Flood Protection Using Multiple Lines of Dikes

A Case Study of the Twin Dike Eemshaven-Delfzijl Project

C.W. Wauben

Technische Universiteit Delft

HKV

TU Delft Delft University of Technology

Waterschap Noorderzijlvest

Flood Protection Using Multiple Lines of Dikes

A Case Study of the Twin Dike Eemshaven-Delfzijl Project

by

C.W. Wauben

to obtain the degree of Master of Science
at the Delft University of Technology,
to be defended publicly on Wednesday 10 July 2019 at 14:00.

Student number:	4243498	
Project duration:	September, 2018 – July, 2019	
Thesis committee:	Prof. dr. ir. M. Kok	TU Delft, chairman
	Ir. J. Tigchelaar	HKV Consultants
	Ir. K. de Jong	Water board Noorderzijlvest
	C.H. Lashley MSc	TU Delft
	Ir. R.J.C. Marijnissen	Wageningen University
	Ir. J.P. van den Bos	TU Delft

An electronic version of this thesis is available at `http://repository.tudelft.nl/`.

Cover photo: Wadden Sea dike west of Eemshaven. In the left, North Cape, the most northern point of the Dutch mainland. 29 March 2019.

Abstract

Flood defences in the Netherlands are regularly assessed in order to see if they satisfy the required safety norms. During the third assessment round (2006-2011) an 11.7 km stretch of dike in the north-eastern part of the Netherlands did not satisfy the required safety norms. Traditionally, a dike is reinforced by elevating the crest, widening the body and strengthening the revetment on the outer slope. Elevating and widening a dike is sometimes an unfavourable option due to the possible loss of valuable natural areas, the obstruction of the view, large costs and the need to demolish buildings. Therefore, on a 2.5 km stretch of this dike, an innovative concept is implemented to achieve the required flood safety while being able to preserve valuable areas: the Twin Dike project.

A lower, second dike is constructed behind the original dike. The original dike is only being strengthened to a minimal extent. This way more water is able to come over the original dike compared to a regular reinforcement. The second dike will make sure water coming over the original dike is retained in the basin in between the two dikes. The original (outer) dike reduces the loads on the second (inner) dike in such a way that the two dikes together provide the required safety. The general idea behind this project is to more efficiently use the area near the dike without losing valuable nature or buildings during dike reinforcements. At the same time, this concept is expected to be cheaper, provides the required flood safety and the area between the dikes is used for experimental purposes like saline agriculture. Currently, there are no pre-defined methods which can be used to easily assess the safety requirements of multiple lines of dikes. Therefore, the following research question is presented:

Looking at the Twin Dike project as a case, is the concept of multiple dikes behind each other an attractive alternative for dike reinforcements, regarding flood safety and costs, to provide the required protection against floods?

In order to answer the research question, it is analysed how flood defences are currently assessed in the Netherlands. Governing loads and failure mechanisms for the Twin Dike are determined. Using current and experimental methods the loads acting on both dikes are determined. This way a conclusion can be presented on the safety provided by the Twin Dike project. Next, a sensitivity analysis is performed to analyse the sensitivity of various aspects on the results. Finally, a cost-benefit analysis is performed. This analysis shows if the Twin Dike can be an attractive concept regarding costs.

According to the Dutch law, the primary flood defences at the location of the Twin Dike project have a maximum allowable probability of causing a flood of 1/3,000 per year. A flood is defined as the loss of water retaining function of a flood defence, causing fatalities or considerable damage to the area protected by the flood defence. The Statutory Assessment Instruments (WBI) and Design Instruments (OI) are used to assess and design these flood defences. These instruments state rules and methods that need to be followed during an assessment or design phase. Multiple failure mechanisms can lead to the failure of a flood defence which all need to be taken into account. The failure mechanisms each have a certain contribution to the total probability of failure. This means that the required safety norm for a single failure mechanism is more strict than the total allowable probability of failure of a flood defence.

This research concludes that erosion of the outer slope and overtopping are governing failure mechanisms for the outer dike of the Twin Dike project. Overtopping volumes are, however, not large enough to cause considerable damage to both the outer and inner dike. Existing assessment methods are not good enough to determine the safety of the Twin Dike project with respect to erosion of the outer slope. In order to more accurately assess the effects of erosion of the outer dike and the consequent loads on the inner dike, a prototype erosion model is used.

The erosion model which is used can calculate erosion- and consequent overtopping volumes over a dike. The model showed that breaches are formed in the outer dike at conditions which have a probability of occurrence of 1/3,000 per year. Since this is a 2D model, a breach width of 20 m was assumed. When such a breach occurs, the water level in the basin in between the two dikes quickly levels with the sea water level. The hydraulic loads are then such, that the inner dike experiences overtopping volumes of hundreds l/s/m. The inner dike cannot withstand these amounts of water. When a breach in the outer dike occurs, a total system failure immediately occurs. The Twin Dike system has a maximum allowable probability of failure of 1/180,000 per year for the failure mechanism erosion of the outer slope. This safety requirement is thus not met.

A sensitivity analysis was performed on the overtopping volumes and erosion model results. Sea level rise was the dominant uncertainty for the overtopping volumes. If sea levels will rise extremely within the design lifetime, overtopping volumes can be three times as large compared to the case with mean sea level rise. However, the overtopping volumes are still low enough such that Twin Dike satisfies the required safety norms. For the erosion model, the average return period for which a breach occurs varies greatly with the uncertainty of various parameters. There is thus a large uncertainty in the exact probability of failure of the Twin Dike. However, using a case with mean values for all variables, the probability of a breach is still larger than allowed by the safety norms.

Also, a cost-benefit analysis was performed to assess if the Twin Dike project is an attractive solution with regards to costs. From the analysis, it was found that the Twin Dike project has the potential to be cheaper than a regular integral reinforcement. However, to satisfy the required safety norms the Twin Dike has to be modified making the costs larger than for a regular integral reinforcement. In this case, from a cost perspective, multiple lines of dikes are not an optimal solution. When this concept is implemented on a larger scale, or when a second dike of sufficient height is already present in the landscape, costs might be reduced making the Twin Dike concept a more favourable option.

Based on the analysis of the Twin Dike project, multiple lines of dikes are not an attractive alternative to traditional dike reinforcement regarding costs and flood protection when existing methods are used. The Twin Dike project does not meet the safety requirements according to the methods stated in the WBI and OI. The failure mechanism erosion of the outer slope can cause breaches in the outer dike with a higher probability than allowed. When this happens, the inner dike cannot provide additional safety, leading to a total system failure. However, the sensitivity analysis showed that there is large uncertainty in the results. Regarding costs, multiple lines of dikes seem not to be the most attractive alternative.

However, this research investigated the failure of the Twin Dike system while the Dutch law states a maximum allowable probability of causing a flood for flood defences. A failure of a flood defence does not equal a flood. To fully assess the concept of multiple lines of dikes, it has to be assessed if the occurring breaches indeed lead to an event which is classified as a flood.

Furthermore, there are uncertainties in the load- and resistance parameters in the area of the Twin Dike. Simulated storms and eyewitness reports give the appearance that loads determined with the regular instruments are possibly overestimated. Also, exact soil characteristics and layout were not determined with the desired precision. An in-depth analysis to further investigate the load- and resistance parameters in the area is recommended to improve the precision of the results.

