0	0.50 m
Golf Periode	Tm-1,0	2.20 s
Piek Periode	Tp	2.42 s
Gemiddelde Periode	Tm	2.20 s
Golf Richting	ß	0.00 °
Storm Duur	tsm	28800 s
Stil Waterlijn	SWL	12.00 m+NAP

Segmenten

Aantal Segmenten: 1

Start van segmenten: X: 0.00, Y: 0.00

Segment	x	y	tangens helling	ruwheids factor	materiaal
0	39.00	13.00	0.33	1.00	Gras, gezaaid

Benodigde kruinhoogte

Overslag	Kruinhoogte
[l/s/m]	[m+NAP]
0.10	13.01
1.00	12.70
10.00	12.38
50.00	12.16

Overslag hoeveelheden per golf

Percentage	Hoeveelheid
[%]	[l/m]
Vmax	68.16
1.00	47.31
10.00	18.78

Berekenings resultaten

Commentaar	De 2%-golfoploop is hoger dan de dijk
Z2Perc	1.07 m
Z2Perc+SWL	13.07 m+NAP
Overslag	0.11 l/s/m
Percentage overslaande golven	3.26 %
Vmax	68.16 l/golf/m

Dwarsprofiel berekening

Z2perc	1.07 m
Ksio	1.30 -
L0	7.55 m
GammaB	1.00 -
GammaF	1.00 -
GBeta oploop	1.00 -
GBeta overslag	1.00 -
TanAlpha	0.33 -
Iteraties	3 -

Figure C-2: Report of the PCOverslag calculation

C.1.3 Comparison with precipitation

Overtopping results in erosion on the inner slope and problems with the amount of water in the hinterland. The crest length of this flood defence is very long compared to a traditional dike design, so the overtopping discharge calculated on the outer crest line is not representable for the overtopping of the inner crest line. In order to take the length of the crest into account, the overtopping volume is compared to precipitation. If the overtopping discharge is in the order of magnitude of the precipitation volume the amount of water that overtops the inner crest line is negligible. The average overtopping volume is divided over the length of the crest to see it as precipitation, as can be seen in Figure C-3.

L	length of the crest
q	average overtopping discharge
$p = q/L$	average precipitation

$$L = 30 \quad m$$
$$q = 1 \quad l/m/s \rightarrow p = 2 \quad l/m^2/min = mm/min$$
$$q = 0.1 \quad l/m/s \rightarrow p = 0.2 \quad l/m^2/min = mm/min$$

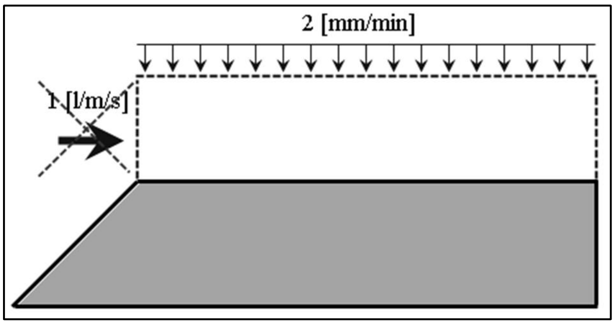


Figure C-3: Schematization of the overtopping discharge as precipitation

For the comparison with precipitation the statistics of extreme precipitation for short and long term in the Netherlands are analysed. In Figure C-4 the overtopping discharge (schematized as precipitation) is drawn against the short term (0 to 120 minutes) statistics of extreme precipitation. As can be concluded for the figure, an overtopping discharge of 1 l/m/s is only valid for the first 20 minutes of a storm, too short for this purpose. A lower overtopping discharge is necessary to be able to compare this with precipitation. An overtopping discharge of 0.1 l/m/s can be compared to precipitation that occurs about once every 3 to 4 years for the duration of 120 minutes, so this is a valid schematization.

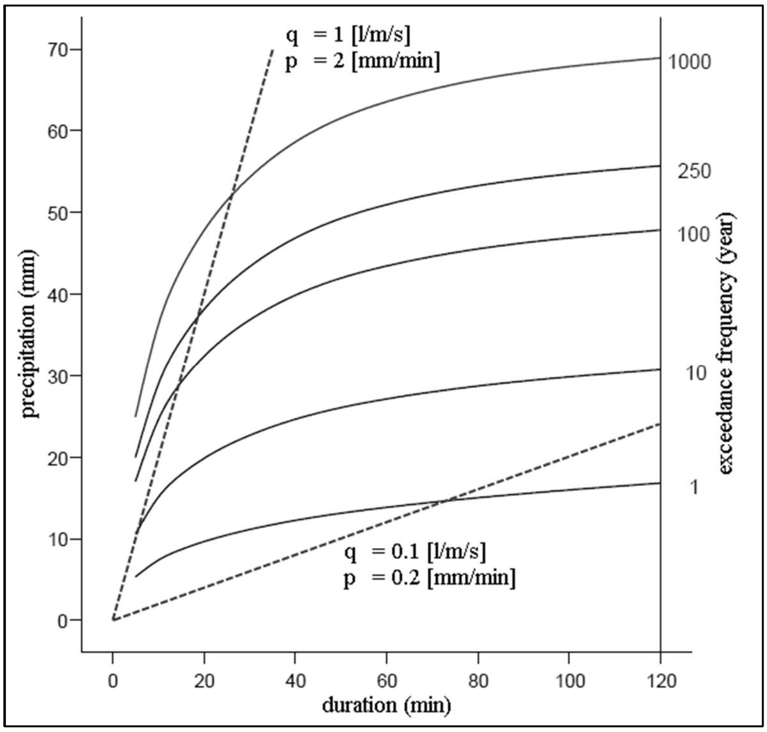


Figure C-4: Statistics of extreme precipitation, short term (Buishand & Wijngaard, 2007)

For short term statistics the overtopping discharge of 0.1 l/m/s is valid, but the duration of a storm with a design water level is somewhat longer. For the long term statistics of precipitation only the 0.1 l/m/s overtopping discharge is considered because for 1 l/m/s the short term statistics were not applicable. The long term statistics are presented in Table C-3 as they are found in the literature.

Table C-3: Statistics of extreme precipitation, long term (Smits et al., 2004)

	Uren				Dagen			
Jaar	4	8	12	24	2	4	8	9
10x per jaar	9	12	13	15	19	-	-	-
5x per jaar	12	15	17	21	26	33	43	45
2x per jaar	16	20	23	28	35	45	61	64
1x per jaar	21	24	27	33	41	52	71	75
1x per 2 jaar	25	29	32	39	48	60	81	86
1x per 5 jaar	31	36	40	47	58	71	94	99
1x per 10 jaar	36	41	46	54	65	80	103	109
1x per 20 jaar	41	47	52	61	73	89	113	118
1x per 25 jaar	43	49	54	63	75	91	115	121
1x per 50 jaar	49	56	61	71	84	100	124	130
1x per 100 jaar	55	62	68	79	92	109	133	138
1x per 200 jaar	61	69	75	87	101	118	141	146
1x per 500 jaar	71	79	86	98	113	130	152	156
1x per 1000 jaar	78	88	95	108	123	140	159	163

In Figure C-5 the data from Table C-3 is plotted against the schematized precipitation of the overtopping discharge of 0.1 l/m/s. The line that represents the overtopping discharge crosses the line of the once in 1000 years exceedance probability of the precipitation roughly at a duration of 7 hours.

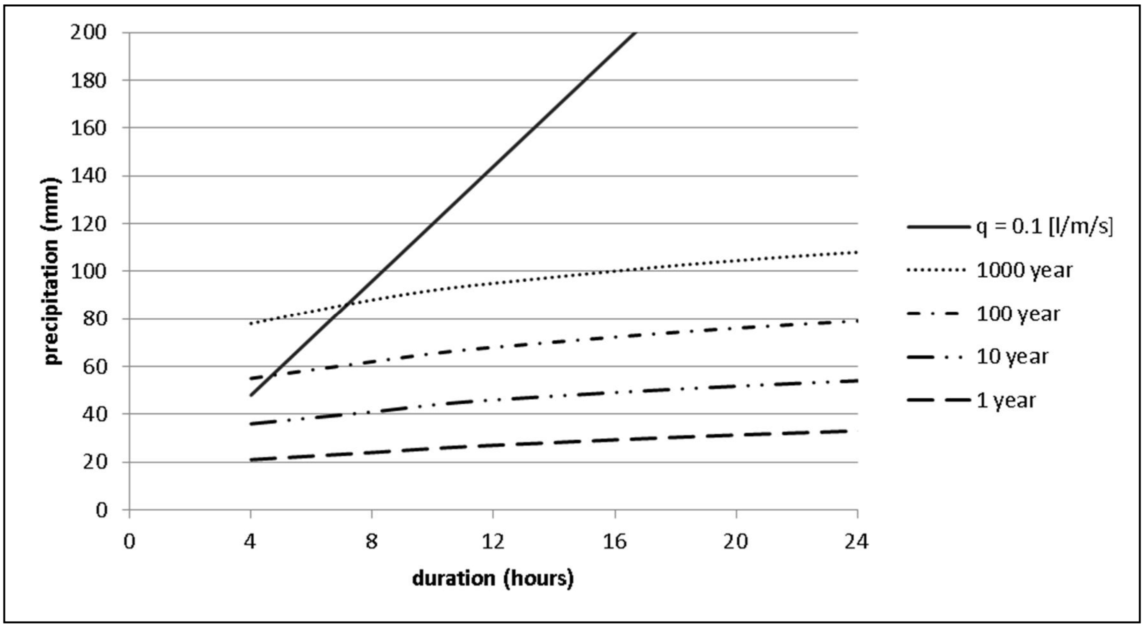


Figure C-5: Overtopping discharge plotted against the long term precipitation statistics

The duration of a storm during design conditions is 8 hours, so even with an overtopping discharge of only 0.1 l/m/s the schematisation as precipitation is questionable, because it would represent precipitation that occurs once in a 1000 years.

C.1.4 Storage on the crest

The volume of water overtopping the outer crest line will be stored on the crest before it is overtopping the inner crest line. Especially the reduction of wave energy on the crest and the slightly higher level of the concrete border on the inner crest line prevent the water from overtopping the inner crest line. To analyse how high this boarder should be is assumed that the volume of water that passes the outer crest line during an 8 hour storm is stored on the crest. Also the conservative assumption is done that the volume of water does not flow back towards the river side or infiltrates the soil. The schematisation of this idea is presented in Figure C-6.

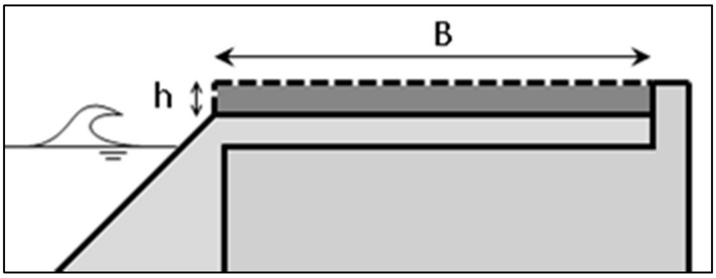


Figure C-6: Volume storage on top of the dike

Calculating the height of the water volume on top of the flood defence, the same height as the border on the inner crest line, is described by the following formula:

$$h = \frac{q \cdot t}{B}$$

Where:

h	height of the water volume	[m]
q	overtopping discharge at the outer crest line	[l/m/s]
t	duration of the storm	[s]
B	width of the flood defence	[m]

Results in:

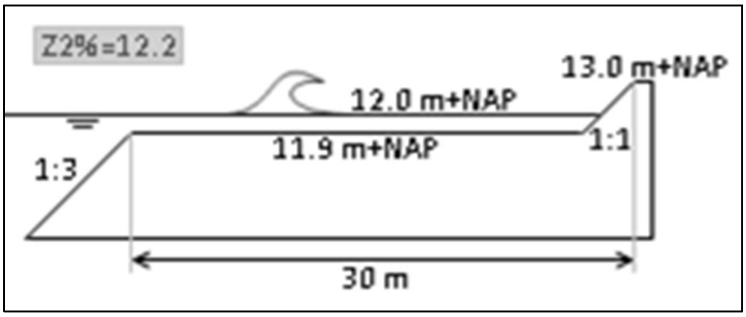
$$q = 1.0 \quad \rightarrow \quad h = \frac{1.0 \cdot 8 \cdot 3600}{29 \cdot 100} \approx 10 \text{ [m]}$$

$$q = 0.1 \quad \rightarrow \quad h = \frac{0.1 \cdot 8 \cdot 3600}{29 \cdot 100} \approx 1 \text{ [m]}$$

The calculations with both overtopping discharges do lead to not acceptable large border heights, so the calculated board heights are not used. The overtopping discharge on the inner crest line is not exactly known but with this design the overtopping discharge will probably be zero during the design conditions.

Conclusion

With the previous comparisons could not be drawn a conclusive statement about the overtopping on the inner crest line of the flood defence. Changing the dimensions of the crest to a situation that can be simulated with the help of PCOverslag can give a better insight in the overtopping on the inner crest line. The inner crest line is now schematised as the overtopping boundary, the crest level is 0.1 m lower than the design water level and the wave properties are not changed. The result of this schematisation leads to a height of 12.2 m+NAP, so below the levels of the overtopping discharges of 1.0 and 0.1 l/m/s for the previous schematisation. The software is not intended to be used in this way but gives a good impression of the effect of a very long crest; in this case the program schematises the crest a berm.



This last calculation is not reliable calculation so the original schematisation is still used in the further design of this flood defence. Optimisation of the design can be done by modelling the long crest and trying to find the actual overtopping discharge on the inner crest line. Besides that, could the allowable overtopping discharge be analysed, depending on the strength of the surface area behind the structure.

C.2 Piping calculations

C.2.1 Empirical calculation rule of Bligh

The empirical rule of Bligh is used to check whether this design satisfies the piping requirement (TAW, 1999):

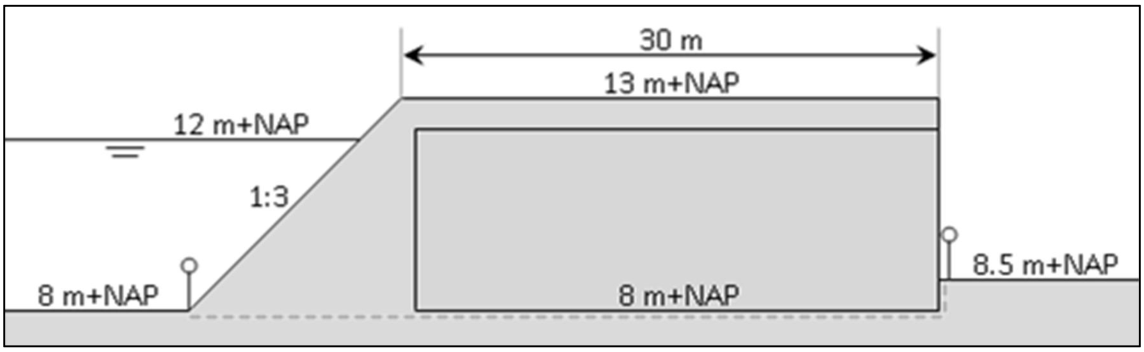
$$L = \sum L_v + \sum L_h \geq C_B \cdot \Delta H$$

Where:

L	total seepage length	[m]
$\sum L_v$	vertical seepage length	[m]
$\sum L_h$	horizontal seepage length	[m]
C_B	Bligh constant	[–]
ΔH	headdrop over the flood defence	[m]

With:

$\sum L_V$	$= 0.5$	[m]
$\sum L_h$	$= 30 + 3 \cdot (13 - 8) = 45$	[m]
L	$= 45.5$	[m]
ΔH	$= 12 - 8.5 = 3.5$	[m]



Results in:

$C_B = 12$ (coarse sand) $\rightarrow$ $L \geq C_B \cdot \Delta H$ $\rightarrow$ $45.5 \geq 42$

$C_B = 15$ (fine sand) $\rightarrow$ $L \geq C_B \cdot \Delta H$ $\rightarrow$ $45.5 \leq 52.5$

As can be seen in the calculations above the calculation depends on the Bligh constant chosen for the aquifer. For the lower value of the Bligh constant the criterion is met but for the higher value the criterion is not met.