Finally, the erosion model that was used is a prototype model. In order to increase the reliability of the results, the model has to be further validated with other models and real-life situations. Also, the erosion model did not take into account stone revetments and 3D effects. Erosion mostly occurred above the stone revetment and already small breaches are able to cause a total system failure. Still, for a more precise statement on the total amount of water being able to come through a breach, 3D effects and a stone revetment have to be taken into account.

Preface

This report is the result of my graduation work for the Master of Science Hydraulic Engineering, specialising in Hydraulic Structures and Flood Risk, at the Delft University of Technology. The research was performed in close collaboration with HKV Consultants and water board Noorderzijlvest.

Firstly, I would like to thank all the members of my graduation committee for their help and constructive criticism. Without them, I would not have been able to achieve these results. I want to express special gratitude to Jan, who was my main supervisor from HKV. Thank you for setting me up with the right people when this was necessary, helping me feel at ease at HKV and helping me with my research, also in times when I was not so sure what to do any more. I also want to thank Matthijs for helping me find this interesting graduation subject. Your enthusiasm definitely helped me in performing this research with more fun. Special thanks go to Kees for making this research possible and providing me with loads of information from water board Noorderzijlvest.

Furthermore, I would like to thank all my friends in and beyond Delft for their mental support and good times, especially Floor. They really helped me through this journey and definitely played their part in making my graduation possible.

Finally, I express my gratitude to everyone at HKV who helped me with my research. Thanks to the employees who helped me work with different models and the gathering of various data. Appreciations go to fellow graduate students at HKV who ensured enjoyable lunch breaks and, in the earlier times of my graduation work, exciting table tennis games.

I hope you enjoy reading.

C.W. Wauben
Delft, June 2019

Contents

List of Tables

List of Figures

1

Introduction

1.1. Background

Flood defences in the Netherlands are regularly assessed in order to see if they still satisfy the required safety norms. Weather conditions, sea level rise, soil subsidence, insights in the behaviour of flood defences and population are all factors that influence the required dimensions of flood defences. In the past few decades, there have been three assessment rounds in which all primary Dutch flood defences have been assessed. The last round lasted from 2006 till 2011. In this round, about a third of the flood defences in the Netherlands did not meet the required safety norms (Inspectie Leefomgeving en Transport, 2013).

With dikes, this problem has traditionally been tackled by increasing the height of the dike and/or strengthening the dike. When this is done, a dike can withstand higher water levels and more severe storms: the safety level is increased. However, heightening of dikes is sometimes also a controversial measure. People living near dikes may not find a dike heightening favourable. A crest level that keeps increasing may not be beneficial for the view. Also, when a dike is heightened its base has to become wider to ensure that the stability of the dike is sufficient. So, with every dike heightening, its footprint will also increase. Dike reinforcement programmes sometimes result in the loss of valuable nature and landscape, houses and sometimes entire villages (van der Velden, 1995).

From 1890 to 2014 sea levels have risen approximately 23 cm (Baart et al., 2018; CBS, PBL, RIVM, & WUR, 2016) in the Netherlands. Predictions of sea level rise for the next century are spread out. Haasnoot et al. (2018) predicts sea levels will rise 0.25 m to more than 3 m in 2100 compared to 1995. However, Baart et al. (2018) found that the average sea level rise in the Netherlands was constant from 1890 to 2017. This could mean that sea level rise will not be so extreme in this century. Either way, sea levels are rising, which means flood defences will have to keep being reinforced in the future.

New solutions are sought in order to satisfy safety norms in flood-prone areas, without increasing dike levels. Recently, multiple projects have been conducted that implement new solutions.

For example, the Room for the Rivers project is being implemented along the main Dutch rivers. By giving more space to the water, the water level decreases. This (partly) eliminates the need to elevate the dikes in this area (Ruimte voor de Rivier, n.d.). Also, a large coastal nourishment along the Hondsbossche- and Pettemer sea defence is eliminating the need to strengthen this sea defence the traditional way and possibly having to move part of a village (Hoogwaterbeschermingsprogramma, n.d.). Another location where an alternative to traditional dike heightening is being developed is in the north-eastern part of the Netherlands. A dike in the province of Groningen, next to the Ems-Dollard estuary, will not be elevated. Instead, a small second dike is constructed behind the existing dike.

In between the port of Eemshaven and the city of Delfzijl, an 11.7 km stretch of dike has not met the required safety norms in the third assessment round (Inspectie Verkeer en Waterstaat, 2011). Figure

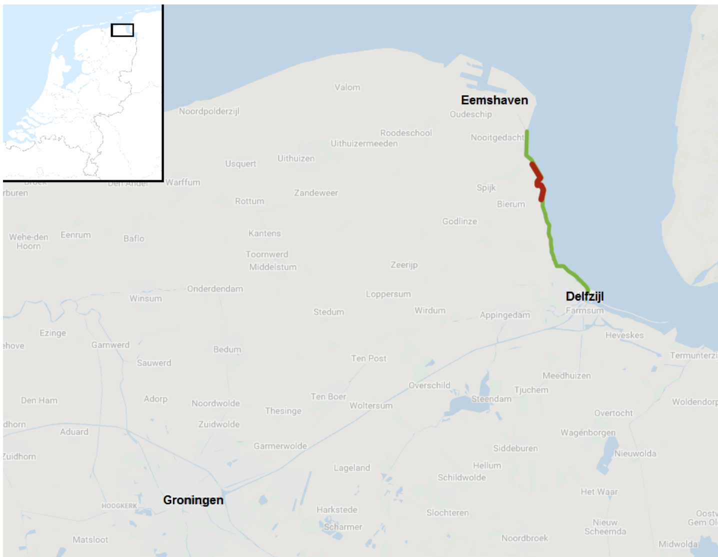


Figure 1.1: Location of the dike section that does not satisfy the required safety norms (Green) with the location of the Twin Dike project (Red)

1.1 shows the exact location of this dike stretch. The dike stretch is managed by the local water board, Noorderzijlvest. A large part of this dike stretch is being integrally reinforced the traditional way. But, on a 2.5 km stretch of this dike, the so-called Twin Dike concept is being realised (Leusink, 2018). On this stretch, the crest of the dike is too low, the grass revetment is not strong enough and the stability of the dike is insufficient (Waterschap Noorderzijlvest, 2016a). The original dike is only altered such that stability issues are resolved. Additionally, a small second dike (crest level 4 m + NAP) is constructed at a distance up to 500 m behind the existing dike. Since the original (outer) dike will keep the same crest level (approximately 8 m + NAP) and the revetment is not altered, more water will be able to come over this dike compared to a traditionally reinforced dike (Wijermars, 2017). The second (inner) dike will make sure water is retained in the basin in between the two dikes (with an approximate area of 0.5 km^2). This way the land lying behind the Twin Dike will not be flooded. The general idea behind this project is to be able to more efficiently use the area near the dike without losing valuable nature or building during dike reinforcements. At the same time, this concept is projected to be cheaper, provides the required flood safety and the area between the dikes can be used for new experimental purposes. Figure 1.2 shows a schematic overview of how the Twin Dike concept works. The area in between these two dikes will be used for new purposes.

At the moment, there are two areas being developed in between the two dikes: area B and area C (area A is an ecological impulse planned north of the Twin Dike area). Area B is planned to house saline agriculture like cockles and sea vegetables. A study shows that saline agriculture has the potential to be more profitable than the current land-use (Kwakernaak & Lenselink, 2015). This will be an experimental location where this innovation will be stimulated. Area C is planned to become a so-called silt engine. Under everyday circumstances, water will be able to enter the area via an inlet structure and deposit its sediment. During storms, the inlet structure can be closed to prevent high water levels in the basin and consequent floods. The deposition of silt will cause the land to naturally grow (elevate) over time, adding to the flood safety of the area (Kwakernaak & Lenselink, 2015). Alternatively, the silt can be excavated and used for future dike reinforcements in the area, which is predicted to be cheaper than using materials from regular sources (Waterschap Noorderzijlvest, 2016c). Also, it will reduce the amount of sediment in the water of the Ems-Dollard estuary which will have a positive ecological effect (Waterschap Noorderzijlvest & Grontmij Nederland B.V., 2016). The reduction of sediment will, however, be negligibly small, but also this area is an experimental location to assess the effects of this innovation. If this pilot project turns out to be successful it can be implemented on larger scales in the future (Kwakernaak & Lenselink, 2015). Figure 1.3 shows an overview of the different areas and their location.

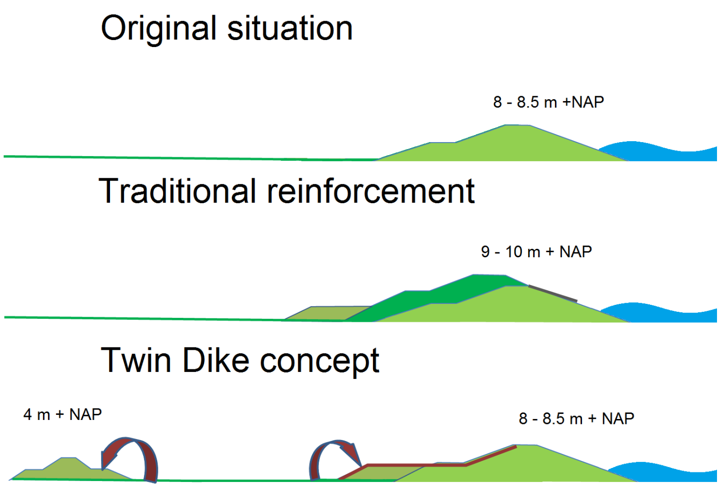


Figure 1.2: Schematisation of a traditional integral dike reinforcement and the Twin Dike concept. Modified from Waterschap Noorderzijlvest (2016c)

Summarising, the Twin Dike project is planned to be an innovative way to deal with integral dike reinforcements. Instead of increasing the width and height of a dike, this project seeks to find new ways in order to meet the required safety conditions while being able to preserve valuable areas. While doing this, it is planned to provide added value for the environment and an economic impulse to the area. Also, this concept is projected to be cheaper than a regular dike reinforcement. However, regardless of all the extra values a dike has, its primary objective will always be the protection of the land behind it against floods. Whether the Twin Dike provides the required safety is still uncertain.

1.2. Problem description

A dike can fail due to multiple causes. Water can seep under a dike undermining its foundation, the weight of the water inside the dike body can cause landslides, high waves can erode the outer slope of the dike or they can splash over the top of the dike eroding the inner slope.

The above-mentioned examples are just a small selection of many possible failures of a dike. Traditionally these problems are tackled by increasing the crest height or width of the dike. But constructing multiple lines of dike bodies behind each other can also guarantee the required safety. This concept was a possible alternative for the Hondsbossche and Pettemer sea defence reinforcement (ComCoast, 2006; ter Horst, 2005) and at the Perkpolder (ter Horst, 2005) in the Netherlands. Currently, this concept is being realised in the north of the Netherlands at the Twin Dike project (Leusink, 2018).

The Twin Dike project tackles the problem of too much water coming over the dike. When the combination of waves and water levels are high enough they can cause water to come over the crest of the dike, this is called overtopping. Traditionally, the crest of the original dike will be elevated if the overtopping volumes are too high. At the Twin Dike project, however, more water can overtop the original dike. But instead of causing a flood, the water is retained in a basin in between two dikes. Another way to look at it is to see the outer dike as a way to reduce the loads on the inner dike. The outer dike is not able to fulfil the safety requirements by itself. But it is strong enough to reduce the loads such that the inner dike will not breach. Both dikes together provide the required safety.

In order for this concept to be an economically attractive alternative to traditional integral dike reinforcements, the inner dike needs to be small compared to the outer dike. If the inner dike needs to be too strong or too high, it is possibly cheaper to reinforce the original dike the traditional way. The above means that a breach may most likely not form in the outer dike. A breach will cause a lot of water to enter the basin. If the inner dike needs to be able to withstand these amounts of water it needs to be strong and probably at least have the same height as the outer dike (because the water level in the basin will level with the sea water level). On the other hand, it is only useful to construct an inner dike if

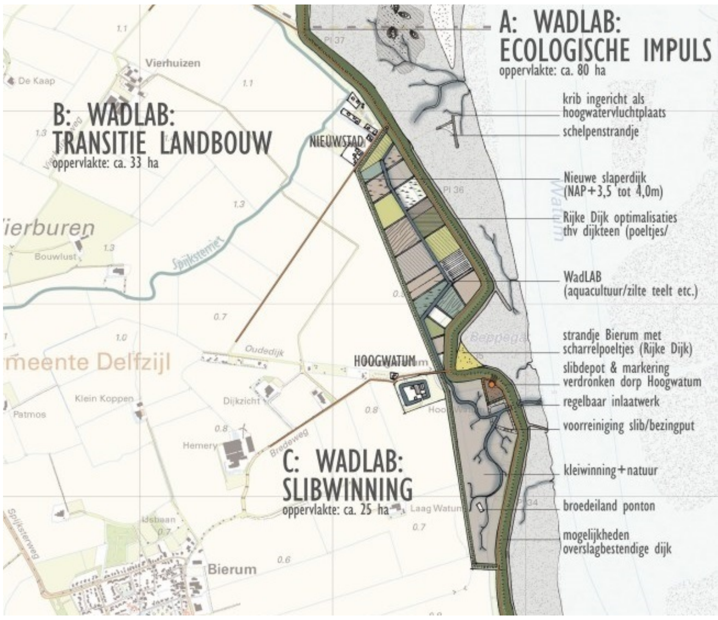


Figure 1.3: Overview of the Twin Dike project (Wageningen University & Research, n.d.)

the amount of water coming over the outer dike will lead to a flood in the first place. Because, if this is not the case, and the overtopping volumes will not lead to a flood, the construction of an inner dike is not necessary. The amount of water which is needed to cause a flood differs per area. It is dependent on the amount of water covering a specific area and the consequential societal disruption, casualties and economic damages. This is again dependent on the hydraulic conditions on the water, the safety level of the flood defence, landscape and land use.

Currently, there are no predefined methods to assess the safety of flood defences which consist of multiple lines of dikes. Also, at the Twin Dike project uncertainty exists in how the loads on the outer dike affect the hydraulic loads in the basin and the consequent loads on the inner dike. This means additional research is preferred on the flood safety benefits of this project.