C.2.2 Empirical calculation rule of Lane

The piping rule from Lane takes the difference into account for horizontal and vertical seepage lengths. The vertical length is of more importance than the horizontal length, which becomes important if a sheet pile wall is applied for an increase of seepage length, but first without the sheet pile wall:

$$L = \sum L_v + \frac{1}{3}\sum L_h \geq C_L \cdot \Delta H$$

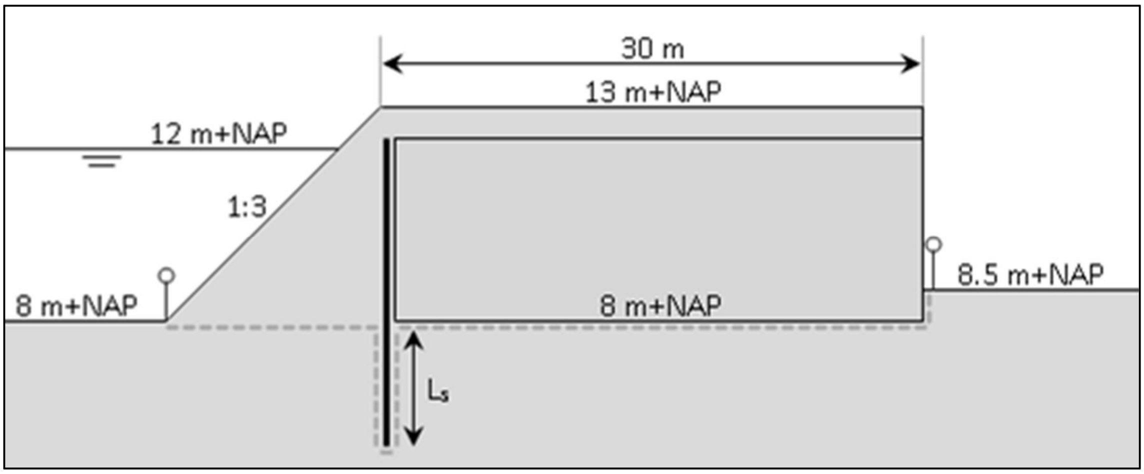
With:

$$\begin{array}{lll} \sum L_V & = 0.5 & [m] \\ \sum L_h & = 30 + 3 \cdot (13 - 8) = 45 & [m] \\ L & = 15.5 & [m] \\ \Delta H & = 12 - 8.5 = 3.5 & [m] \end{array}$$

Lane also defined other constants, resulting in:

$$\begin{array}{lllll} C_L = 5 \text{ (coarse sand)} & \rightarrow & L \geq C_L \cdot \Delta H & \rightarrow & 15.5 \leq 17.5 \\ C_L = 6 \text{ (middle fine sand)} & \rightarrow & L \geq C_L \cdot \Delta H & \rightarrow & 15.5 \leq 21.0 \\ C_L = 7 \text{ (fine sand)} & \rightarrow & L \geq C_L \cdot \Delta H & \rightarrow & 15.5 \leq 24.5 \end{array}$$

As can be seen in the calculations above the flood defence fails according to the rule of Lane. In order to fulfil the rule of Lane the length of a sheet pile wall is calculated.



$$\begin{array}{lllllll} C_L = 5 & \rightarrow & 17.5 - 15.5 = 2 & \rightarrow & L_S = 1.0 & [m] \\ C_L = 6 & \rightarrow & 21.0 - 15.5 = 5.5 & \rightarrow & L_S = 2.75 & [m] \\ C_L = 7 & \rightarrow & 24.5 - 15.5 = 9 & \rightarrow & L_S = 4.5 & [m] \end{array}$$

To get a better insight in the failure mechanism of piping a calculation is made with the software MSeep. In the next paragraph is the input data for the MSeep program given and paragraph thereafter treats the output of the MSeep program.

C.2.3 Input MSeep

In this paragraph is the input data stated for the MSeep calculation. First of all the soil structure is shown in Figure C-7.

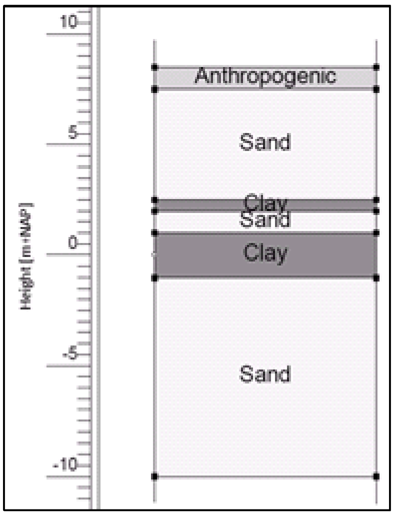


Figure C-7: Input soil structure

Together with the input date from Table C-4 the cross section shown in Figure C-8 is created. The exact soil profile under the flood defence is not known. For the piping calculation is this conservative soil profile assumed.

Table C-4: Input data

Parameter	Value	Unit
Crest height	13.0	m+NAP
Design water level	12.0	m+NAP
Water level land side	8.5	m+NAP
Surface level land side	8.5	m+NAP

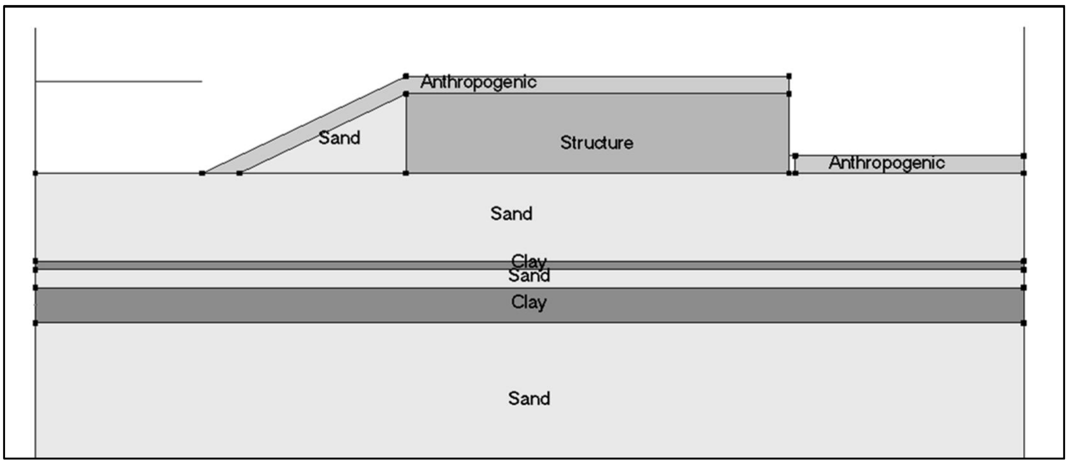


Figure C-8: Overview input data

The next thing to do is determine the material parameters for the input of the program, see Figure C-8.

Table C-5: Material parameters input

Name	Horizontal permeability	Vertical permeability	Gamma-sat	Porosity	Particle diameter	Bedding angle
	[m/s]	[m/s]	[kN/m^3]	[-]	[µm]	[°]
Sand	$1*10^{-4}$	$1*10^{-4}$	20	0.3	200	35
Structure	$1*10^{-12}$	$1*10^{-4}$	20	0.0	10000	72
Clay	$1*10^{-7}$	$1*10^{-7}$	17	0.4	4	20
Anthropogenic	$1*10^{-5}$	$1*10^{-5}$	17	0.4	100	30

The program also needs a boundary line for the erosion and the boundary condition on the enclosing lines, see Figure C-9. All the enclosing lines are given the property of a closed boundary, except two. The left line (number 2) is given the property of a boundary with potential; the design water level. The right line (number 7) has also the property of a boundary with potential, in this case equal to the surface level.

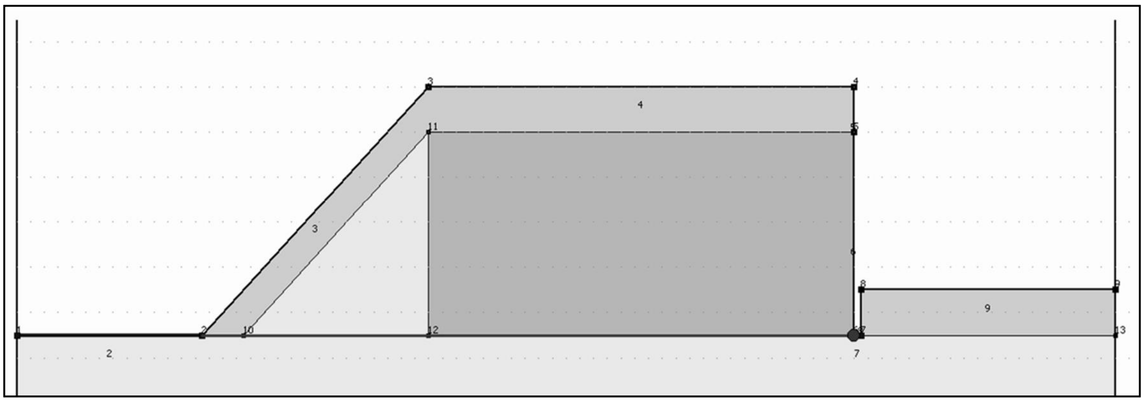


Figure C-9: Erosion input

C.2.4 Output MSeep

The calculation output of MSeep is shown below. The potentials and flow velocities are presented in Figure C-10 and Figure C-11. It clearly shows the flow through the top sand layer and the decrease of potential. The first figure shows that in case of no sheet pile wall the critical head is equal to 10.2 m+NAP, so that is lower than the design water level of 12.0 m+NAP. The second figure shows that the application of a sheet pile down till 6.5 m+NAP increases the critical water level to 12.9 m+NAP, which is larger than the design water level.

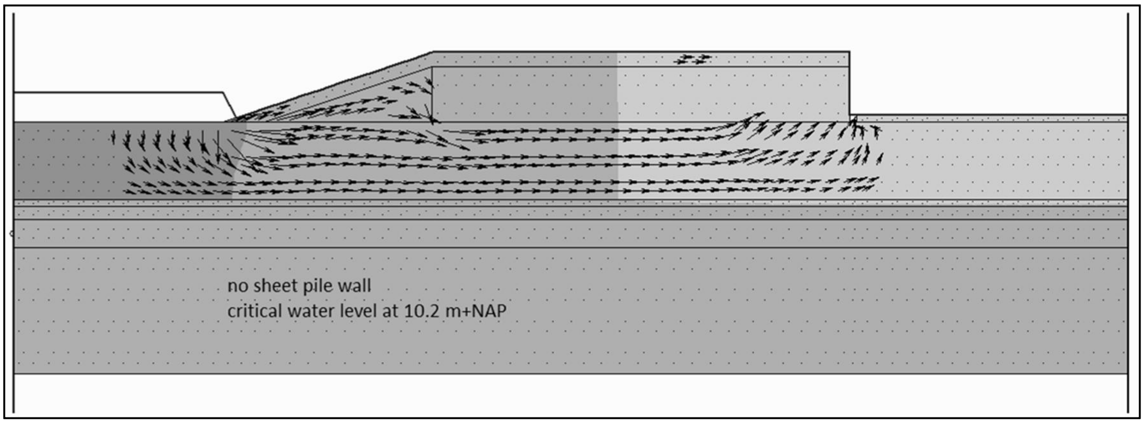


Figure C-10: Potentials and flow velocities in case of no sheet pile wall

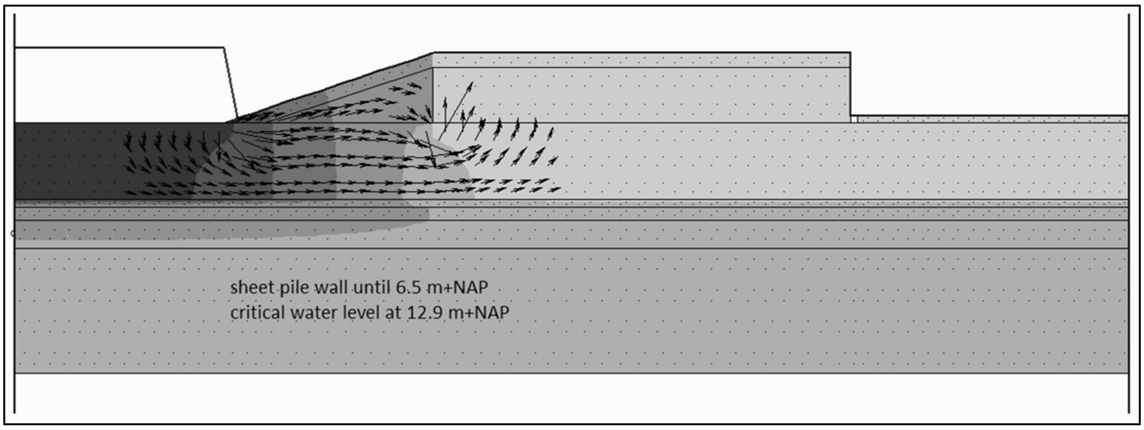


Figure C-11: Potentials and flow velocities in case of a sheet pile wall

C.3 Stability of the structure

For the stability calculations of the structure is the schematisation of the forces used as is presented in Figure C-12. For the calculation of the soil pressure under the structure are variable loads in and on top of the structure used. In order to simplify the calculations are forces like the soil pressure on the right side of the structure neglected, because this force is very small compared to the other forces and the omission of this makes the calculations more conservative.

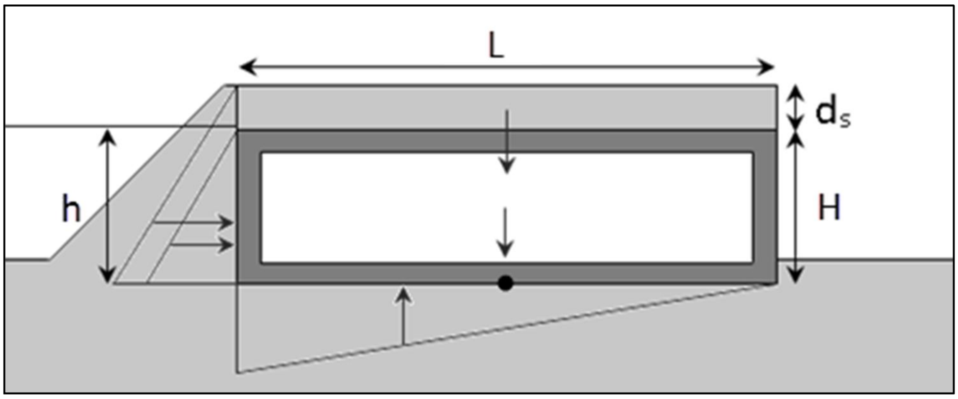


Figure C-12: Schematisation of forces for the stability of the structure

There are 4 cases considered:

- Design water level with exceedance probability of 1/1250 per year and reliability class 2
- Design water level with exceedance probability of 1/1250 per year and reliability class 3
- Design water level with exceedance probability of 1/10000 per year and reliability class 2
- Design water level with exceedance probability of 1/10000 per year and reliability class 3

These cases are used because the differences between the cases can be obtained in this way, which is important for the failure probability of the failure mechanisms. Because both the design water level as the reliability class contribute to the reliability of the failure mechanisms all possible combinations of those two are obtained. In input parameters which are used for the calculations are presented in Table C-6.

Table C-6: Input parameters for the stability calculations

Parameter	Value	Unit
design water level (1/1250)	11.60	m+NAP
design water level (1/10000)	12.00	m+NAP
height crest	13.00	m+NAP
top structure	12.00	m+NAP
bottom structure	8.00	m+NAP
length structure	30.00	m
thickness wall	0.40	m
thickness bottom and roof	0.50	m
thickness soil layer top	1.00	m
height structure	4.00	m
unit weight soil	20.00	kN/m3
unit weight water	10.00	kN/m3
unit weight concrete	25.00	kN/m3

coefficient of friction	0.50	-
active soil pressure coefficient	0.80	-
variable load inside	3.00	kN/m2
variable load on top	3.00	kN/m2

The calculations for the horizontal and overturning stability in case of a design water level with an exceedance probability of 1/1250 per year are presented in Table C-7. From these calculations is obtained that the difference between RC2 and RC3 is quite large. This is the result of the large difference between the partial factors for hydraulic loads applied in the two cases.

Table C-7: Horizontal en overturning stability with design water level of 1/1250 per year

Parameter	Force properties			RC2			RC3		
	Force [kN]	Direction	Arm [m]	y	Force [kN]	Moment [kN]	y	Force [kN]	Moment [kN]
water pressure	65	right	1.20	1.28	83	100	1.55	100	121
soil pressure	100	right	1.67	1.10	110	183	1.20	120	200
weight structure	830	down	0.00	0.90	747	0	0.90	747	0
weight soil	600	down	0.00	0.90	540	0	0.90	540	0
water pressure	405	up	5.00	1.28	518	2592	1.55	628	3139
sum horizontal	165	right			193			220	
sum vertical	1025	down			769			659	
hor friction	513	left			384			330	
sum moments						2875			3459
check H					0.50			0.67	
check M						0.75			1.05

In order to check whether the assumption of a shallow foundation is correct the soil pressure is calculated. The variable load inside and on top of the structure is in this case of importance, in contrast to the horizontal and overturning stability. This way can be obtained whether the assumed shallow foundation is a correct assumption. The maximum calculated soil pressure is smaller than the bearing capacity of the soil under the structure, which is in the order of 200 to 500 kPa.