The paragraphs above have led to the following problem statement:

It is unknown whether the concept of multiple dikes behind each other fits in the Dutch policy of flood protection regarding flood safety and costs. More information is preferred on what the governing loads are and if these loads will indeed lead to a flood which can best be counteracted by the construction of multiple lines of dikes.

1.3. Objective

The objective of this research is to find an answer to the following research question:

Looking at the Twin Dike project as a case, is the concept of multiple dikes behind each other an attractive alternative for dike reinforcements, regarding flood safety and costs, to protect the hinterland against floods?

In order to find an answer to the research question, the following sub-questions are formulated:

- How are Dutch flood defences assessed?
- What are hydraulic boundary conditions (water levels, waves and wind) that occur in the Twin Dike project region?
- What are governing failure mechanisms for the Twin Dike project?

- How does the outer dike influence the loads on the inner dike and what are these loads on the inner dike?
- What is the effect of a secondary high water wave on the safety of the Twin Dike project?
- Does the Twin Dike project provide the required safety?
- Looking with a cost-benefit perspective, is the Twin Dike project an attractive alternative for traditional dike reinforcements?
- What possible alternatives are there for the Twin Dike project?
- How is the applicability of the Twin Dike concept affected by other locations and boundary conditions?

1.4. Method

The research will start with an explanation of how flood defences are currently assessed in the Netherlands. Different software programs which are used to assess the safety of Dutch flood defences are elaborated on. It will be explained how a probabilistic assessment program like Hydra-NL currently determines hydraulic boundary conditions and how these are then used to assess flood defences.

Next, the system of the Twin Dike will be analysed. It will be analysed what failure paths are governing and what the relevant failure mechanisms are. These relevant failure mechanisms will be assessed using existing software.

Since the crest of the original dike will not be elevated, and the revetment on the outer slope will not be pulled up, it is expected that erosion of the outer slope and the consequent overtopping are a governing mechanism. Experimental models and methods will be used to elaborate on the effect of these failure mechanisms on the loads on the inner dike.

Parameters with different uncertainties are used in the different models. To assess the impact of the uncertainties on the results a sensitivity analysis is performed. Here, the most important parameters are fluctuated and the effect on the results are analysed. Also, the development of a storm over time is interesting for erosion and overtopping occurring at the outer dike. The current models are only able to predict the course of a storm in a coarse manner. A large dataset is available which will be used to better understand the storm development over time in this region. It will be analysed how the results differ from the results obtained from the current models.

Additionally, a cost-benefit analysis is performed for this project. It is described what the costs of the dike reinforcement are and these are compared with the regular integral reinforcements conducted north and south of this project area. Using this information possible alternatives to the Twin Dike project are analysed.

Finally, it is concluded if multiple lines of dikes are indeed an attractive alternative for traditional dike reinforcements. The conclusion gives statements on the safety and costs of the entire Twin Dike project. Is the Twin Dike project indeed a good alternative, regarding safety and costs, to protect the land behind it against floods? Are other solutions maybe more preferable? Can there be other situations where multiple lines of dikes are an attractive alternative?

1.5. Outline

The first part of this report will explain the current flood protection concept in the Netherlands. This will be described in Chapter 2. Chapter 3 will describe the case study that is going to be used as an example of flood protection using multiple lines of dikes: The Twin Dike project. Chapter 4 will try to assess this project using existing assessment instruments. From this chapter it will follow that erosion of the outer slope is a governing and more complicated failure mechanism. This failure mechanism will be elaborated in more detail in Chapter 5. In Chapter 6 a sensitivity analysis will be performed to assess the influence of different variables on the results. Additionally, a cost-benefit analysis is performed in

Chapter 7 in order to analyse the costs of the Twin Dike project and to compare several alternatives. Finally, the conclusions, discussion and recommendations are presented in Chapter 8.

2 Flood Protection in the Netherlands

This chapter gives an overview of the flood protection concept in the Netherlands. First, an overview is given on historical events which shaped the flood protection concept we use in the Netherlands today. Next, the different kind of categories of flood defences and the stated safety norms are discussed. Finally, the assessment criteria and their background are explained.

2.1. History

Already since millennia, people in the Netherlands have had to protect themselves against the dangers of water. Before the beginning of the Christian era, people started elevating their land to protect themselves and their assets against the dangers of flooding. These man-made hills, called mounds, made sure everything on it would stay dry during a flood. What started with individual houses and farms being built on mounds, resulted in entirely elevated villages in the centuries to come.

Another way people protected themselves against the dangers of flooding was the construction of earthen embankments around the land they did not want to flood. Already in Roman times, people constructed these small levees, or dikes, around their fields to protect their crops. The first dikes were relatively small circles build by the local inhabitants to protect their house, village or nearby fields. In the centuries to come, the network got more vast and complex until around the year 1300 a big part of flood-prone Netherlands was contiguously surrounded by dikes (Rijksdienst voor het Cultureel Erfgoed, 2018).

Dikes were however not only used to protect (low lying) land against the possibility of flooding. Around the year 1500 people started using dikes in combination with pumps to reclaim land which was previously not eligible for living. Dikes made sure water was not able to flow onto the land which was encircled by the dikes. At the same time, windmills made sure groundwater infiltration and rainfall were pumped out. These so-called polders were created all over the Netherlands.

Historic Floods Unfortunately, the construction of dikes and mounds did not mean these areas were not flooded any more. In the past, loads acting on a dike were not determined by sophisticated probabilistic methods, as they are today. The height of a dike was simply determined by setting it equal to the highest known water level, sometimes including some safety factor: the highest water level known plus one meter. This method, unfortunately, meant that occasionally a big flood occurred. The list below summarises the major floods in the Netherlands in the past $\pm$ 600 years (Watersnoodmuseum, 2018).

- The St. Elizabeth's floods on 19 November 1404 and 1421 caused a lot of damage in the provinces of Zeeland and Holland. Thousands of people lost their lives during these floods.
- The All Saints' Flood on 1 November 1570 caused major floods all along the coast from Flanders, Belgium, all the way to Germany. Multiple villages submerged under a thick layer of silt deposited by the flood. More than 20,000 people lost their lives and ten thousand more got homeless.

- The Christmas Flood on Christmas night 1717 caused floods along the coast from the Netherlands all the way up to Scandinavia. Cities like Amsterdam, Groningen and even Arnhem were flooded. Around 14,000 people lost their lives. This would be the last flood of this magnitude in the province of North-Holland.

- The Zuiderzee flood on 14 January 1916 was not as severe as the floods discussed before. Villages around the Zuiderzee were flooded due to a severe storm with high wind speeds. 16 people lost their lives. This disaster was the main reason for the implementation of the Zuiderzee works to be accelerated. The plan was to construct a 33 km dam at the mouth of the Zuiderzee. This way, a lake was formed that did not have a connection to the sea. Storms in the former Zuiderzee are now less severe, protecting the villages around the former sea. Also, a large freshwater basin was created and parts of the lake were reclaimed and used for agriculture. The dam was finished in 1933.

Figure 2.1: The Zuiderzee Works: The Afsluitdijk and multiple polders (Almere Tours, n.d.)

- The North Sea flood of 1953 is the last big flood in the Netherlands. Dikes in Zeeland and South-Holland breached in the middle of the night during a strong north-western storm. Almost 2,000 people lost their lives and around 100,000 got homeless. Immediately after the disaster, the Delta Committee was created. The committee assisted the minister in making decisions on what measures were necessary to prevent future floods. In the next year, the committee proposed one of the most revolutionary hydraulic engineering projects in the world: The Delta Plan. The Delta Plan consisted of the construction of the Delta Works, a large shortening of the coastline by the constructions of dams and barriers where estuaries were closed off with the sea. By shortening the coastline, protecting the land was made easier.

- In 1993 and 1995 river water levels in the Rhine and Meuse rose to extreme levels. While no dikes breached, about 250.000 people were evacuated in 1995. These events sped up projects like Room for the River, where rivers are given more space in order to prevent future floods.

Since 1996 all the primary Dutch flood defences are periodically assessed to see if required safety conditions are still satisfied. Until now there have been three assessment rounds in which all the primary Dutch flood defences have been assessed. The first in 1996-2001, the second in 2001-2006 and the third in 2006-2011. Around a third of the primary flood defences of the Netherlands did not meet the required safety standards in the third round (Inspectie Leefomgeving en Transport, 2013), which was assessed according to the old safety standards (see Section 2.3). The results of the third assessment round are the start of the Flood Protection Programme.

Figure 2.2: Overview of the Delta Works in the south western part of the Netherlands (Stichting Deltawerken Online, 2018)

The Flood Protection Programme (FPP) is a programme where water boards and the government work together to bundle strength and knowledge. All knowledge regarding the aspects of improving primary flood defences is shared. The goal is to have all the flood defences that were disapproved in the third assessment round up to date and functioning properly again by 2050 according to the new safety standards. It is reviewed every year and set up for a period of 6 years.

The dike stretch in between Eemshaven and Delfzijl did not meet the safety standards in the third assessment round. The goal to have this dike section safe according to the safety standards again is part of the Flood Protection Programme.

2.2. Classifying flood defences

Not all flood defences are the same. They can differ in type and in importance. The different flood defences are discussed in this section.

2.2.1. Classification by type

So far the definition of flood defences and dikes have been used interchangeably. A dike is a kind of flood defence. There are many other kinds of flood defences. The list below gives an overview.

- **Dike** A dike is a human-made water retaining structure consisting of soil, which lies in between water and land and is constructed with sufficient elevation and strength to protect the land behind it against high water and waves.

- **Dam** A dam is basically the same as a dike. The difference is that a dam separates two water bodies. A dam is not always constructed out of soil. It can also be made out of concrete caissons or rip-rap.

- **Dune** A dune is a sand ridge along the coast, generally formed by natural processes. Its vast volume of sand prevents water from coming through.

- **Storm surge barrier** A storm surge barrier is a movable barrier which can temporarily close of a river or estuary. Under normal conditions, the barrier is opened and water can flow through. During a storm, the barrier is closed to prevent high water levels in the estuary or river.

- **Other hydraulic structures** Other structures can be part of a flood defence as well. For example, a quay wall in a port or a sluice at the end of a canal. But also culverts, pumping stations and cuts can be part of flood defences.

2.2.2. Classification by importance

Not all flood defences are equally important. Some flood defences protect the land from the vast North Sea while others protect the land from a minor canal. Flood defences in the Netherlands can be divided into three groups when looking at their importance regarding the protection of the land against floods. The three groups are the primary flood defences, the regional flood defences and other flood defences.

Primary flood defences protect the land against floods from main external sources of water. These main sources are the sea, the Rhine and Meuse and their branches, the IJsselmeer, the lakes around Flevoland and the lakes and estuaries in Zeeland (Rijkswaterstaat, 2018a). Primary flood defences protect areas where possible floods can have a lot of casualties and major economic consequences. As of 2017, the Netherlands has around 3,450 km of primary flood defence (Waterveiligheidsportaal, 2018). Figure 2.3 shows all the primary flood defences. The national government determines the safety standards of the primary flood defences.

Figure 2.3: Primary flood defences in the Netherlands (in black). Modified from Slootjes and van der Most (2016)

In 2017, the safety principle in the Netherlands has been modified (see Section 2.3). Before 2017 the primary flood defences were further categorised in category A, B and C defences.

- *Category A. These were the primary flood defences protecting the land against the main water bodies.*
- *Category B. These were defences that separate the main water bodies from other water bodies.*
- *Category C. These defences were also primary flood defences but did not lie next to main water bodies. These were additional defences located behind a category A or B defence.*

Category B defences were also so-called front barriers. A breach in one of these barriers does not necessarily lead to a flood. These flood defences separate water from water. As long as the rear barriers still function this will not lead to a flood. In a way, the Twin Dike project can be seen as a system *of category B and C defences. The outer dike can be seen as a category B defence. It reduces the load on the inner dike (category C). The categorisation of primary flood defences in category A, B and C defences does not exist any more.*

Regional flood defences lay next to canals and small lakes. A breach in a regional flood defence can have large consequences but usually has a smaller impact than a breach of a primary flood defence. Also, defences that are compartmenting land behind primary flood defences are sometimes classified as regional flood defences. The total length of all the regional flood defences in the Netherlands is around 14,000 km (Rijkswaterstaat, 2018b). The local province determines the safety standards for regional flood defences.

Other flood defences are defences with safety standards determined by the local water board. These are not regulated by national or provincial law. They can be small defences next to a small trench, canal or pond or an old dike that is still present in the landscape. Also, compartmental barriers or summer dikes next to rivers are sometimes classified as other flood defences.

2.2.3. Compartmental barriers

Another kind of flood defence is the compartmental barrier. Compartmental barriers are mostly classified as regional flood defences according to the classifications above, but can also be a primary or other flood defence. Compartmental barriers are different from usual barriers: in normal circumstances, they do not lie next to water bodies. Because of this difference, this type of barrier is discussed in more detail.

A compartment barrier is a barrier or dike which is used to withhold floods when a primary flood defence has breached. A compartmental barrier is used to reduce the consequences of a flood. This can be done in several ways. The compartmental barrier can, for example, prevent floods in an area altogether. If a primary flood defence has breached the compartmental barrier can prevent floods in areas behind the compartmental barrier, see Figure 2.4. It can also function as a temporary barrier. This way, it does not prevent flooding of the area behind it altogether, but it can buy some extra time for evacuation. Finally, it can also be used to divert floods. A compartmental barrier can then divert water to an economically less important area instead of having an important city flooded. A compartmental barrier can be an old dike that still exists in the landscape. But also currently used canal dikes, levees of railways and highways and even sound walls can function as compartmental barriers.