Table C-8: Soil pressure with design water level of 1/1250 per year

Parameter	Force properties			RC2			RC3		
	Force [kN]	Direction	Arm [m]	y	Force [kN]	Moment [kN]	y	Force [kN]	Moment [kN]
water pressure	65	right	1.20	1.28	83	100	1.55	100	121
soil pressure	100	right	1.67	1.10	110	183	1.20	120	200
water pressure	405	up	5.00	1.28	518	2592	1.55	628	3139
sum moments						2875			3459
weight structure	830	down	0.00	1.10	913		1.20	996	
weight soil	600	down	0.00	1.10	660		1.20	720	
variable inside	90	down	0.00	1.50	135		1.65	149	
variable outside	90	down	0.00	1.50	135		1.65	149	
sum vertical	1610	down			1843			2013	
soil pressure (kPa)						80.60			90.16

The same calculations are done for the exceedance probability of 1/10000 per year. The difference between the cases, apart from the design water level, is the partial load factors. In order to calculate the failure probability of these failure mechanisms the four cases are obtained. The applied design water level with a corresponding exceedance probability as well as the applied partial factors both contribute to the reliability of the failure mechanisms.

Table C-9: Horizontal en overturning stability with design water level of 1/10000 per year

Parameter	Force properties			RC2			RC3		
	Force [kN]	Direction	Arm [m]	y	Force [kN]	Moment [kN]	y	Force [kN]	Moment [kN]
water pressure	80	right	1.33	1.26	101	134	1.27	102	135
soil pressure	100	right	1.67	1.10	110	183	1.20	120	200
weight structure	830	down	0.00	0.90	747	0	0.90	747	0
weight soil	600	down	0.00	0.90	540	0	0.90	540	0
water pressure	450	up	5.00	1.26	567	2835	1.27	572	2858
sum horizontal	180	right			211			222	
sum vertical	980	down			720			716	
hor friction	490	left			360			358	
sum moments						3153			3193
check H					0.59			0.62	
check M						0.88			0.89

Table C-10: Soil pressure with design water level of 1/10000 per year

Parameter	Force properties			RC2			RC3		
	Force [kN]	Direction	Arm [m]	y	Force [kN]	Moment [kN]	y	Force [kN]	Moment [kN]
water pressure	80	right	1.33	1.26	101	134	1.27	102	135
soil pressure	100	right	1.67	1.10	110	183	1.20	120	200
water pressure	450	up	5.00	1.26	567	2835	1.27	572	2858
sum moments						3153			3193
weight structure	830	down	0.00	1.10	913		1.20	996	
weight soil	600	down	0.00	1.10	660		1.20	720	
variable inside	90	down	0.00	1.50	135		1.65	149	
variable outside	90	down	0.00	1.50	135		1.65	149	
sum vertical	1610	down			1843			2013	
soil pressure (kPa)						82.45			88.39

C.4 Strength of the structural elements

The structure consists out of several elements which can individually fail due to overloading. The elements that are considered are:

- Wall
- Roof slab
- Column

In the upcoming paragraphs are the assumed dimensions of the structural elements checked.

C.4.1 Wall

The wall of the structure experiences the water pressure and the soil pressure. The wall is schematised as a beam on two supports. The soil pressure is split up in two different forces as is presented in Figure C-13.

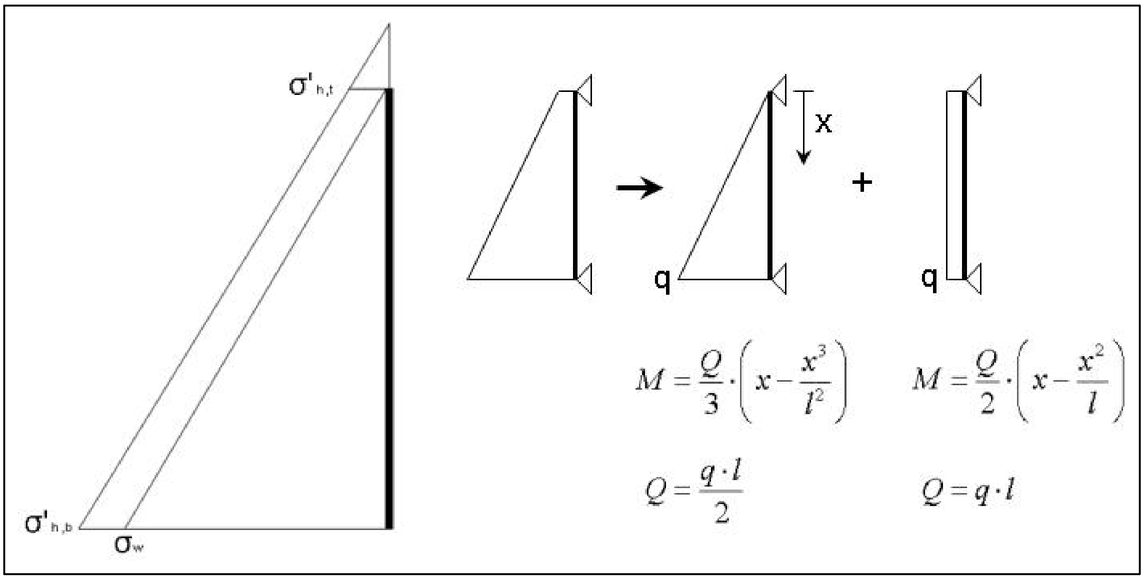


Figure C-13: Schematisation of the wall

The following data is used as input of the calculation of the acting moment:

crest height	13.0	$[m+NAP]$
top structure	12.0	$[m+NAP]$
bottom structure	8.0	$[m+NAP]$
soil density	20	$[kN/m^3]$
water density	10	$[kN/m^3]$
K_a	0.8	$[-]$

The different cases obtained for the stability of the structure are not consisted for the structural calculations. This is done because the calculations on the stability of the structure already demonstrate the difference between the different obtained cases. For the structural calculations is chosen for the combination of reliability class 3 and a design water level with an exceedance probability of 1/10000 per year. This results in partial factor for hydraulic load (according to the guideline hydraulic structures) which is smaller than 1.3 (the partial factor for permanent loads). Having a partial factor for the hydraulic loads which is smaller than the partial factor for permanent loads seems incorrect and there is the partial factor for hydraulic loads chosen equal to the partial factor of permanent loads. Table C-11 shows the partial factors used for the structural calculations.

Table C-11: Partial load factors

Type of load	Value
Permanent	1.30
Hydraulic	1.30
Variable	1.65

The bending moment is calculated with the forget-me-nots presented in Figure C-13 and the partial factors described above. The bending moment is equal to 116 kNm. The force which the reinforcement steel should be able to absorb can be calculated with the following formula:

$$M = F_s \cdot z$$

Where:

M bending moment
F_s force in the reinforcement
z internal lever arm $= 0.75 \cdot d$
d wall thickness

Rewritten to:

$$F_s = \frac{M}{0.75 \cdot d} = \frac{116}{0.75 * 0.4} = 388 \text{ kN}$$

This is the minimum required force which should be absorbed by the reinforcement steel. The amount of reinforcement bars is calculated by dividing the required amount of reinforcement area by the area of a single bar.

$$A_s = \frac{F_s}{f_s} = \frac{388 \cdot 10^3}{435} = 892 \text{ mm}^2$$

$$A_{bar} = \frac{1}{4} \cdot \pi \cdot d^2 = \frac{1}{4} \cdot \pi \cdot 12^2 = 113 \text{ mm}^2$$

$$\#_{bar} = \frac{A_s}{A_{bar}} = \frac{892}{113} = 7.9 \rightarrow 8$$

Where:

A_s required amount of reinforcement steel
f_s tensile strength steel
A_{bar} reinforcement area of a single bar
$\#_{bar}$ number of bars
d diameter of the reinforcement steel $= 12$ mm

With the number of bars known the reinforcement ratio can be calculated.

$$A_s = \#_{bar} \cdot A_{bar} = 8 \cdot 113 = 905 \text{ mm}^2$$

$$\omega = \frac{A_s}{b_c \cdot d_c} = \frac{905}{1000 \cdot 400} = 0.23\%$$

Where:

ω reinforcement ratio

b_c concrete width

d_c concrete thickness

The wall is checked in bending moment resistance and the required amount of reinforcement is calculated. The resulting reinforcement ratio is higher than the minimum reinforcement ratio but smaller than the economical reinforcement ratio. This holds that the design is slightly over dimensioned. Another phenomenon that needs to be checked is buckling of the wall. This is done by checking the concrete compressive stress resulting from the bending moment as well as the normal force. The wall is safe for buckling if the resulting force in the cross section not exceeds the maximum compressive strength of concrete.

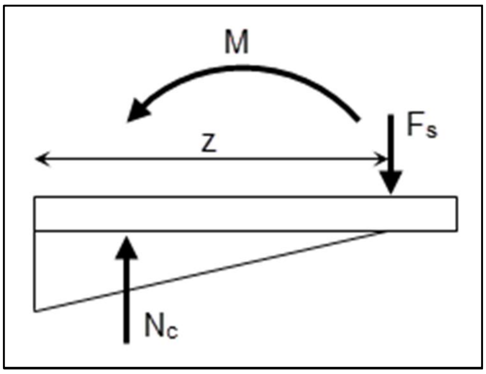


Figure C-14: Forces in the cross section

The compressive force is equal to the tensile force in the reinforcement steel to balance the force in that direction. The concrete compressive stress resulting from the bending moment is calculated assuming a linear distributed compressive stress from the outer fibre to the reinforcement at a distance z from the outer fibre, see Figure C-14.

$$N_c = \frac{1}{2} \cdot b \cdot z \cdot f_c = F_s$$

Where:

N_c concrete compressive force

b width

d thickness

f_c concrete compressive stress

F_s tensile force steel

Rewriting it in order to obtain the concrete compressive stress leads to:

$$f_c = \frac{2 \cdot F_s}{b \cdot z} = \frac{2 \cdot 388 \cdot 10^3}{1000 \cdot 0.75 \cdot 400} = 2.6 \text{ N/mm}^2$$

The compressive stress resulting from the normal force consists of the weight of the roof slab, the weight of the soil and the variable load on top of flood defence. The area for which the forces are included has a width of one running metre and a length of half the distance between the wall and the first column, resulting in the following forces:

$$F_{roof} = \gamma \cdot d_r \cdot l \cdot h \cdot \gamma_c = 1.3 \cdot 0.5 \cdot 1 \cdot 3 \cdot 25 = 49.75 \text{ kN}$$
$$F_{soil} = \gamma \cdot d_s \cdot l \cdot h \cdot \gamma_s = 1.3 \cdot 1 \cdot 1 \cdot 3 \cdot 20 = 78 \text{ kN}$$
$$F_v = \gamma \cdot l \cdot h \cdot q_v = 1.65 \cdot 1 \cdot 3 \cdot 10 = 49.5 \text{ kN}$$
$$F_{tot} = F_{roof} + F_{soil} + F_{var} = 176.25 \text{ kN}$$

The compressive stress resulting for the normal force is as follows:

$$f_c = \frac{F_{tot}}{b \cdot d} = \frac{176.25 \cdot 10^3}{1000 \cdot 400} = 0.5 \text{ N/mm}^2$$

The total concrete compressive force resulting from the normal force and the bending moment is equal to:

$$f_c = 0.5 + 2.6 = 3.1 \text{ N/mm}^2$$

The total concrete compressive force is much smaller than the concrete compressive strength so buckling will not be an issue for the wall.

C.4.2 Roof slab

The second element to be checked is the roof slab of the structure.. The forces acting on the roof in the governing situation are its own weight, the weight of the soil on top of the structure, a variable load on top of the soil and a normal force resulting from the water and soil pressure. The roof is supported with four columns and the side walls, having a centre to centre spacing of 6 m, see Figure C-15.

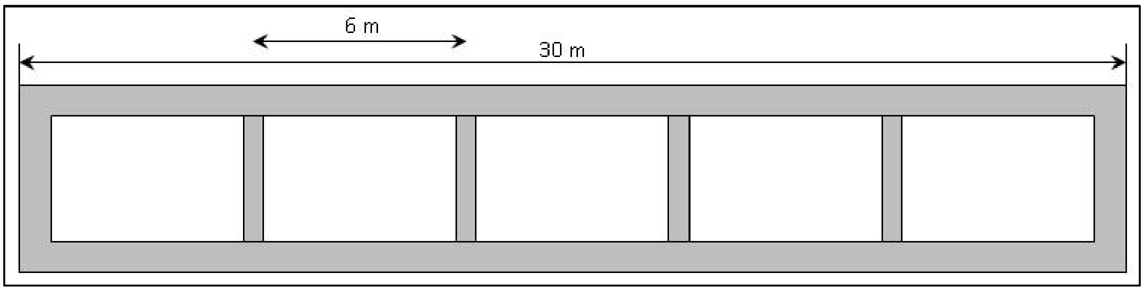


Figure C-15: Dimensions roof slab

The following assumptions are used for the calculations:

thickness roof slab	0.5	m
thickness soil layer	1.0	m
variable load	9.0	kN/m^2
length span	6.0	m
weight concrete	25	kN/m^3
weight soil	20	kN/m^3

The same partial factors are used for this element as for the previous element but are presented again in Table C-12 for the completeness.

Table C-12: Partial load factors

Type of load	Value
Permanent	1.30
Hydraulic	1.30
Variable	1.65

The roof slab is schematised as a beam on two supports with a distributed load and a normal force acting on it, see Figure C-16.

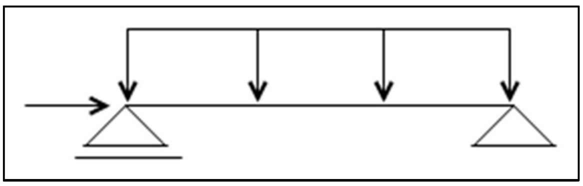


Figure C-16: Schematisation of the roof slab

The roof slab is assessed in a similar way as the wall for the bending moment resistance.

$$M = F_s \cdot z$$

Where:

M bending moment
F_s force in the reinforcement
z internal lever arm $= 0.75 \cdot d$
d slab thickness

Rewritten to:

$$F_s = \frac{M}{0.75 \cdot d} = \frac{264.4}{0.75 * 0.5} = 705 \text{ kN}$$

This is the minimum required force which should be absorbed by the reinforcement steel. The amount of reinforcement bars is calculated by dividing the required amount of reinforcement area by the area of a single bar.

$$A_s = \frac{F_s}{f_s} = \frac{705 \cdot 10^3}{435} = 1621 \text{ mm}^2$$

$$A_{bar} = \frac{1}{4} \cdot \pi \cdot d^2 = \frac{1}{4} \cdot \pi \cdot 16^2 = 201 \text{ mm}^2$$

$$\#_{bar} = \frac{A_s}{A_{bar}} = \frac{1621}{201} = 8.1 \rightarrow 9$$

Where:

A_s required amount of reinforcement steel
f_s tensile strength steel
A_{bar} reinforcement area of a single bar
$\#_{bar}$ number of bars
d diameter of the reinforcement steel $= 16$ mm

With the number of bars known the reinforcement ratio can be calculated.

$$A_s = \#_{bar} \cdot A_{bar} = 9 \cdot 201 = 1810 \text{ mm}^2$$

$$\omega = \frac{A_s}{b_c \cdot d_c} = \frac{1810}{1000 \cdot 500} = 0.36\%$$

Where:

ω reinforcement ratio
b_c concrete width
d_c concrete thickness

The roof slab is checked in bending moment resistance and the required amount of reinforcement is calculated. The resulting reinforcement ratio is higher than the minimum reinforcement ratio but smaller than the economical reinforcement ratio. This holds that the design is slightly over dimensioned. Another phenomenon that needs to be checked is buckling of the roof slab. This is done by checking the concrete compressive stress resulting from the bending moment as well as the normal force. The roof slab is safe for buckling if the resulting force in the cross section not exceeds the maximum compressive strength of concrete.

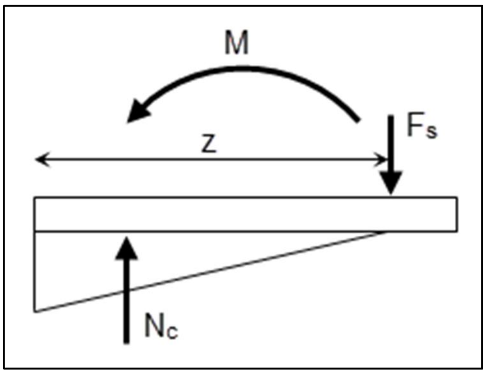


Figure C-17: Forces in the cross section

The compressive force is equal to the tensile force in the reinforcement steel to balance the force in that direction. The concrete compressive stress resulting from the bending moment is calculated assuming a linear distributed compressive stress from the outer fibre to the reinforcement at a distance z from the outer fibre, see Figure C-14.

$$N_c = \frac{1}{2} \cdot b \cdot z \cdot f_c = F_s$$

Where:

N_c concrete compressive force
b width
d thickness
f_c concrete compressive stress
F_s tensile force steel

Rewriting it in order to obtain the concrete compressive stress leads to:

$$f_c = \frac{2 \cdot F_s}{b \cdot z} = \frac{2 \cdot 705 \cdot 10^3}{1000 \cdot 0.75 \cdot 500} = 3.8 \text{ N/mm}^2$$

The normal force in the roof slab results from the water and soil pressure acting on the wall. Figure C-18 presents the distribution of the forces to the roof and bottom slab.

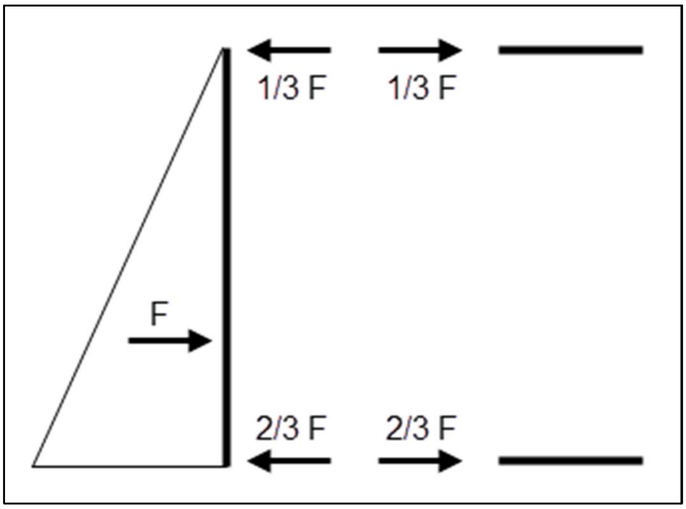


Figure C-18: Normal force in roof slab

$$
\begin{aligned}
F_{wat} &= \frac{1}{3}\cdot\gamma\cdot\frac{1}{2}\cdot\gamma_w\cdot h^2 = \frac{1}{3}\cdot 1.3\cdot\frac{1}{2}\cdot 10\cdot 4^2 && = 34.67 \text{ kN} \\
F_{soil} &= \frac{1}{3}\cdot\gamma\cdot\frac{1}{2}\cdot(\gamma_s-\gamma_w)\cdot(h_1-h_2)^2\cdot K_a + \frac{1}{2}\cdot\gamma\cdot(\gamma_s-\gamma_w)\cdot h_2\cdot h_1\cdot K_a \\
&= \frac{1}{3}\cdot 1.3\cdot\frac{1}{2}\cdot(20-10)\cdot(5-1)^2\cdot 0.8 + \frac{1}{2}\cdot 1.3\cdot(20-10)\cdot 1\cdot 4\cdot 0.8 && = 48.53 \text{ kN} \\
F_{tot} &= 83.2 \text{ kN (per running metre)}
\end{aligned}
$$

The normal force in the roof slab is calculated per running metre. The force in the roof slab must be transferred down to the bottom slab to make equilibrium with the shear stress under the structure. The force is transferred down via intermediate walls; this increases the normal force in the roof slab locally because it would concentrate around the intermediate walls. Therefore is the normal force calculated for a width of four centre to centre distances; a width of 24 m.

$$F_{tot} = 1996.8 \text{ kN (for 24 m width)}$$

The compressive stress resulting for the normal force near the intermediate wall is as follows:

$$f_c = \frac{F_{tot}}{b\cdot d} = \frac{1996.8\cdot 10^3}{1000\cdot 500} = 4.0 \text{ N/mm}^2$$

The total concrete compressive force resulting from the normal force and the bending moment is equal to:

$$f_c = 3.8 + 4.0 = 7.8 \text{ N/mm}^2$$

The total concrete compressive force is much smaller than the concrete compressive strength so buckling will not be an issue for the roof slab.

Bottom slab

The bottom slab has to carry the weight of the structure and loads on top of it. The bottom slab is supported by the subsoil, also the transmission of the horizontal forces to the subsoil acts on the bottom slab. The governing failure mechanism of the bottom slab is assumed to be buckling in the upward direction due to the axial force in the bottom slab together with the water pressure from under the structure. Buckling can be prevented if the concrete compressive force is not exceeding the concrete compressive strength. The axial force in the bottom slab is twice the force in the roof slab due to the pressure distribution on the wall. The compressive stress in the roof slab is calculated for a centre to centre distance between the intermediate walls of 24 m. The compressive stress in the bottom slab is equally distributed over the length of the structure. This results in a compressive force which is twelve times lower than the compressive strength in the roof slab.

$$f_c = 0.3 \text{ N/mm}^2$$

The other part is the compressive stress due to the water pressure under the structure.

$$M = \frac{1}{8} \cdot q \cdot l^2 = \frac{1}{8} \cdot 50 \cdot 6^2 = 225 \text{ kNm}$$
$$F = \frac{M}{0.75 \cdot d} = \frac{225}{0.75 \cdot 0.5} = 600 \text{ kN}$$
$$f_c = \frac{F}{0.5 \cdot 0.75 \cdot d \cdot b} = 3.2 \text{ N/mm}^2$$

The total compressive force in the bottom slab does not exceed the maximum compressive strength of the concrete, therefore the bottom slab safe against buckling in the upward direction.

C.4.3 Columns

The columns have to carry the weight of the roof slab, the weight of the soil and the variable weigh. The weight of the column itself is neglected because it is very small compared to the weight it has to carry. The area of the roof slab which transfers the force to the column is equal to the squared of the centre to centre distance between the columns, see Figure C-19.

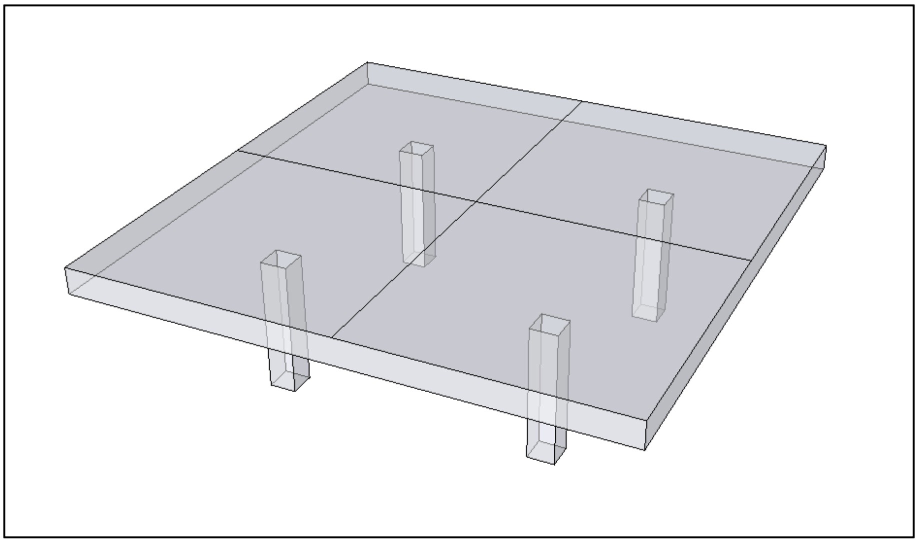


Figure C-19: Force on the column

thickness roof slab	0.5	m
thickness soil layer	1.0	m
variable load	9.0	kN/m^2
length loading area	6.0	m
width loading area	6.0	m
weight concrete	25	kN/m^3
weight soil	20	kN/m^3

The width of the column is calculated by dividing the total force on the column by the allowable concrete compressive strength. This results in the required area for the column and taking the root of that area and rounding results in the dimension of the column.

$$d^2 = \frac{F}{f_c} \quad \rightarrow \quad d = \sqrt{\frac{F}{f_c}} \quad \rightarrow \quad d = \sqrt{\frac{2056}{25}} = 287 \text{ mm} \quad \rightarrow 300 \text{ mm}$$

Where:

d	dimension of the column
F	acting force
f_c	compressive strength

Buckling of the column can also play a role in failure of the column this is checked with the by comparing the buckling force to the actual force in the column.

$$F \leq F_k = \frac{\pi^2 \cdot E \cdot I}{l_k^2}$$

Where:

F	acting force	2056	kN
E	modulus of elasticity	30	GPa
I	moment of inertia $= 1/12 \cdot b \cdot h^3$	$6.75 \cdot 10^8$	mm^4
l_k	buckling length	3.0	m
F_k	buckling force	22207	kN

The actual force is much smaller than the buckling force so the column is safe for buckling.

D FORM analyses

D.1 Horizontal stability

D.1.1 Failure probability of the design

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

$restart$

$with(stats):$

$with(Statistics):$

$with(plots):$

$interface(rtablesize = 15):$

$unprotect(\gamma)$

$n := 12:$

$J := 10:$

Creating matrices to be able to store and call values:

$\mu := Matrix(n, J):$

$\sigma := Matrix(n, J):$

$d := Matrix(n, 1):$

$\sigma z := Matrix(1, J):$

$\mu z := Matrix(1, J):$

$\beta := Matrix(1, J):$

$P := Matrix(1, J):$

$\alpha := Matrix(n, J):$

$X := Matrix(n, J + 1):$

$Xkar := Matrix(n, 1):$

$q := Matrix(1, J):$

Defining the performance function:

$R := k \cdot \Sigma V:$

$S := \Sigma H:$

$k := evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot \phi\ degrees, radians\right)\right)\right):$

$\Sigma V := F_{v,\ soil} + F_{v,\ conc} - F_{v,\ wat}:$

$\Sigma H := F_{h,\ wat} + F_{h,\ soil}:$

$F_{v,\ soil} := L \cdot d_s \cdot \gamma_{s1}:$

$F_{v,\ conc} := \left(2 \cdot d_w \cdot H + d_r \cdot L + d_b \cdot L\right) \cdot \gamma_c:$

$F_{v,\ wat} := 0.5 \cdot \gamma_w \cdot h \cdot L:$

$F_{h,\ wat} := 0.5 \cdot \gamma_w \cdot h^2:$

$F_{h,\ soil} := K_a \cdot 0.5 \cdot \left(\gamma_{s2} - \gamma_w\right) \cdot \left(H + d_s\right)^2:$

$K_a := 0.7:$

$Z := R - S$

$$\tan(0.012\ \phi)\ \left(L\ d_s\ \gamma_{s1} + \left(2\ d_w\ H + d_r\ L + d_b\ L\right)\ \gamma_c - 0.500\ \gamma_w\ h\ L\right) - 0.500\ \gamma_w\ h^2 - 0.350\ \left(\gamma_{s2} - \gamma_w\right)\ \left(H + d_s\right)^2$$

$parameters := (h, L, H, d_s, d_w, d_r, d_b, \gamma_w, \gamma_{s1}, \gamma_{s2}, \gamma_c, \phi)$:

$Z := unapply(Z, parameters)$:

Calculating the partial derivatives of the performance function:

for *i* **from** 1 **to** *n* **do** $d_{i,\,1} := unapply\left(\frac{d}{d\,parameters_i} Z(parameters), parameters\right)$: **end do**:

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1$:

$parameters_i$

$$h$$

$\mu_{i,..} := 0$:

$\sigma_{i,..} := 0$:

$i := 2$:

$parameters_i$

$$L$$

$\mu_{i,..} := 30$:

$\sigma_{i,..} := 0.03$:

$i := 3$:

$parameters_i$

$$H$$

$\mu_{i,..} := 4$:

$\sigma_{i,..} := 0.03$:

$i := 4$:

$parameters_i$

$$d_s$$

$\mu_{i,..} := 1$:

$\sigma_{i,..} := 0.05$:

$i := 5$:

$parameters_i$

$$d_w$$

$\mu_{i,..} := 0.4$:

$\sigma_{i,..} := 0.01$:

$i := 6$:

$parameters_i$

$$d_r$$

$\mu_{i,..} := 0.5$:

$\sigma_{i,..} := 0.01$:

$i := 7$:

$parameters_i$

$$d_b$$

$\mu_{i,..} := 0.5$:

$\sigma_{i,..} := 0.01$

$i := 8$:

$parameters_i$

γ_w

$\mu_{i,..} := 10 :$

$\sigma_{i,..} := 0.2 :$

$i := 9 :$

$parameters_i$

γ_{s1}

$\mu_{i,..} := 18 :$

$\sigma_{i,..} := 0.9 :$

$i := 10 :$

$parameters_i$

γ_{s2}

$\mu_{i,..} := 20 :$

$\sigma_{i,..} := 1.0 :$

$i := 11 :$

$parameters_i$

γ_c

$\mu_{i,..} := 25 :$

$\sigma_{i,..} := 0.5 :$

$i := 12 :$

$parameters_i$

ϕ

$\mu_{i,..} := 32.5 :$

$\sigma_{i,..} := 2 :$

Defining that the first design points are the mean values of the parameters:

for i **from** 1 **to** n **do** $X_{i,\,1} := \mu_{i,\,1}$: **end do**:

Generating a table with the mean values and standard deviations:

$$UNITS := \left\langle 'm','m','m','m','m','m','m','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}','deg' \right\rangle :$$

$$\left\langle \langle 'parameter' | '\mu' | '\sigma' | 'unit' \rangle, \left\langle \langle parameters \rangle \middle| \mu_{..,\,1} \middle| \sigma_{..,\,1} \middle| UNITS \right\rangle \right\rangle$$

$$\begin{bmatrix} parameter & \mu & \sigma & unit \\ h & 0 & 0 & m \\ L & 30 & 0.030 & m \\ H & 4 & 0.030 & m \\ d_s & 1 & 0.050 & m \\ d_w & 0.400 & 0.010 & m \\ d_r & 0.500 & 0.010 & m \\ d_b & 0.500 & 0.010 & m \\ \gamma_w & 10 & 0.200 & \frac{kN}{m^3} \\ \gamma_{s1} & 18 & 0.900 & \frac{kN}{m^3} \\ \gamma_{s2} & 20 & 1.000 & \frac{kN}{m^3} \\ \gamma_c & 25 & 0.500 & \frac{kN}{m^3} \\ \phi & 32.500 & 2 & deg \end{bmatrix}$$

Defining the cumulative, inverse and probability density functions of the normal distribution:

$\Phi := unapply(evalf(CDF(Normal(0,1),x)),x) :$

$\Phi I := unapply(Quantile(Normal(0,1),x),x,numeric) :$

$\varphi := unapply(evalf(PDF(Normal(0,1),x)),x) :$

Determining the parameters for the Gumbel distribution:

$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000\rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17\rangle, v)$

$$10.429 + 0.163\ \ln(v)$$

$\alpha 1 := tcoeff(l, \ln(v)) - 8 :$

$\beta 1 := coeff(l, \ln(v)) :$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$F := unapply(CDF(Gumbel(\alpha 1, \beta 1), x), x) :$

$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric) :$

$f := unapply(PDF(Gumbel(\alpha 1, \beta 1), x), x) :$

Defining the first design point of the water level:

$X_{l,\ 1} := FI(\mathrm{evalf}(1 - 10^{-4}))$

$$3.928$$

Running the FORM analysis for J iterations:

$$
\begin{aligned}
&\textbf{\textit{for } } j \textbf{\textit{ from }} 1 \textbf{\textit{ to }} J \textbf{\textit{ do}}\\
&\sigma_{1,j} := evalf\left(\frac{\varphi(\Phi I(F(X_{1,j})))}{f(X_{1,j})}\right):\\
&\mu_{1,j} := evalf(X_{1,j} - \Phi I(F(X_{1,j}))\cdot\sigma_{1,j}):\\
&\mu z_{1,j} := Z(seq(X_{i,j}, i = 1..n)) + add(d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot(\mu_{i,j} - X_{i,j}), i = 1..n):\\
&\sigma z_{1,j} := \sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}:\\
&\beta_{1,j} := \frac{\mu z_{1,j}}{\sigma z_{1,j}}:\\
&P_{1,j} := \Phi(-\beta_{1,j}):\\
&\textbf{\textit{for } } i \textbf{\textit{ from }} 1 \textbf{\textit{ to }} n \textbf{\textit{ do}}\\
&\alpha_{i,j} := -\frac{d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j}}{\sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}}:\\
&X_{i,j+1} := \mu_{i,j} + \alpha_{i,j}\cdot\beta_{1,j}\cdot\sigma_{i,j}:\\
&\textbf{\textit{end do}}:\\
&X_{1,j+1} := FI(\Phi(\alpha_{1,j}\cdot\beta_{1,j})):\\
&\textbf{\textit{end do}}:
\end{aligned}
$$