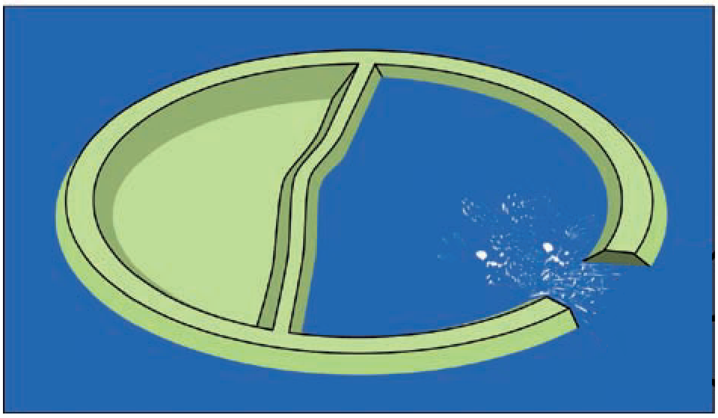


Figure 2.4: Example of how a dike breach in a dike ring is compartmentalised by an compartmental barrier. Modified from Geerse et al. (2007)

In the first instance, these barriers might sound like an attractive solution to minimize damage and casualties after a breach of a primary flood defence has occurred. However, it can also have negative consequences. A compartmental barrier will result in a smaller area being flooded after a breach in a primary flood defence. The water depths in this area will, however, be larger and will rise quicker than in the case with no compartmental barrier. The area is smaller so the water entering the area from the breach will have to spread over a smaller area. If the compartmental barrier prevents a flood in an adjacent compartment, the prevented damages and casualties have to be weighed with the additional damages and casualties in the flooded compartment. This, however, assumes a compartmental barrier will never breach. It can also be the case that after the compartment has been flooded the compartmental barrier breaches. This can then cause additional damages and casualties in the adjacent compartment while the current compartment also suffered additional damages and casualties. This example illustrates that a compartmental barrier is not always a good solution to deal with the possible consequences of a flood, its effects have to be carefully investigated.

Geerse et al. (2007) developed a guideline on how to assess a compartmental barrier and its effects. Different scenarios with compartmental barriers have to be assessed including one case where there is no compartmental barrier. Then the effects of a breach have to be analysed and a cost-benefit analysis has to be performed for all the cases. This way the consequences of a flood with different scenarios of compartmental barriers and no barrier at all are known. Only then a good conclusion can be made on the effect of a compartmental barrier.

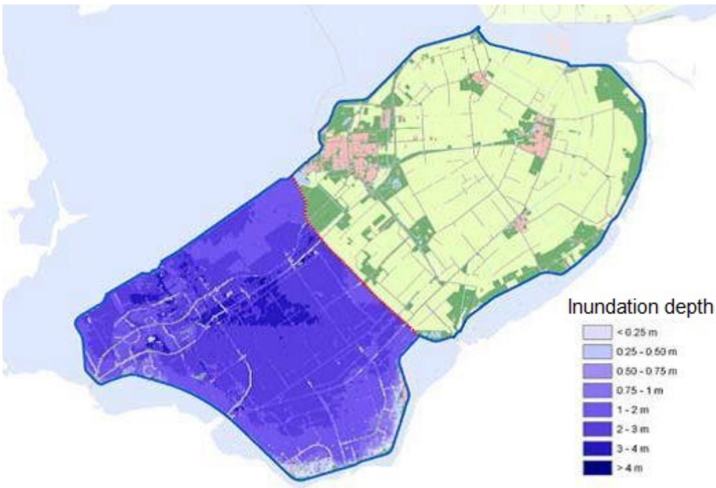


Figure 2.5: Simulation of a flood compartmentalised by an compartmental barrier (Geerse et al., 2007)

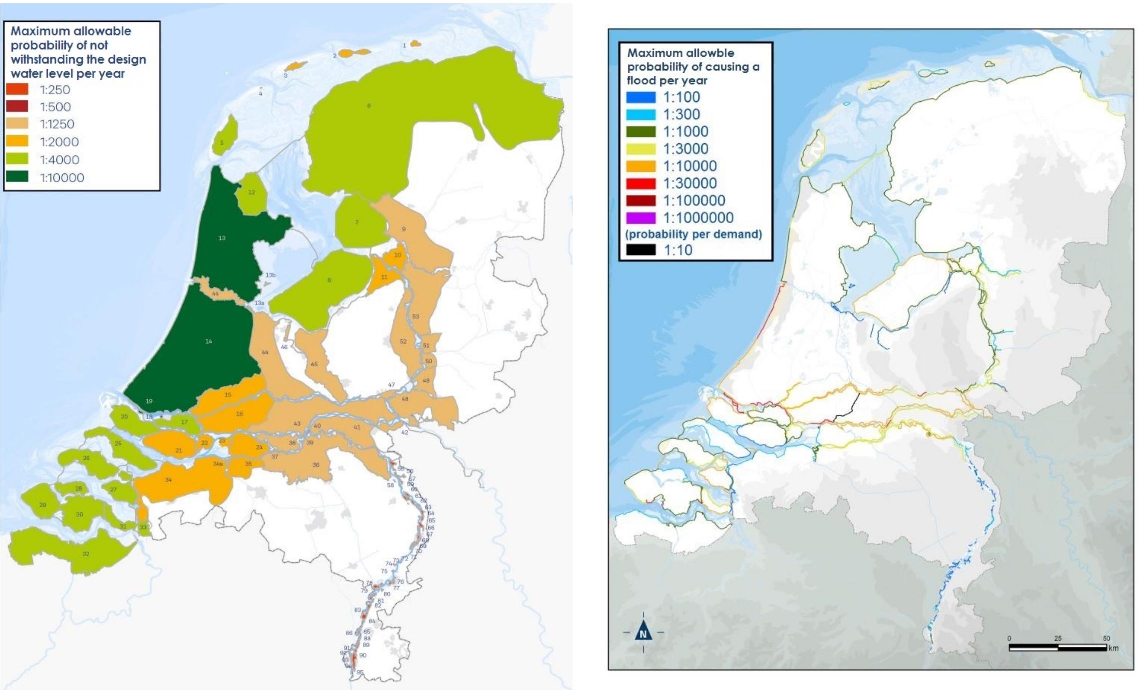
Figure 2.5 shows maximum inundations depths after a breach in a dike in southern Flevoland, the Netherlands (Geerse et al., 2007). In the figure, it can be seen that the compartmental barrier (which lies approximately in the middle) prevents flooding of the upper half of the area. The study shows that in a case where this compartmental barrier is not present, damages and casualties are be around twice as high. This compartmental barrier has a positive effect.

The Twin Dike project is similar to these compartmental barriers, only on a much smaller scale. The area between the outer and inner dike can theoretically be seen as a compartment. The inner dike is a compartmental barrier that prevents floods in the compartment laying behind it.

2.3. Safety norms of flood defences

As mentioned in Section 2.1, in the past required dike elevation was determined by using the highest recorded water level. This approach changed after the flood of 1953. Already in 1939, Wemelsfelder (1939) proposed an approach in which return periods of extreme water levels were used. In 1960 the Delta Committee proposed safety norms where dike rings next to sea had to withstand water levels which occurred with a certain average return period (Bötger & Te Linde, 2014). Dike rings which had a high value regarding inhabitants and economic importance were given high protection levels. Areas that had less value were given a lower protection level. For example, Dike ring 14, which contains Rotterdam and The Hague, had to be able to withstand water levels with a probability of exceedance of 1/10,000 per year. While dike ring 5 on the island of Texel only had to be able to withstand water levels with a probability of exceedance of 1/4,000 per year. This system got expanded to the river dike rings in the 1970s.