Generating a table containing the mean value, standard deviation, reliability index and failure probability of the performance function of each iteration:

$$\langle\langle 'iteration', '\mu', '\sigma', '\beta', 'P_f'\rangle | \langle convert(\langle seq(i, i = 1..J)\rangle, vector), \langle \mu z\rangle, \langle \sigma z\rangle, \langle\beta\rangle, \langle P\rangle\rangle\rangle$$

iteration	1	2	3	4	5	6	7	8	9	10
μ	382.464	478.157	508.115	509.719	509.915	509.935	509.937	509.937	509.937	509.937
σ	69.694	89.926	95.563	95.866	95.903	95.906	95.907	95.907	95.907	95.907
β	5.488	5.317	5.317	5.317	5.317	5.317	5.317	5.317	5.317	5.317
P_f	$2.035\ 10^{-8}$	$5.267\ 10^{-8}$	$5.273\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$	$5.274\ 10^{-8}$

Generating a table containing the failure probability, exceedance probability and the difference:

$$\langle\langle 'P_f', 'exc\,prob\,h', 'difference'\rangle | \langle P_{1,J}, 1 - F(X_{1,J+1}), (1 - F(X_{1,J+1})) - P_{1,J}\rangle\rangle$$

P_f	$5{,}27 \times 10^{-8}$
exc prob h	$1{,}30 \times 10^{-7}$
difference	$7{,}76 \times 10^{-8}$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$$
\begin{aligned}
&determ := \mu_{..,J}:\\
&determ_1 := 4:\\
&design := X_{..,J+1}:\\
&\gamma := \frac{design}{\sim determ}:\\
&\langle\langle 'nr' | 'sym' | '\alpha' | 'X_d' | 'X' | '\gamma'\rangle, \langle\langle seq(i, i = 1..n)\rangle | \langle parameters\rangle | \alpha_{..,J} | design | determ | \gamma\rangle\rangle
\end{aligned}
$$

nr	sym	α	X_d	X	γ
1	h	0.969	5.010	4	1.252
2	L	−0.002	30.000	30	1.000
3	H	0.009	4.001	4	1.000
4	d_s	−0.084	0.978	1	0.978
5	d_w	−0.008	0.400	0.400	0.999
6	d_r	−0.029	0.498	0.500	0.997
7	d_b	−0.029	0.498	0.500	0.997
8	γ_w	0.067	10.071	10	1.007
9	γ_{s1}	−0.103	17.507	18	0.973
10	γ_{s2}	0.090	20.481	20	1.024
11	γ_c	−0.065	24.828	25	0.993
12	ϕ	−0.160	30.798	32.500	0.948

Calculating the partial factors of the forces:

$$\gamma_{F_{h,\,wat}} := \frac{0.5 \cdot design_8 \cdot design_1^2}{0.5 \cdot determ_8 \cdot determ_1^2}$$

1.580

$$\gamma_{F_{h,\,soil}} := \frac{Ka \cdot 0.5 \cdot (design_{10} - design_8) \cdot (design_3 + design_4)^2}{Ka \cdot 0.5 \cdot (determ_{10} - determ_8) \cdot (determ_3 + determ_4)^2}$$

1.032

$$\gamma_{F_{v,\,wat}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\; degrees, radians\right)\right)\right) \cdot 0.5 \cdot design_8 \cdot design_1 \cdot design_2}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\; degrees, radians\right)\right)\right) \cdot 0.5 \cdot determ_8 \cdot determ_1 \cdot determ_2}$$

1.189

$$\gamma_{F_{v,\,soil}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\; degrees, radians\right)\right)\right) \cdot design_2 \cdot design_4 \cdot design_9}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\; degrees, radians\right)\right)\right) \cdot determ_2 \cdot determ_4 \cdot determ_9}$$

0.896

$$\gamma_{F_{v,\,conc}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\; degrees, radians\right)\right)\right) \cdot (2 \cdot design_5 \cdot design_3 + design_6 \cdot design_2 + design_7 \cdot design_2) \cdot design_{11}}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\; degrees, radians\right)\right)\right) \cdot (2 \cdot determ_5 \cdot determ_3 + determ_6 \cdot determ_2 + determ_7 \cdot determ_2) \cdot determ_{11}}$$

0.934

D.1.2 Calibration of the partial factors

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

$restart$

$with(stats):$

$with(Statistics):$

$with(plots):$

$interface(rtablesize = 15):$

$unprotect(\gamma)$

$n := 12:$

$J := 10:$

Creating matrices to be able to store and call values:

$\mu := Matrix(n, J):$

$\sigma := Matrix(n, J):$

$d := Matrix(n, 1):$

$\sigma z := Matrix(1, J):$

$\mu z := Matrix(1, J):$

$\beta := Matrix(1, J):$

$P := Matrix(1, J):$

$\alpha := Matrix(n, J):$

$X := Matrix(n, J + 1):$

$Xkar := Matrix(n, 1):$

$q := Matrix(1, J):$

Defining the performance function:

$R := k \cdot \Sigma V:$

$S := \Sigma H:$

$k := evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot \phi\ degrees, radians\right)\right)\right):$

$\Sigma V := F_{v,\,soil} + F_{v,\,conc} - F_{v,\,wat}:$

$\Sigma H := F_{h,\,wat} + F_{h,\,soil}:$

$F_{v,\,soil} := L \cdot d_s \cdot \gamma_{s1}:$

$F_{v,\,conc} := \left(2 \cdot d_w \cdot H + d_r \cdot L + d_b \cdot L\right) \cdot \gamma_c:$

$F_{v,\,wat} := 0.5 \cdot \gamma_w \cdot h \cdot L:$

$F_{h,\,wat} := 0.5 \cdot \gamma_w \cdot h^2:$

$F_{h,\,soil} := K_a \cdot 0.5 \cdot \left(\gamma_{s2} - \gamma_w\right) \cdot \left(H + d_s\right)^2:$

$K_a := 0.7:$

$Z := R - S$

$$\tan(0.012\,\phi)\,\left(L\, d_s\, \gamma_{s1} + \left(2\, d_w\, H + d_r\, L + d_b\, L\right)\gamma_c - 0.500\,\gamma_w\, h\, L\right) - 0.500\,\gamma_w\, h^2 - 0.350\,\left(\gamma_{s2} - \gamma_w\right)\left(H + d_s\right)^2$$

$parameters := \left(h, L, H, d_s, d_w, d_r, d_b, \gamma_w, \gamma_{s1}, \gamma_{s2}, \gamma_c, \phi\right):$

$Z := unapply(Z, parameters):$

Calculating the partial derivatives of the performance function:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } d_{i,\,1} := unapply\left(\frac{d}{d\,parameters_i} Z(parameters), parameters\right) : \textbf{ end do}:$

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1 :$

$parameters_i$

$$h$$

$\mu_{i,..} := 0 :$

$\sigma_{i,..} := 0 :$

$i := 2 :$

$parameters_i$

$$L$$

$\mu_{i,..} := 30 :$

$\sigma_{i,..} := 0.03 :$

$i := 3 :$

$parameters_i$

$$H$$

$\mu_{i,..} := 4 :$

$\sigma_{i,..} := 0.03 :$

$i := 4 :$

$parameters_i$

$$d_s$$

$\mu_{i,..} := 1 :$

$\sigma_{i,..} := 0.05 :$

$i := 5 :$

$parameters_i$

$$d_w$$

$\mu_{i,..} := 0.4 :$

$\sigma_{i,..} := 0.01 :$

$i := 6 :$

$parameters_i$

$$d_r$$

$\mu_{i,..} := 0.5 :$

$\sigma_{i,..} := 0.01 :$

$i := 7 :$

$parameters_i$

$$d_b$$

$\mu_{i,..} := 0.5 :$

$\sigma_{i,..} := 0.01$

$i := 8 :$

$parameters_i$

$$\gamma_w$$

$\mu_{i,..} := 10 :$

$\sigma_{i,..} := 0.2 :$

$i := 9 :$

$parameters_i$

γ_{s1}

$\mu_{i,..} := 18 :$

$\sigma_{i,..} := 0.9 :$

$i := 10 :$

$parameters_i$

γ_{s2}

$\mu_{i,..} := 20 :$

$\sigma_{i,..} := 1.0 :$

$i := 11 :$

$parameters_i$

γ_c

$\mu_{i,..} := 25 :$

$\sigma_{i,..} := 0.5 :$

$i := 12 :$

$parameters_i$

ϕ

$\mu_{i,..} := 32.5 :$

$\sigma_{i,..} := 2 :$

Defining that the first design points are the mean values of the parameters:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } X_{i,\,1} := \mu_{i,\,1} : \textbf{end do}:$

Generating a table with the mean values and standard deviations:

$UNITS := \left\langle 'm','m','m','m','m','m','m','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}','deg' \right\rangle :$

$\left\langle \langle 'parameter' | '\mu' | '\sigma' | 'unit' \rangle, \langle \langle parameters \rangle | \mu_{..,\,1} | \sigma_{..,\,1} | UNITS \rangle \right\rangle$

parameter	μ	σ	unit
h	0	0	m
L	30	0.030	m
H	4	0.030	m
d_s	1	0.050	m
d_w	0.400	0.010	m
d_r	0.500	0.010	m
d_b	0.500	0.010	m
γ_w	10	0.200	$\frac{kN}{m^3}$
γ_{s1}	18	0.900	$\frac{kN}{m^3}$
γ_{s2}	20	1.000	$\frac{kN}{m^3}$
γ_c	25	0.500	$\frac{kN}{m^3}$
ϕ	32.500	2	deg

Defining the cumulative, inverse and probability density functions of the normal distribution:

$\Phi := unapply(evalf(CDF(Normal(0, 1), x)), x) :$

$\Phi I := unapply(Quantile(Normal(0,1),x),x,numeric):$

$\varphi := unapply(evalf(PDF(Normal(0,1),x)),x):$

Determining the parameters for the Gumbel distribution:

$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000\rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17\rangle, v)$

$$10.429 + 0.163\ln(v)$$

$\alpha 1 := tcoeff(l, \ln(v)) - 8:$

$\beta 1 := coeff(l, \ln(v)):$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$F := unapply(CDF(Gumbel(\alpha 1, \beta 1), x), x):$

$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric):$

$f := unapply(PDF(Gumbel(\alpha 1, \beta 1), x), x):$

Defining the first design point of the water level:

$X_{1,1} := FI(\mathrm{evalf}(1-10^{-4}))$

$$3.928$$

Running the FORM analysis for J iterations:

$\boldsymbol{for}\ j\ \boldsymbol{from}\ 1\ \boldsymbol{to}\ J\ \boldsymbol{do}$

$\sigma_{1,j} := evalf\left(\frac{\varphi(\Phi I(F(X_{1,j})))}{f(X_{1,j})}\right):$

$\mu_{1,j} := evalf(X_{1,j} - \Phi I(F(X_{1,j}))\cdot\sigma_{1,j}):$

$\beta_{1,j} := evalf(-\Phi I(10^{-6})):$

$\boldsymbol{for}\ i\ \boldsymbol{from}\ 1\ \boldsymbol{to}\ n\ \boldsymbol{do}$

$\alpha_{i,j} := -\frac{d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j}}{\sqrt{add\left((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n\right)}}:$

$X_{i,j+1} := \mu_{i,j} + \alpha_{i,j}\cdot\beta_{1,j}\cdot\sigma_{i,j}:$

$\boldsymbol{end\ do}:$

$X_{1,j+1} := FI(\Phi(\alpha_{1,j}\cdot\beta_{1,j})):$

$\boldsymbol{end\ do}:$

Generating a table containing the failure probability, exceedance probability and the difference:

$\langle\langle 'P_f', 'exc\,prob\,h', 'difference'\rangle | \langle P_{1,J}, 1 - F(X_{1,J+1}), (1 - F(X_{1,J+1})) - P_{1,J}\rangle\rangle$

$$\begin{bmatrix} P_f & 0{,}00\times 10^{0} \\ exc\,prob\,h & 2{,}95\times 10^{-6} \\ difference & 2{,}95\times 10^{-6} \end{bmatrix}$$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$determ := \mu_{..,J}:$

$determ_1 := 4:$

$design := X_{..,J+1}:$

$$\gamma := \frac{design}{\sim determ}:$$

$$\left\langle \left\langle 'nr' \middle| 'sym' \middle| '\alpha' \middle| 'X_d' \middle| 'X' \middle| '\gamma' \right\rangle, \left\langle \langle seq(i, i = 1..n) \rangle \middle| \langle parameters \rangle \middle| \alpha_{..., 1} \middle| design \middle| determ \middle| \gamma \right\rangle \right\rangle$$

nr	sym	α	X_d	X	γ
1	h	0.953	4.502	4	1.125
2	L	−0.003	30.000	30	1.000
3	H	0.011	4.002	4	1.000
4	d_s	−0.097	0.977	1	0.977
5	d_w	−0.009	0.400	0.400	0.999
6	d_r	−0.034	0.498	0.500	0.997
7	d_b	−0.034	0.498	0.500	0.997
8	γ_w	0.065	10.062	10	1.006
9	γ_{s1}	−0.119	17.489	18	0.972
10	γ_{s2}	0.106	20.504	20	1.025
11	γ_c	−0.075	24.822	25	0.993
12	ϕ	−0.212	30.486	32.500	0.938

Calculating the partial factors of the forces:

$$\gamma_{F_{h,\ wat}} := \frac{0.5 \cdot design_8 \cdot design_1^2}{0.5 \cdot determ_8 \cdot determ_1^2}$$

$$1.274$$

$$\gamma_{F_{h,\ soil}} := \frac{Ka \cdot 0.5 \cdot (design_9 - design_8) \cdot (design_3 + design_4)^2}{Ka \cdot 0.5 \cdot (determ_9 - determ_8) \cdot (determ_3 + determ_4)^2}$$

$$0.921$$

$$\gamma_{F_{v,\ wat}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\ degrees,\ radians\right)\right)\right) \cdot 0.5 \cdot design_8 \cdot design_1 \cdot design_2}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\ degrees,\ radians\right)\right)\right) \cdot 0.5 \cdot determ_8 \cdot determ_1 \cdot determ_2}$$

$$1.056$$

$$\gamma_{F_{v,\ soil}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\ degrees,\ radians\right)\right)\right) \cdot design_2 \cdot design_4 \cdot design_{10}}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\ degrees,\ radians\right)\right)\right) \cdot determ_2 \cdot determ_4 \cdot determ_{10}}$$

$$0.934$$

$$\gamma_{F_{v,\ conc}} := \frac{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot design_{12}\ degrees,\ radians\right)\right)\right) \cdot (2 \cdot design_5 \cdot design_3 + design_6 \cdot design_2 + design_7 \cdot design_2) \cdot design_{11}}{evalf\left(\tan\left(convert\left(\frac{2}{3} \cdot determ_{12}\ degrees,\ radians\right)\right)\right) \cdot (2 \cdot determ_5 \cdot determ_3 + determ_6 \cdot determ_2 + determ_7 \cdot determ_2) \cdot determ_{11}}$$

$$0.923$$

D.2 Overturning stability

D.2.1 Failure probability of the design

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

$restart$

$with(stats)$:

$with(Statistics)$:

$with(plots)$:

$interface(rtablesize = 15)$:

$unprotect(\gamma)$

$n := 11$:

$J := 10$:

Creating matrices to be able to store and call values:

$\mu := Matrix(n, J)$:

$\sigma := Matrix(n, J)$:

$d := Matrix(n, 1)$:

$\sigma z := Matrix(1, J)$:

$\mu z := Matrix(1, J)$:

$\beta := Matrix(1, J)$:

$P := Matrix(1, J)$:

$\alpha := Matrix(n, J)$:

$X := Matrix(n, J+1)$:

$Xkar := Matrix(n, 1)$:

$q := Matrix(1, J)$:

Defining the performance function:

$R := \frac{1}{6} \cdot L$:

$S := \frac{\Sigma M}{\Sigma V}$:

$\Sigma M := F_{h,\,wat} \cdot \frac{1}{3} \cdot h + F_{h,\,soil} \cdot \frac{1}{3} \cdot (H + d_s) + F_{v,\,wat} \cdot \frac{1}{6} \cdot L$:

$\Sigma V := F_{v,\,conc} + F_{v,\,soil} - F_{v,\,wat}$:

$F_{v,\,soil} := L \cdot d_s \cdot \gamma_{s1}$:

$F_{v,\,conc} := (2 \cdot d_w \cdot H + d_r \cdot L + d_b \cdot L) \cdot \gamma_c$:

$F_{v,\,wat} := 0.5 \cdot \gamma_w \cdot (h - 0.5) \cdot L$:

$F_{h,\,wat} := 0.5 \cdot \gamma_w \cdot h^2$:

$F_{h,\,soil} := K_a \cdot 0.5 \cdot (\gamma_{s2} - \gamma_w) \cdot (H + d_s)^2$:

$K_a := 0.7$:

$Z := R - S$

$$\frac{1}{6} L - \frac{0.167\, \gamma_w\, h^3 + 0.117\, (\gamma_{s2} - \gamma_w)\, (H + d_s)^3 + 0.083\, \gamma_w\, (h - 0.500)\, L^2}{(2\, d_w\, H + d_r\, L + d_b\, L)\, \gamma_c + L\, d_s\, \gamma_{s1} - 0.500\, \gamma_w\, (h - 0.500)\, L}$$

$parameters := (h, L, H, d_s, d_w, d_r, d_b, \gamma_w, \gamma_{s1}, \gamma_{s2}, \gamma_c)$:

$Z := unapply(Z, parameters)$:

Calculating the partial derivatives of the performance function:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } d_{i,\,1} := unapply\left(\frac{d}{d\, parameters_i} Z(parameters),\, parameters\right) \textbf{end do}$:

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1$:
$parameters_i$

$$h$$

$\mu_{i,..} := 0$:
$\sigma_{i,..} := 0$:

$i := 2$:
$parameters_i$

$$L$$

$\mu_{i,..} := 30$:
$\sigma_{i,..} := 0.03$:

$i := 3$:
$parameters_i$

$$H$$

$\mu_{i,..} := 4$:
$\sigma_{i,..} := 0.03$:

$i := 4$:
$parameters_i$

$$d_s$$

$\mu_{i,..} := 1$:
$\sigma_{i,..} := 0.05$:

$i := 5$:
$parameters_i$

$$d_w$$

$\mu_{i,..} := 0.4$:
$\sigma_{i,..} := 0.01$:

$i := 6$:
$parameters_i$

$$d_r$$

$\mu_{i,..} := 0.5$:
$\sigma_{i,..} := 0.01$:

$i := 7$:
$parameters_i$

$$d_b$$

$\mu_{i,..} := 0.5$:
$\sigma_{i,..} := 0.01$

$i := 8$:
$parameters_i$

$$\gamma_w$$

$\mu_{i,..} := 10 :$

$\sigma_{i,..} := 0.2 :$

$i := 9 :$

$parameters_i$

$$\gamma_{s1}$$

$\mu_{i,..} := 18 :$

$\sigma_{i,..} := 0.9 :$

$i := 10 :$

$parameters_i$

$$\gamma_{s2}$$

$\mu_{i,..} := 20 :$

$\sigma_{i,..} := 1.0 :$

$i := 11 :$

$parameters_i$

$$\gamma_c$$

$\mu_{i,..} := 25 :$

$\sigma_{i,..} := 0.5 :$

Defining that the first design points are the mean values of the parameters:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } X_{i,\,1} := \mu_{i,\,1} : \textbf{end do}:$

Generating a table with the mean values and standard deviations:

$UNITS := \left\langle 'm','m','m','m','m','m','m','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^3}' \right\rangle :$

$\langle \langle 'parameter' | '\mu' | '\sigma' | 'unit' \rangle, \langle \langle parameters \rangle | \mu_{..,\,1} | \sigma_{..,\,1} | UNITS \rangle \rangle$

parameter	μ	σ	unit
h	0	0	m
L	30	0.030	m
H	4	0.030	m
d_s	1	0.050	m
d_w	0.400	0.010	m
d_r	0.500	0.010	m
d_b	0.500	0.010	m
γ_w	10	0.200	$\frac{kN}{m^3}$
γ_{s1}	18	0.900	$\frac{kN}{m^3}$
γ_{s2}	20	1.000	$\frac{kN}{m^3}$
γ_c	25	0.500	$\frac{kN}{m^3}$

Defining the cumulative, inverse and probability density functions of the normal distribution:

$\Phi := unapply(statevalf[cdf, normald[0, 1]](x), x) :$

$\Phi I := unapply(statevalf[icdf, normald[0, 1]](x), x) :$

$\varphi := unapply(statevalf[pdf, normald[0, 1]](x), x) :$

Determining the parameters for the Gumbel distribution:

$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000\rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17\rangle, v)$

$$10.429 + 0.163\ \ln(v)$$

$\alpha 1 := tcoeff(l, \ln(v)) - 8 :$

$\beta 1 := coeff(l, \ln(v)) :$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$F := unapply(CDF(Gumbel(\alpha 1, \beta 1), x), x) :$

$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric) :$

$f := unapply(PDF(Gumbel(\alpha 1, \beta 1), x), x) :$

Defining the first design point of the water level:

$X_{1,\,1} := FI(evalf(1 - 10^{-4}))$

$$3.928$$

Running the FORM analysis for J iterations:

$$
\begin{aligned}
&\textbf{for } j \textbf{ from } 1 \textbf{ to } J \textbf{ do}\\
&\sigma_{1,j} := evalf\left(\frac{\varphi(\Phi I(F(X_{1,j})))}{f(X_{1,j})}\right):\\
&\mu_{1,j} := evalf(X_{1,j} - \Phi I(F(X_{1,j}))\cdot\sigma_{1,j}):\\
&\mu z_{1,j} := Z(seq(X_{i,j}, i = 1..n)) + add(d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot(\mu_{i,j} - X_{i,j}), i = 1..n):\\
&\sigma z_{1,j} := \sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}:\\
&\beta_{1,j} := \frac{\mu z_{1,j}}{\sigma z_{1,j}}:\\
&P_{1,j} := \Phi(-\beta_{1,j}):\\
&\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do}\\
&\alpha_{i,j} := -\frac{d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j}}{\sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}}:\\
&X_{i,j+1} := \mu_{i,j} + \alpha_{i,j}\cdot\beta_{1,j}\cdot\sigma_{i,j}:\\
&\textbf{end do}:\\
&X_{1,j+1} := FI(\Phi(\alpha_{1,j}\cdot\beta_{1,j})):\\
&\textbf{end do}:
\end{aligned}
$$

Generating a table containing the mean value, standard deviation, reliability index and failure probability of the performance function of each iteration:

$\langle\langle 'iteration','\mu','\sigma','\beta','P_f'\rangle \mid \langle convert(\langle seq(i, i = 1..J)\rangle, vector), \langle \mu z\rangle, \langle \sigma z\rangle, \langle \beta\rangle, \langle P\rangle\rangle\rangle$

iteration	1	2	3	4	5	6	7	8	9	10
μ	5.407	12.502	9.660	9.331	9.325	9.325	9.325	9.325	9.325	9.325
σ	1.013	2.535	1.983	1.915	1.914	1.914	1.914	1.914	1.914	1.914
β	5.337	4.932	4.873	4.872	4.872	4.872	4.872	4.872	4.872	4.872
P_f	$4.723\ 10^{-8}$	$4.064\ 10^{-7}$	$5.500\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$	$5.529\ 10^{-7}$

Generating a table containing the failure probability, exceedance probability and the difference:

$\langle\langle 'P_f','exc\ prob\ h','difference'\rangle \mid \langle P_{1,J}, 1 - F(X_{1,J+1}), (1 - F(X_{1,J+1})) - P_{1,J}\rangle\rangle$

$$\begin{bmatrix} P_f & 5{,}53\times 10^{-7} \\ exc\,prob\,h & 8{,}32\times 10^{-7} \\ difference & 2{,}79\times 10^{-7} \end{bmatrix}$$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$determ := \mu_{..,J}:$

$determ_1 := 4:$

$design := X_{..,J+1}:$

$\gamma := \dfrac{design}{\sim determ}:$

$\langle\langle 'nr'|'sym'|'\alpha'|'X_d'|'X'|'\gamma'\rangle, \langle\langle seq(i, i=1..n)\rangle|\langle parameters\rangle|\alpha_{..,J}|design|determ|\gamma\rangle\rangle$

nr	sym	α	X_d	X	γ
1	h	0.983	4.708	4	1.177
2	L	−0.000	30.000	30	1.000
3	H	−0.000	4.000	4	1.000
4	d_s	−0.095	0.977	1	0.977
5	d_w	−0.007	0.400	0.400	0.999
6	d_r	−0.028	0.499	0.500	0.997
7	d_b	−0.028	0.499	0.500	0.997
8	γ_w	0.095	10.092	10	1.009
9	γ_{s1}	−0.098	17.569	18	0.976
10	γ_{s2}	0.011	20.052	20	1.003
11	γ_c	−0.062	24.850	25	0.994

Calculating the partial factors of the forces:

$$\gamma_{M_{h,\,wat}} := \frac{\frac{1}{3}\cdot design_1}{\frac{1}{3}\cdot determ_1}\cdot\frac{0.5\cdot design_8\cdot design_1^2}{0.5\cdot determ_8\cdot determ_1^2}$$

1.646

$$\gamma_{M_{h,\,soil}} := \frac{\frac{1}{3}\cdot\left(design_3+design_4\right)}{\frac{1}{3}\cdot\left(determ_3+determ_4\right)}\cdot\frac{K_a\cdot 0.5\cdot\left(design_{10}-design_8\right)\cdot\left(design_3+design_4\right)^2}{K_a\cdot 0.5\cdot\left(determ_{10}-determ_8\right)\cdot\left(determ_3+determ_4\right)^2}$$

0.982

$$\gamma_{M_{v,\,wat}} := \frac{\frac{1}{6}\cdot design_2}{\frac{1}{6}\cdot determ_2}\cdot\frac{0.5\cdot design_8\cdot design_1\cdot design_2}{0.5\cdot determ_8\cdot determ_1\cdot determ_2}$$

1.188

$$\gamma_{F_{v,\,wat}} := \frac{0.5\cdot design_8\cdot design_1\cdot design_2}{0.5\cdot determ_8\cdot determ_1\cdot determ_2}$$

1.188

$$\gamma_{F_{v,\,conc}} := \frac{\left(2\cdot design_5\cdot design_3+design_6\cdot design_2+design_7\cdot design_2\right)\cdot design_{11}}{\left(2\cdot determ_5\cdot determ_3+determ_6\cdot determ_2+determ_7\cdot determ_2\right)\cdot determ_{11}}$$

0.991

$$\gamma_{F_{v,\,soil}} := \frac{design_2\cdot design_4\cdot design_9}{determ_2\cdot determ_4\cdot determ_9}$$

0.953

D.2.2 Calibration of the partial factors

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

$restart$

$with(stats):$

$with(Statistics):$

$with(plots):$

$interface(rtablesize = 15):$

$unprotect(\gamma)$

$n := 11:$

$J := 10:$

Creating matrices to be able to store and call values:

$\mu := Matrix(n, J):$

$\sigma := Matrix(n, J):$

$d := Matrix(n, 1):$

$\sigma z := Matrix(1, J):$

$\mu z := Matrix(1, J):$

$\beta := Matrix(1, J):$

$P := Matrix(1, J):$

$\alpha := Matrix(n, J):$

$X := Matrix(n, J + 1):$

$Xkar := Matrix(n, 1):$

$q := Matrix(1, J):$

Defining the performance function:

$R := \frac{1}{6} \cdot L:$

$S := \frac{\Sigma M}{\Sigma V}:$

$\Sigma M := F_{h,\,wat} \cdot \frac{1}{3} \cdot h + F_{h,\,soil} \cdot \frac{1}{3} \cdot (H + d_s) + F_{v,\,wat} \cdot \frac{1}{6} \cdot L:$

$\Sigma V := F_{v,\,conc} + F_{v,\,soil} - F_{v,\,wat}:$

$F_{v,\,soil} := L \cdot d_s \cdot \gamma_{s1}:$

$F_{v,\,conc} := (2 \cdot d_w \cdot H + d_r \cdot L + d_b \cdot L) \cdot \gamma_c:$

$F_{v,\,wat} := 0.5 \cdot \gamma_w \cdot (h - 0.5) \cdot L:$

$F_{h,\,wat} := 0.5 \cdot \gamma_w \cdot h^2:$

$F_{h,\,soil} := K_a \cdot 0.5 \cdot (\gamma_{s2} - \gamma_w) \cdot (H + d_s)^2:$

$K_a := 0.7:$

$Z := R - S$

$$\frac{1}{6}\, L - \frac{0.167\, \gamma_w\, h^3 + 0.117\, (\gamma_{s2} - \gamma_w)\, (H + d_s)^3 + 0.083\, \gamma_w\, (h - 0.500)\, L^2}{(2\, d_w\, H + d_r\, L + d_b\, L)\, \gamma_c + L\, d_s\, \gamma_{s1} - 0.500\, \gamma_w\, (h - 0.500)\, L}$$

$parameters := (h, L, H, d_s, d_w, d_r, d_b, \gamma_w, \gamma_{s1}, \gamma_{s2}, \gamma_c):$

$Z := unapply(Z, parameters):$

Calculating the partial derivatives of the performance function:

$$\textbf{\textit{for}}\ i\ \textbf{\textit{from}}\ 1\ \textbf{\textit{to}}\ n\ \textbf{\textit{do}}\ d_{i,\,1} := unapply\left(\frac{d}{d\,parameters_i}Z(parameters),\ parameters\right)\textbf{\textit{end}}\ \textbf{do}:$$

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1 :$

$parameters_i$

$$h$$

$\mu_{i,..} := 0 :$

$\sigma_{i,..} := 0 :$

$i := 2 :$

$parameters_i$

$$L$$

$\mu_{i,..} := 30 :$

$\sigma_{i,..} := 0.03 :$

$i := 3 :$

$parameters_i$

$$H$$

$\mu_{i,..} := 4 :$

$\sigma_{i,..} := 0.03 :$

$i := 4 :$

$parameters_i$

$$d_s$$

$\mu_{i,..} := 1 :$

$\sigma_{i,..} := 0.05 :$

$i := 5 :$

$parameters_i$

$$d_w$$

$\mu_{i,..} := 0.4 :$

$\sigma_{i,..} := 0.01 :$

$i := 6 :$

$parameters_i$

$$d_r$$

$\mu_{i,..} := 0.5 :$

$\sigma_{i,..} := 0.01 :$

$i := 7 :$

$parameters_i$

$$d_b$$

$\mu_{i,..} := 0.5 :$

$\sigma_{i,..} := 0.01$

$i := 8 :$

$parameters_i$

$$\gamma_w$$

$\mu_{i,..} := 10 :$

$\sigma_{i,..} := 0.2 :$

$i := 9 :$
$parameters_i$

γ_{s1}

$\mu_{i,..} := 18 :$
$\sigma_{i,..} := 0.9 :$
$i := 10 :$
$parameters_i$

γ_{s2}

$\mu_{i,..} := 20 :$
$\sigma_{i,..} := 1.0 :$
$i := 11 :$
$parameters_i$

γ_c

$\mu_{i,..} := 25 :$
$\sigma_{i,..} := 0.5 :$

Defining that the first design points are the mean values of the parameters:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } X_{i,\,1} := \mu_{i,\,1} : \textbf{end do}:$

Generating a table with the mean values and standard deviations:

$UNITS := \left\langle 'm','m','m','m','m','m','m',' \frac{kN}{m^3} ',' \frac{kN}{m^3} ',' \frac{kN}{m^3} ',' \frac{kN}{m^3} ' \right\rangle :$
$\langle \langle 'parameter' | '\mu' | '\sigma' | 'unit' \rangle, \langle \langle parameters \rangle | \mu_{..,\,1} | \sigma_{..,\,1} | UNITS \rangle \rangle$

parameter	μ	σ	unit
h	0	0	m
L	30	0.030	m
H	4	0.030	m
d_s	1	0.050	m
d_w	0.400	0.010	m
d_r	0.500	0.010	m
d_b	0.500	0.010	m
γ_w	10	0.200	$\frac{kN}{m^3}$
γ_{s1}	18	0.900	$\frac{kN}{m^3}$
γ_{s2}	20	1.000	$\frac{kN}{m^3}$
γ_c	25	0.500	$\frac{kN}{m^3}$

Defining the cumulative, inverse and probability density functions of the normal distribution:

$\Phi := unapply(statevalf[cdf, normald[0, 1]](x), x) :$

$\Phi I := unapply(statevalf[icdf, normald[0, 1]](x), x) :$

$\varphi := unapply(statevalf[pdf, normald[0, 1]](x), x) :$

Determining the parameters for the Gumbel distribution:

$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000\rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17\rangle, v)$

$$10.429 + 0.163 \ln(v)$$

$\alpha 1 := tcoeff(l, \ln(v)) - 8 :$

$\beta 1 := coeff(l, \ln(v)) :$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$F := unapply(CDF(Gumbel(\alpha 1, \beta 1), x), x) :$

$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric) :$

$f := unapply(PDF(Gumbel(\alpha 1, \beta 1), x), x) :$

Defining the first design point of the water level:

$X_{1,\,1} := FI(evalf(1 - 10^{-4}))$

$$3.928$$

Running the FORM analysis for J iterations:

for *j* **from** *1* **to** *J* **do**

$\sigma_{1,\,j} := evalf\left(\dfrac{\varphi(\Phi I(F(X_{1,\,j})))}{f(X_{1,\,j})}\right) :$

$\mu_{1,\,j} := evalf(X_{1,\,j} - \Phi I(F(X_{1,\,j}))\cdot\sigma_{1,\,j}) :$

$\beta_{1,\,j} := evalf(-\Phi I(10^{-6})) :$

for *i* **from** *1* **to** *n* **do**

$\alpha_{i,\,j} := -\dfrac{d_{i,\,1}(seq(X_{i,\,j}, i = 1..n))\cdot\sigma_{i,\,j}}{\sqrt{add((d_{i,\,1}(seq(X_{i,\,j}, i = 1..n))\cdot\sigma_{i,\,j})^2, i = 1..n)}} :$

$X_{i,\,j+1} := \mu_{i,\,j} + \alpha_{i,\,j}\cdot\beta_{1,\,j}\cdot\sigma_{i,\,j} :$

end do:

$X_{1,\,j+1} := FI(\Phi(\alpha_{1,\,j}\cdot\beta_{1,\,j})) :$

end do:

Generating a table containing the failure probability, exceedance probability and the difference:

$\langle\langle 'P_f','exc\,prob\,h','difference'\rangle | \langle P_{1,\,J}, 1 - F(X_{1,\,J+1}), (1 - F(X_{1,\,J+1})) - P_{1,\,J}\rangle\rangle$

$$\begin{bmatrix} P_f & 0{,}00 \times 10^{0} \\ exc\,prob\,h & 1{,}48 \times 10^{-6} \\ difference & 1{,}48 \times 10^{-6} \end{bmatrix}$$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$determ := \mu_{..,\,J} :$

$determ_1 := 4 :$

$design := X_{..,\,J+1} :$

$\gamma := \dfrac{design}{\sim determ} :$

$\langle\langle 'nr' | 'sym' | '\alpha' | 'X_d' | 'X' | '\gamma'\rangle, \langle\langle seq(i, i = 1..n)\rangle | \langle parameters\rangle | \alpha_{..,\,J} | design | determ | \gamma\rangle\rangle$

nr	sym	α	X_d	X	γ
1	h	0.983	4.614	4	1.154
2	L	−0.000	30.000	30	1.000
3	H	−0.000	4.000	4	1.000
4	d_s	−0.095	0.977	1	0.977
5	d_w	−0.007	0.400	0.400	0.999
6	d_r	−0.028	0.499	0.500	0.997
7	d_b	−0.028	0.499	0.500	0.997
8	γ_w	0.095	10.090	10	1.009
9	γ_{s1}	−0.099	17.579	18	0.977
10	γ_{s2}	0.011	20.053	20	1.003
11	γ_c	−0.062	24.853	25	0.994

Calculating the partial factors of the forces:

$$\gamma_{M_{h,\,wat}} := \frac{\frac{1}{3}\cdot design_1}{\frac{1}{3}\cdot determ_1}\cdot\frac{0.5\cdot design_8\cdot design_1^2}{0.5\cdot determ_8\cdot determ_1^2}$$

1.549

$$\gamma_{M_{h,\,soil}} := \frac{\frac{1}{3}\cdot\left(design_3+design_4\right)}{\frac{1}{3}\cdot\left(determ_3+determ_4\right)}\cdot\frac{K_a\cdot 0.5\cdot\left(design_{10}-design_8\right)\cdot\left(design_3+design_4\right)^2}{K_a\cdot 0.5\cdot\left(determ_{10}-determ_8\right)\cdot\left(determ_3+determ_4\right)^2}$$

0.983

$$\gamma_{M_{v,\,wat}} := \frac{\frac{1}{6}\cdot design_2}{\frac{1}{6}\cdot determ_2}\cdot\frac{0.5\cdot design_8\cdot design_1\cdot design_2}{0.5\cdot determ_8\cdot determ_1\cdot determ_2}$$

1.164

$$\gamma_{F_{v,\,wat}} := \frac{0.5\cdot design_8\cdot design_1\cdot design_2}{0.5\cdot determ_8\cdot determ_1\cdot determ_2}$$

1.164

$$\gamma_{F_{v,\,conc}} := \frac{\left(2\cdot design_5\cdot design_3+design_6\cdot design_2+design_7\cdot design_2\right)\cdot design_{11}}{\left(2\cdot determ_5\cdot determ_3+determ_6\cdot determ_2+determ_7\cdot determ_2\right)\cdot determ_{11}}$$

0.992

$$\gamma_{F_{v,\,soil}} := \frac{design_2\cdot design_4\cdot design_9}{determ_2\cdot determ_4\cdot determ_9}$$

0.955

D.3 Strength wall

D.3.1 Failure probability of the design

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

$restart$

$with(stats):$

$with(Statistics):$

$with(plots):$

$interface(rtablesize = 15):$

$unprotect(\gamma)$

$n := 8:$

$J := 10:$

Creating matrices to be able to store and call values:

$\mu := Matrix(n, J):$

$\sigma := Matrix(n, J):$

$d := Matrix(n, 1):$

$\sigma z := Matrix(1, J):$

$\mu z := Matrix(1, J):$

$\beta := Matrix(1, J):$

$P := Matrix(1, J):$

$\alpha := Matrix(n, J):$

$X := Matrix(n, J + 1):$

$Xkar := Matrix(n, 1):$

$q := Matrix(1, J):$

Defining the performance function:

$R := f_s \cdot A_s \cdot 0.75 \cdot d_w:$

$S := M_{w1} + M_{w2} + M_{s1} + M_{s2}:$

$M_{w1} := 0.064 \cdot h \cdot \gamma_w \cdot H^2:$

$M_{w2} := \frac{1}{8} \cdot (h - H) \cdot \gamma_w \cdot H^2:$

$M_{s1} := 0.064 \cdot H \cdot (\gamma_s - \gamma_w) \cdot H^2 \cdot K_a:$

$M_{s2} := \frac{1}{8} \cdot d_s \cdot (\gamma_s - \gamma_w) \cdot H^2 \cdot K_a:$

$K_a := 0.7:$

$Z := R - S$

$$0.750 f_s A_s d_w - 0.064\, h\, \gamma_w H^2 - \frac{1}{8}\,(h - H)\,\gamma_w H^2 - 0.045\, H^3 \left(\gamma_s - \gamma_w\right) - 0.088\, d_s \left(\gamma_s - \gamma_w\right) H^2$$

$parameters := \left(h, H, d_s, d_w, \gamma_w, \gamma_s, f_s, A_s\right):$

$Z := unapply(Z, parameters):$

Calculating the partial derivatives of the performance function:

$\textbf{\textit{for}}\ i\ \textbf{\textit{from}}\ 1\ \textbf{\textit{to}}\ n\ \textbf{\textit{do}}\ d_{i,\,1} := unapply\left(\frac{d}{d\,parameters_i} Z(parameters),\, parameters\right) \textbf{\textit{end}}\ \mathbf{do}:$

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1:$
$parameters_i$

$$h$$

$\mu_{i,..} := 0:$
$\sigma_{i,..} := 0:$

$i := 2:$
$parameters_i$

$$H$$

$\mu_{i,..} := 4:$
$\sigma_{i,..} := 0.03:$

$i := 3:$
$parameters_i$

$$d_s$$

$\mu_{i,..} := 1:$
$\sigma_{i,..} := 0.05:$

$i := 4:$
$parameters_i$

$$d_w$$

$\mu_{i,..} := 0.4:$
$\sigma_{i,..} := 0.01:$

$i := 5:$
$parameters_i$

$$\gamma_w$$

$\mu_{i,..} := 10:$
$\sigma_{i,..} := 0.2:$

$i := 6:$
$parameters_i$

$$\gamma_s$$

$\mu_{i,..} := 20:$
$\sigma_{i,..} := 1.0:$

$i := 7:$
$parameters_i$

$$f_s$$

$\mu_{i,..} := 0:$
$\sigma_{i,..} := 0:$

$i := 8:$
$parameters_i$

$$A_s$$

$\mu_{i,..} := 0.0009:$
$\sigma_{i,..} := 0.00001:$

Defining that the first design points are the mean values of the parameters:

for i from 1 to n do $X_{i,\,1} := \mu_{i,\,1}$ *: end do:*

Generating a table with the mean values and standard deviations:

$$UNITS := \left\langle 'm','m','m','m','\frac{kN}{m^3}','\frac{kN}{m^3}','\frac{kN}{m^2}','m^2' \right\rangle :$$

$$\langle \langle 'parameter'|'\mu'|'\sigma'|'unit' \rangle, \langle \langle parameters \rangle \,|\mu_{..,\,1}|\sigma_{..,\,1}|UNITS \rangle \rangle$$

parameter	μ	σ	unit
h	0	0	m
H	4	0.030	m
d_s	1	0.050	m
d_w	0.400	0.010	m
γ_w	10	0.200	$\frac{kN}{m^3}$
γ_s	20	1.000	$\frac{kN}{m^3}$
f_s	0	0	$\frac{kN}{m^2}$
A_s	0.001	0.000	m^2

Defining the cumulative, inverse and probability density functions of the normal distribution:

$$\Phi := unapply(evalf(CDF(Normal(0, 1), x)), x) :$$

$$\Phi I := unapply(Quantile(Normal(0, 1), x), x, numeric) :$$

$$\varphi := unapply(evalf(PDF(Normal(0, 1), x)), x) : plot(\varphi(x), x) :$$

Determining the parameters for the Gumbel distribution:

$$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000 \rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17 \rangle, v)$$

$$10.429 + 0.163 \ln(v)$$

$$\alpha 1 := tcoeff(l, \ln(v)) - 8 :$$

$$\beta 1 := coeff(l, \ln(v)) :$$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$$F := unapply(evalf(CDF(Gumbel(\alpha 1, \beta 1), x)), x) :$$

$$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric) :$$

$$f := unapply(evalf(PDF(Gumbel(\alpha 1, \beta 1), x)), x) :$$

Defining the first design point of the water level:

$$X_{I,\,I} := FI(\mathrm{evalf}(1 - 10^{-4}))$$

$$3.928$$

Determining the parameters for the lognormal distribution:

$$\beta 2 := evalf\left(\mathrm{sqrt}\left(\ln\left(1 + \frac{35000^2}{500000^2}\right)\right)\right) :$$

$$\alpha 2 := evalf(\ln(500000 - 0.5 \cdot \beta 2^2)) :$$

Defining the cumulative, inverse and probability density functions of the lognormal distribution:

$LND := unapply(evalf(CDF(LogNormal(\alpha 2, \beta 2), x)), x)$:

$LNID := unapply(Quantile(LogNormal(\alpha 2, \beta 2), x), x, numeric)$:

$lnd := unapply(evalf(PDF(LogNormal(\alpha 2, \beta 2), x)), x)$: $plot(lnd(x), x = 0..1000000)$:

Defining the first design point of the water level:

$X_{7,1} := 435000$:

Running the FORM analysis for J iterations:

for j **from** 1 **to** J **do**

$\sigma_{1,j} := evalf\left(\dfrac{\varphi(\Phi I(F(X_{1,j})))}{f(X_{1,j})}\right)$:

$\mu_{1,j} := evalf(X_{1,j} - \Phi I(F(X_{1,j}))\cdot\sigma_{1,j})$:

$\sigma_{7,j} := evalf\left(\dfrac{\varphi(\Phi I(LND(X_{7,j})))}{lnd(X_{7,j})}\right)$:

$\mu_{7,j} := evalf(X_{7,j} - \Phi I(LND(X_{7,j}))\cdot\sigma_{7,j})$:

$\mu z_{1,j} := Z(seq(X_{i,j}, i = 1..n)) + add(d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot(\mu_{i,j} - X_{i,j}), i = 1..n)$:

$\sigma z_{1,j} := \sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}$:

$\beta_{1,j} := \dfrac{\mu z_{1,j}}{\sigma z_{1,j}}$:

$P_{1,j} := \Phi(-\beta_{1,j})$:

for i **from** 1 **to** n **do**

$\alpha_{i,j} := -\dfrac{d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j}}{\sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n))\cdot\sigma_{i,j})^2, i = 1..n)}}$:

$X_{i,j+1} := \mu_{i,j} + \alpha_{i,j}\cdot\beta_{1,j}\cdot\sigma_{i,j}$:

end do:

$X_{1,j+1} := evalf(FI(\Phi(\alpha_{1,j}\cdot\beta_{1,j})))$:

$X_{7,j+1} := evalf(LNID(\Phi(\alpha_{7,j}\cdot\beta_{1,j})))$:

end do:

Generating a table containing the mean value, standard deviation, reliability index and failure probability of the performance function of each iteration:

$\langle\langle 'iteration', '\mu', '\sigma', '\beta', 'P_f' \rangle | \langle convert(\langle seq(i, i = 1..J)\rangle, vector), \langle \mu z\rangle, \langle \sigma z\rangle, \langle\beta\rangle, \langle P\rangle\rangle\rangle$

iteration	1	2	3	4	5	6	7	8	9	10
μ	124.810	153.400	156.275	155.976	155.997	155.996	155.996	155.996	155.996	155.996
σ	21.838	27.832	28.335	28.281	28.285	28.284	28.284	28.284	28.284	28.284
β	5.715	5.512	5.515	5.515	5.515	5.515	5.515	5.515	5.515	5.515
P_f	$5.479\ 10^{-9}$	$1.777\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$	$1.741\ 10^{-8}$

Generating a table containing the failure probability, exceedance probability and the difference:

$\langle\langle 'P_f', 'exc\,prob\,h', 'difference' \rangle | \langle P_{1,J}, 1 - F(X_{1,J+1}), (1 - F(X_{1,J+1})) - P_{1,J}\rangle\rangle$

P_f	$1{,}74 \times 10^{-8}$
exc prob h	$1{,}26 \times 10^{-7}$
difference	$1{,}09 \times 10^{-7}$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$determ := \mu_{..,J}$:

$determ_1 := 4$:

$determ_7 := 500000$:

$design := X_{..,J+1}$:

$\gamma := \frac{design}{\sim determ}$:

$\langle\langle 'nr' | 'sym' | '\alpha' | 'X_d' | 'X' | '\gamma' \rangle, \langle\langle seq(i, i = 1..n)\rangle | \langle parameters\rangle | \alpha_{..,J} | design | determ | \gamma\rangle\rangle$

nr	sym	α	X_d	X	γ
1	h	0.935	5.015	4	1.254
2	H	0.050	4.008	4	1.002
3	d_s	0.027	1.007	1	1.007
4	d_w	−0.106	0.394	0.400	0.985
5	γ_w	0.020	10.022	10	1.002
6	γ_s	0.152	20.839	20	1.042
7	f_s	−0.293	$4.466\ 10^5$	500000	0.893
8	A_s	−0.047	0.001	0.001	0.997

Calculating the partial factors of the forces:

$$\gamma_{M_R} := \frac{design_7 \cdot design_8 \cdot design_4}{determ_7 \cdot determ_8 \cdot determ_4}$$

0.878

$$\gamma_{M_w} := \frac{0.064 \cdot design_1 \cdot design_5 \cdot design_2^2 + \frac{1}{8} \cdot (design_1 - design_2) \cdot design_5 \cdot design_2^2}{0.064 \cdot determ_1 \cdot determ_5 \cdot determ_2^2 + \frac{1}{8} \cdot (determ_1 - determ_2) \cdot determ_5 \cdot determ_2^2}$$