As of 1 January 2017, these safety norms have been altered (Ministerie van Infrastructuur en Milieu, n.d.). Instead of looking at the probability of exceedance of a certain water level, the focus shifted to the consequences. Flood defences are given a maximum allowable probability of causing a flood: the probability per year of not fulfilling the water retaining function may not be larger than this value. Flood defences are grouped in so-called dike trajectories which have an average length of 15 km. Per dike trajectory loads are similar and a possible breach has similar consequences (Kok, Jongejan, Nieuwjaar, & Tanczos, 2017). Every individual in the Netherlands has a maximum risk to be exposed to a flood, which was not the case before. Then, depending on the societal and economic impact of a breach and flooding, a safety level (maximum allowable probability of failure) is assigned to a dike trajectory. This way, trajectories in the same ring can have varying safety norms. An overview of the old and new safety norms for the Netherlands can be seen in Figure 2.6. In the new safety norms, dike trajectories also have a signal value next to the maximum allowable probability of causing a flood. The signal value is also a probability per year. This value is more strict than the maximum allowable failure probability. When the safety levels of a flood defence reaches below this value, the responsible minister has to be informed. When this value is reached there is enough time to take action, such as a dike reinforcement project, before the maximum allowable probability of failure is reached.



(a) Old safety norms (Kok et al., 2017)

(b) Safety norms as of 2017 (Slootjes & van der Most, 2016)

Figure 2.6: Flood safety norms in the Netherlands

The dike stretch in between Eemshaven and Delfzijl did not meet the safety standards in the third assessment round. During the third assessment round, flood defences were assessed using the old safety norms. The Twin Dike location was thus not able to withstand conditions with a return period of 4,000 years. Also with the new safety norms, the Twin Dike area does not satisfy the norms: the probability of flooding is higher than 1/3,000 per year.

2.4. Predicting the consequences of a flood

The required safety norms of flood defences are for a large part dependent on the consequences of a flood. The predicted consequences of a flood can be defined in roughly three factors. Namely economic damages, the loss of life and other consequences.

Economic damages Economic damages are usually calculated by splitting the area of interest into different land use categories. These categories each have their own value per surface area. High rise buildings are more valuable than low rise buildings and high-tech industry has more economic value than agricultural areas. Each of these land use categories has a corresponding damage curve. The damage curve shows which fraction of the total value of each land use category is lost for certain water depths. Figure 2.7 shows an example of an area divided into land use categories. Figure 2.8 shows an example of damage functions for different land use categories. It can be seen that, in this example, agricultural areas lose 90% of their value at an inundation depth of 2 metres. While a seaport only loses 25% of its value. This does not mean that with a flood depth of 2 metres there is more damage in agricultural areas than at seaports. A seaport might have a higher value per surface area so that the total damage per surface area can be more at the seaport than at the agricultural areas.

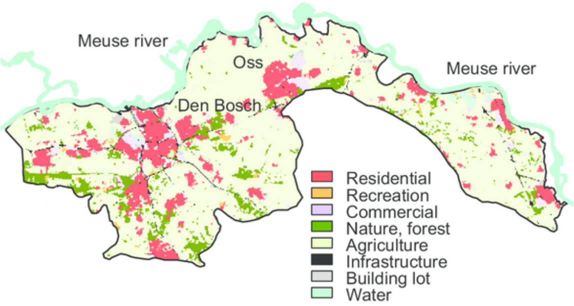


Figure 2.7: Area split up in land use categories next to the Meuse. Modified from Bouwer et al. (2009)

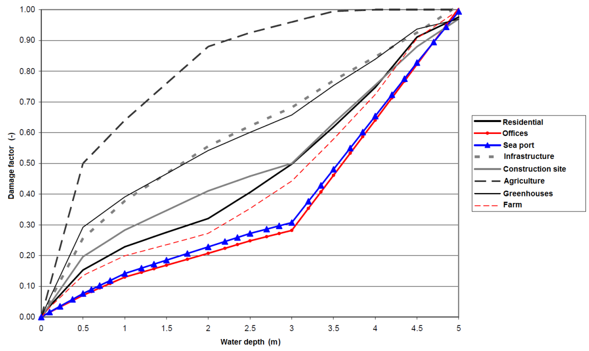


Figure 2.8: Example of damage functions. Modified from Klijn et al. (2007)

If the flood depths occurring at a certain location are known, these can be linked to the corresponding land use type and damage function resulting in the damage on that location. Of course, also other factors can be incorporated into the damage functions, like flow velocity and flood duration.

When dike trajectories were assigned a safety norm, economic damages were taken into account. Areas where a flood leads to large economic damages, are better protected than areas with little economic value. Usually, a cost-benefit analysis is performed in order to find out which protection level is most favourable.

Loss of life To determine the predicted loss of life again the occurring water depth is important. However, more important is the velocity of rising of the water level. If water levels rise slowly, people will be able to find shelter easier than if water rises quickly. Also, evacuation plays a role. In some areas of the Netherlands, a large fraction of the population might be evacuated before a flood occurs. The mortality rate can be estimated with Equation 2.1 (S. Jonkman et al., 2017). Figure 2.9 shows an example of mortality fractions dependent on the rising rate of the water and the inundation depth. The quicker the rising rate, the higher the mortality fraction.

$$N_{loss} = F_d \cdot (1 - F_E) \cdot N_{PAR} \tag{2.1}$$

In which:

N_{loss}	=	estimated loss of life
F_d	=	mortality fraction
F_E	=	evacuation fraction
N_{PAR}	=	number of people at risk

Evacuation fractions differ greatly per location in the Netherlands. Figure 2.10 shows an example of evacuation fractions in the Netherlands. It can be seen that evacuation fractions near the rivers in the hinterland are a lot higher than near the coast. This has multiple reasons. The first is that high waters from rivers are usually known days earlier than high waters from the sea. High water in rivers is caused by water entering the river upstream. It can take days before this water causes problems in the Netherlands. Also, population density, available infrastructure and distance from safe grounds play a

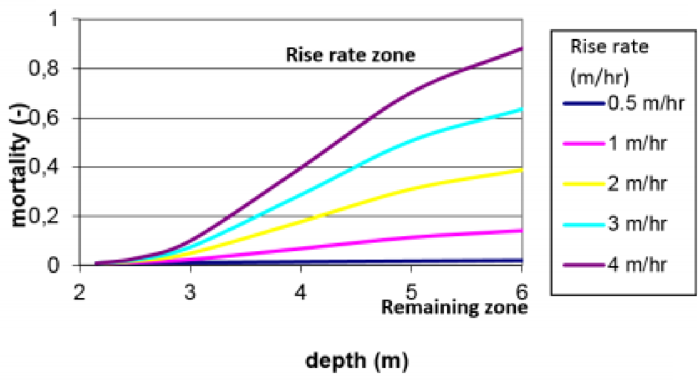


Figure 2.9: Example of mortality fractions (S. Jonkman et al., 2017)

role in determining the evacuation fractions.