1.757

$$\gamma_{M_s} := \frac{0.064 \cdot design_2 \cdot (design_6 - design_5) \cdot design_2^2 \cdot K_a + \frac{1}{8} \cdot design_3 \cdot (design_6 - design_5) \cdot design_2^2 \cdot K_a}{0.064 \cdot determ_2 \cdot (determ_6 - determ_5) \cdot determ_2^2 \cdot K_a + \frac{1}{8} \cdot determ_3 \cdot (determ_6 - determ_5) \cdot determ_2^2 \cdot K_a}$$

1.090

D.3.2 Calibration of the partial factors

Start of the script, loading some packages, setting the number of parameters (n) and the amount of iteration steps (J):

```
restart
with(stats) :
with(Statistics) :
with(plots) :
interface(rtablesize = 15) :
unprotect(γ)
n := 8 :
J := 10 :
```

Creating matrices to be able to store and call values:

```
μ := Matrix(n, J) :
σ := Matrix(n, J) :
d := Matrix(n, 1) :
σz := Matrix(1, J) :
μz := Matrix(1, J) :
β := Matrix(1, J) :
P := Matrix(1, J) :
α := Matrix(n, J) :
X := Matrix(n, J + 1) :
Xkar := Matrix(n, 1) :
q := Matrix(1, J) :
```

Defining the performance function:

$R := f_s \cdot A_s \cdot 0.75 \cdot d_w :$

$S := M_{w1} + M_{w2} + M_{s1} + M_{s2} :$

$M_{w1} := 0.064 \cdot h \cdot \gamma_w \cdot H^2 :$

$M_{w2} := \frac{1}{8} \cdot (h - H) \cdot \gamma_w \cdot H^2 :$

$M_{s1} := 0.064 \cdot H \cdot \left(\gamma_s - \gamma_w\right) \cdot H^2 \cdot K_a :$

$M_{s2} := \frac{1}{8} \cdot d_s \cdot \left(\gamma_s - \gamma_w\right) \cdot H^2 \cdot K_a :$

$K_a := 0.7 :$

$Z := R - S$

$$0.750 f_s A_s d_w - 0.064\, h\, \gamma_w H^2 - \frac{1}{8}\,(h - H)\, \gamma_w H^2 - 0.045\, H^3 \left(\gamma_s - \gamma_w\right) - 0.088\, d_s \left(\gamma_s - \gamma_w\right) H^2$$

$parameters := \left(h, H, d_s, d_w, \gamma_w, \gamma_s, f_s, A_s\right) :$

$Z := unapply(Z, parameters) :$

Calculating the partial derivatives of the performance function:

$\textbf{for } i \textbf{ from } 1 \textbf{ to } n \textbf{ do } d_{i,\,1} := unapply\left(\frac{d}{d\, parameters_i} Z(parameters),\, parameters\right) \textbf{end do}:$

Defining the mean values and standard deviations of the normally distributed parameters:

$i := 1 :$
$parameters_i$

h

$\mu_{i,..} := 0 :$
$\sigma_{i,..} := 0 :$

$i := 2 :$
$parameters_i$

H

$\mu_{i,..} := 4 :$
$\sigma_{i,..} := 0.03 :$

$i := 3 :$
$parameters_i$

d_s

$\mu_{i,..} := 1 :$
$\sigma_{i,..} := 0.05 :$

$i := 4 :$
$parameters_i$

d_w

$\mu_{i,..} := 0.4 :$
$\sigma_{i,..} := 0.01 :$

$i := 5 :$
$parameters_i$

γ_w

$\mu_{i,..} := 10 :$
$\sigma_{i,..} := 0.2 :$

$i := 6 :$
$parameters_i$

γ_s

$\mu_{i,..} := 20 :$
$\sigma_{i,..} := 1.0 :$

$i := 7 :$
$parameters_i$

f_s

$\mu_{i,..} := 0 :$
$\sigma_{i,..} := 0 :$

$i := 8 :$
$parameters_i$

A_s

$\mu_{i,..} := 0.0009 :$
$\sigma_{i,..} := 0.00001 :$

Defining that the first design points are the mean values of the parameters:

for *i* **from** *1* **to** *n* **do** $X_{i,\,1} := \mu_{i,\,1}$: **end do**:

Generating a table with the mean values and standard deviations:

$UNITS := \left\langle 'm','m','m','m',' \frac{kN}{m^3} ',' \frac{kN}{m^3} ',' \frac{kN}{m^2} ','m^2' \right\rangle :$

$\left\langle \langle 'parameter' | 'μ' | 'σ' | 'unit' \rangle, \left\langle \langle parameters \rangle \middle| μ_{..,1} \middle| σ_{..,1} \middle| UNITS \right\rangle \right\rangle$

parameter	μ	σ	unit
h	0	0	m
H	4	0.030	m
d_s	1	0.050	m
d_w	0.400	0.010	m
γ_w	10	0.200	$\frac{kN}{m^3}$
γ_s	20	1.000	$\frac{kN}{m^3}$
f_s	0	0	$\frac{kN}{m^2}$
A_s	0.001	0.000	m^2

Defining the cumulative, inverse and probability density functions of the normal distribution:

$\Phi := unapply(evalf(CDF(Normal(0, 1), x)), x) :$

$\Phi I := unapply(Quantile(Normal(0, 1), x), x, numeric) :$

$\varphi := unapply(evalf(PDF(Normal(0, 1), x)), x) : plot(\varphi(x), x) :$

Determining the parameters for the Gumbel distribution:

$l := LogarithmicFit(\langle 1250, 1250, 1250, 100000, 100000, 100000 \rangle, \langle 11.67, 11.55, 11.55, 12.37, 12.37, 12.17 \rangle, v)$

$$10.429 + 0.163 \ln(v)$$

$\alpha 1 := tcoeff(l, \ln(v)) - 8 :$

$\beta 1 := coeff(l, \ln(v)) :$

Defining the cumulative, inverse and probability density functions of the Gumbel distribution:

$F := unapply(evalf(CDF(Gumbel(\alpha 1, \beta 1), x)), x) :$

$FI := unapply(Quantile(Gumbel(\alpha 1, \beta 1), x), x, numeric) :$

$f := unapply(evalf(PDF(Gumbel(\alpha 1, \beta 1), x)), x) :$

Defining the first design point of the water level:

$X_{1,1} := FI(\mathrm{evalf}(1 - 10^{-4}))$

$$3.928$$

Determining the parameters for the lognormal distribution:

$\beta 2 := evalf\left(\mathrm{sqrt}\left(\ln\left(1 + \frac{35000^2}{500000^2}\right)\right)\right) :$

$\alpha 2 := evalf(\ln(500000 - 0.5 \cdot \beta 2^2)) :$

Defining the cumulative, inverse and probability density functions of the lognormal distribution:

$LND := unapply(evalf(CDF(LogNormal(\alpha 2, \beta 2), x)), x) :$

$LNID := unapply(Quantile(LogNormal(\alpha 2, \beta 2), x), x, numeric)$:
$lnd := unapply(evalf(PDF(LogNormal(\alpha 2, \beta 2), x)), x)$: $plot(lnd(x), x = 0..1000000)$:

Defining the first design point of the water level:

$X_{7,1} := 435000$:

Running the FORM analysis for J iterations:

for *j* **from** 1 **to** *J* **do**
$\sigma_{1,j} := evalf\left(\frac{\varphi(\Phi I(F(X_{1,j})))}{f(X_{1,j})}\right)$:
$\mu_{1,j} := evalf(X_{1,j} - \Phi I(F(X_{1,j})) \cdot \sigma_{1,j})$:
$\sigma_{7,j} := evalf\left(\frac{\varphi(\Phi I(LND(X_{7,j})))}{lnd(X_{7,j})}\right)$:
$\mu_{7,j} := evalf(X_{7,j} - \Phi I(LND(X_{7,j})) \cdot \sigma_{7,j})$:
$\beta_{1,j} := evalf(-\Phi I(10^{-6}))$:
for *i* **from** 1 **to** *n* **do**
$\alpha_{i,j} := -\frac{d_{i,1}(seq(X_{i,j}, i = 1..n)) \cdot \sigma_{i,j}}{\sqrt{add((d_{i,1}(seq(X_{i,j}, i = 1..n)) \cdot \sigma_{i,j})^2, i = 1..n)}}$:
$X_{i,j+1} := \mu_{i,j} + \alpha_{i,j} \cdot \beta_{1,j} \cdot \sigma_{i,j}$:
end do:
$X_{1,j+1} := evalf(FI(\Phi(\alpha_{1,j} \cdot \beta_{1,j})))$:
$X_{7,j+1} := evalf(LNID(\Phi(\alpha_{7,j} \cdot \beta_{1,j})))$:
end do:

Generating a table containing the failure probability, exceedance probability and the difference:

$\langle\langle 'P_f', 'exc\,prob\,h', 'difference' \rangle | \langle P_{1,J}, 1 - F(X_{1,J+1}), (1 - F(X_{1,J+1})) - P_{1,J} \rangle\rangle$

$$\begin{bmatrix} P_f & 0{,}00 \times 10^{0} \\ exc\,prob\,h & 6{,}98 \times 10^{-6} \\ difference & 6{,}98 \times 10^{-6} \end{bmatrix}$$

Generating a table containing the influence factors, design points and partial factors for each parameter:

$determ := \mu_{..,J}$:
$determ_1 := 4$:
$determ_7 := 500000$:
$design := X_{..,J+1}$:
$\gamma := \frac{design}{\sim determ}$:
$\langle\langle 'nr' | 'sym' | '\alpha' | 'X_d' | 'X' | '\gamma' \rangle, \langle\langle seq(i, i = 1..n)\rangle | \langle parameters \rangle | \alpha_{..,J} | design | determ | \gamma \rangle\rangle$

nr	sym	α	X_d	X	γ
1	h	0.914	4.362	4	1.090
2	H	0.045	4.006	4	1.002
3	d_s	0.031	1.007	1	1.007
4	d_w	-0.122	0.394	0.400	0.985
5	γ_w	0.007	10.007	10	1.001
6	γ_s	0.174	20.829	20	1.041
7	f_s	-0.337	$4.471\ 10^5$	500000	0.894
8	A_s	-0.054	0.001	0.001	0.997

Calculating the partial factors of the forces:

$$\gamma_{M_R} := \frac{\text{design}_7 \cdot \text{design}_8 \cdot \text{design}_4}{\text{determ}_7 \cdot \text{determ}_8 \cdot \text{determ}_4}$$

0.879

$$\gamma_{M_w} := \frac{0.064 \cdot \text{design}_1 \cdot \text{design}_5 \cdot \text{design}_2^2 + \frac{1}{8} \cdot \left(\text{design}_1 - \text{design}_2\right) \cdot \text{design}_5 \cdot \text{design}_2^2}{0.064 \cdot \text{determ}_1 \cdot \text{determ}_5 \cdot \text{determ}_2^2 + \frac{1}{8} \cdot \left(\text{determ}_1 - \text{determ}_2\right) \cdot \text{determ}_5 \cdot \text{determ}_2^2}$$

1.269

$$\gamma_{M_s} := \frac{0.064 \cdot \text{design}_2 \cdot \left(\text{design}_6 - \text{design}_5\right) \cdot \text{design}_2^2 \cdot K_a + \frac{1}{8} \cdot \text{design}_3 \cdot \left(\text{design}_6 - \text{design}_5\right) \cdot \text{design}_2^2 \cdot K_a}{0.064 \cdot \text{determ}_2 \cdot \left(\text{determ}_6 - \text{determ}_5\right) \cdot \text{determ}_2^2 \cdot K_a + \frac{1}{8} \cdot \text{determ}_3 \cdot \left(\text{determ}_6 - \text{determ}_5\right) \cdot \text{determ}_2^2 \cdot K_a}$$

1.089

D.4 Validation of Maple with VaP

The FORM analyses carried out with Maple for the horizontal stability, overturning stability and strength of the wall are validated with VaP. This software is able to deal with stochastic variables in mathematical expressions. The program is used for the reliability analyses of the failure mechanisms. The same performance functions, as are used in Maple, are inserted into the program as well as the distributions of the parameters. After that the FORM analyses and Monte Carlo simulations are performed. However, the Monte Carlo simulation is limited to 1E6 samples which makes impossible to obtain a failure probability of 1E-7 or smaller. The results of the FORM analyses from Maple are compared with the results of VaP and presented in Table D-1.

Table D-1: Partial load factors

Failure mechanism	Failure probability	
	Maple	VaP
Horizontal stability	5.27E-8	5.32E-8
Overturning stability	5.53E-7	5.27E-7
Strength wall	1.74E-8	1.61E-8

The results of the calculations done with the two programs are almost the same. The differences between the two programs might be the difference in accuracy of the calculations, but are so small that they are negligible. The input and output of the VaP calculations are presented in the upcoming paragraphs.

D.4.1 Horizontal stability

```
Limit State Function G :

    G = tan((2/3)*p)*(L*ds*ys1+(2*dw*H+dr*L+db*L)*yc-0.5*yw*h*L)
         -0.5*yw*h^2-0.35*(ys2-yw)*(H+ds)^2

Variables of G:

    H       N          4.000          0.030
    L       N         30.000          0.030
    db      N          0.500          0.010
    dr      N          0.500          0.010
    ds      N          1.000          0.050
    dw      N          0.400          0.010
    h       GL         2.415          6.107
    p       N         32.500          2.000
    yc      N         25.000          0.500
    ys1     N         18.000          0.900
    ys2     N         20.000          1.000
    yw      N         10.000          0.200

FORM Analysis of G:

    HL - Index = 5.32         P(G<0) = 5.32e-08

    Name         Alpha        Design Value
    H            0.009          4.001
    L           -0.002         30.000
    db          -0.029          0.498
    dr          -0.029          0.498
    ds          -0.083          0.978
    dw          -0.008          0.400
    h            0.969          5.012
    p           -0.159         30.807
    yc          -0.064         24.829
    ys1         -0.103         17.510
    ys2          0.090         20.478
    yw           0.066         10.071

Crude Monte Carlo Analysis of G:

    1 run with 1000000 samples:
    1.   m = 275.717      s = 37.2418     p = 0
```

D.4.2 Overturning stability

```
Limit State Function G :

    G = 1/6*L-(0.167*yw*h^3+0.117*(ys2-yw)*(H+ds)^3+0.083*yw*(h-0.5)*L^2)
          /((2*dw*H+dr*L+db*L)*yc+L*ds*ys1-0.5*yw*(h-0.5)*L)

Variables of G:

    H       N          4.000         0.030
    L       N         30.000         0.030
    db      N          0.500         0.010
    dr      N          0.500         0.010
    ds      N          1.000         0.050
    dw      N          0.400         0.010
    h       GL         2.415         6.107
    yc      N         25.000         0.500
    ys1     N         18.000         0.900
    ys2     N         20.000         1.000
    yw      N         10.000         0.200

FORM Analysis of G:

    HL - Index = 4.88         P(G<0) = 5.27e-07

    Name         Alpha         Design Value
    H          -2.757e-04         4.000
    L          -1.805e-04        30.000
    db           -0.028          0.499
    dr           -0.028          0.499
    ds           -0.094          0.977
    dw           -0.007          0.400
    h             0.984          4.716
    yc           -0.061         24.851
    ys1          -0.098         17.571
    ys2           0.011         20.052
    yw            0.094         10.092

Crude Monte Carlo Analysis of G:

    1 run with 1000000 samples:
    1.    m = 3.42381      s = 0.221555     p = 0
```

D.4.3 Strength wall

```
Limit State Function G :

    G = 0.75*fs*As*dw-0.064*h*yw*H^2-(1/8)*(h-H)*yw*H^2
         -0.045*H^3*(ys-yw)-0.088*ds*(ys-yw)*H^2

Variables of G:

    As      N        9.000e-04      1.000e-05
    H       N            4.000          0.030
    ds      N            1.000          0.050
    dw      N            0.400          0.010
    fs      LN          13.120          0.065
    h       GL           2.415          6.107
    ys      N           20.000          1.000
    yw      N           10.000          0.200

FORM Analysis of G:

    HL - Index = 5.53          P(G<0)  = 1.61e-08

    Name         Alpha         Design Value
    As          -0.047          8.974e-04
    H            0.050            4.008
    ds           0.027            1.007
    dw          -0.107            0.394
    fs          -0.273          4.523e+05
    h            0.941            5.057
    ys           0.152           20.838
    yw           0.021           10.023

Crude Monte Carlo Analysis of G:

    1 run with 1000000 samples:
    1.    m = 96.2153       s = 12.2937      p = 0
```