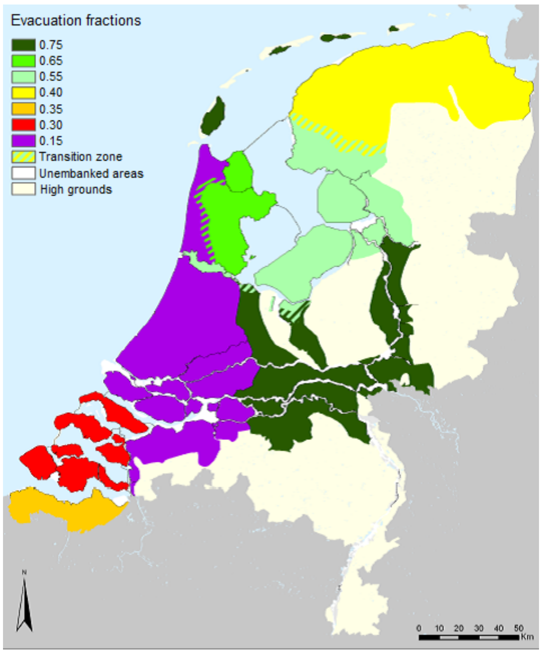


Figure 2.10: Evacuation fractions in the Netherlands (S. Jonkman et al., 2017)

Also, the possible life losses are taken into account when assigning a safety level to a dike trajectory. The law states that each individual in the Netherlands has a minimal safety against becoming a casualty due to a flood. The chance of deceasing due to a flood is not allowed to be higher than 1/100,000 per year for each individual. Also, the risk of large groups becoming a casualty all at once is taken into account. Multiple events with little casualties have a different impact than one event with many casualties. An event in which large amounts of people decease can cause national insecurity and unrest. Areas where large groups of people are in danger are better protected.

Other Besides the loss of life and economic damages also other consequences are possible. For example, damage to the environment or damage to objects that cause large societal disruption. The safety norms make sure that areas, where a flood can lead to large societal disruptions because of the destruction of an important object, are better protected. An example is the nuclear power plant of Borssele.

2.5. Failure of flood defences

Unfortunately, the construction of flood defences does not mean an area is completely safe. There is always a (small) chance that a flood defence will fail and cause a flood. There is however a difference in the failure of a flood defence and a flood itself. Also, a failure and a flood can be interpreted in different ways by different people and institutions.

2.5.1. Failure and flood

Failure of a flood defence is defined as a flood defence not being able to fulfil its water retaining function any more. With dikes, this usually means a breach has formed. A breach is a situation where part of the dike breaks away, creating an opening where water can enter the land lying behind the dike. A failure of a flood defence *can* lead to a flood.

A flood is generally defined as an event in which considerable amounts of water cover land that is usually not covered by water. This water can originate from the sea, rivers and lakes but also excess rainfall and increasing groundwater levels can cause floods. A flood can thus be differently interpreted depending on the interpretation of 'considerable amounts of water'. The Dutch law defines a flood as 'the loss of water retaining function within a dike trajectory causing the area protected by the dike trajectory to flood in such a way that fatalities or substantial economic damage occur' (*Waterwet*, 2009). The definition of fatalities is very clear. But the definition of 'substantial' economic damage can again be differently interpreted per situation. Per situation is has to be assessed what the possible economic damages and its impact are, in order to properly determine if an event is classified as a flood. According to Kok et al. (2017), an average water depth of less than 20 cm in an area with the same zip code number does not classify as a flood. Because only from these water depths casualties and large economic damages start to occur. This paragraph shows that the definition of a flood is highly subject to interpretation. A local farmer might think of a flood differently than a water board, province or the national government. The safety norms discussed in Section 2.3 state the maximum allowable probability that a failure of the flood defence results in a flood. But, failure of a flood defence does not immediately imply a flood. If the amounts of water coming through a breach cause negligible damage, no flood has occurred.

Failure of a flood defence can also be interpreted in different ways. As described above, failure of a flood defence means the water retaining function is not fulfilled any more. In the Netherlands, flood defences are assessed using the Statutory Assessment Instruments (WBI) (Ministerie van Infrastructuur en Milieu, n.d.). New flood defences or reinforcements of existing flood defences are designed using the Design Instruments (OI) (Rijkswaterstaat, 2017). These instruments state the rules and methods that need to be followed when assessing or designing a primary flood defence. The WBI defines a broad range of failure mechanisms which need to be taken into account when assessing a primary flood defence on its safety. The definition of failure is however quite conservative. When a flood defence does not satisfy the required safety norms according to the WBI, usually, some residual strength is still left before a breach is formed. The failure mechanisms, as used by the WBI are described in the next section.

2.5.2. Failure mechanisms (WBI)

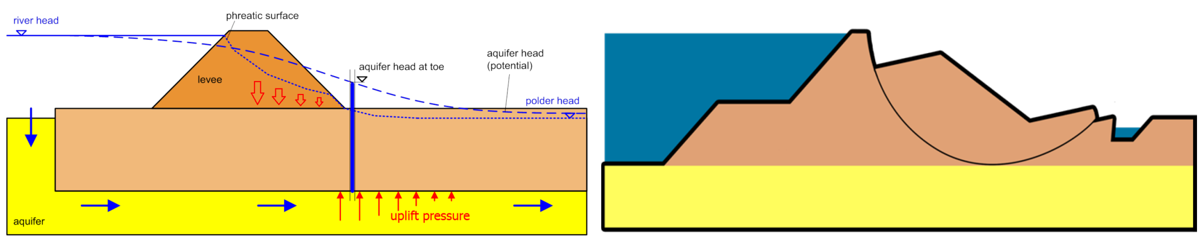
The different failure mechanisms as used by the WBI will be discussed in this section. All the failure mechanisms are listed in Table 2.1 and are discussed in more detail below.

Macro-instability of the inner slope (STBI) Macro-instability of the inner slope (also called sliding of the inner slope) can occur when water levels at the water side of the dike rise. Water infiltrates into the dike and causes saturation of the dike. Water pressure in the dike leads to a decreased shear strength of the soil. The extra weight of the dike exerts a downward force while uplift forces behind the dike exert an upward force. See Figure 2.11a. Eventually, the crest of the dike can slide down while the toe of the dike gets pushed up. A dike has failed according to this failure mechanism when a slide occurs. See Figure 2.11b for an illustration of the failure mechanism.

Failure mechanism	Code
Dikes and dams	
Macro-instability of the inner slope	STBI
Macro-instability of the outer slope	STBU
Piping	STPH
Micro-instability	STMI
Revetment	
Wave impact on asphalt	AGK
Overpressure of water under asphalt	AWO
Grass cover erosion outer slope	GEBU
Grass cover sliding outer slope	GABU
Grass cover erosion crest and inner slope	GEKB
Grass cover sliding inner slope	GABI
Stability of the stone revetment	ZST
Dunes	
Dune erosion	DA
Structures	
Height of structure	HTKW
Reliability closure of structure	BSKW
Piping at structure	PKW
Strength and stability point structure	STKWp
Strength and stability littoral structure	STKWl
Foreshore	
Wave erosion of foreshore	VLGA
Sliding of foreshore	VLAF
Liquefaction of eshore	VLZV
Non-water retaining objects	
Buildings	NWObe
Vegetation	NWObo
Cables and pipes	NWOkl
Other structures	NWOoc
Jetties	HAV
Technical innovations	INN

Table 2.1: Failure mechanisms as defined by the WBI with their corresponding codes (Ministerie van Infrastructuur en Milieu, 2017b)

Macro-instability of the outer slope (STBU) Sliding of the outer slope is a risk when water levels decline rapidly. Then, pore water in the dike does not have enough time to leave the dike body. The water pressure causes decreased shear strength of the soil and without water pressing on the dike on the outside, the soil can become unstable leading to a slide. This is the same as sliding of the inner slope, only on the other side of the dike. A dike has failed according to this failure mechanism when a slide occurs. Figure 2.12 shows an illustration of the failure mechanism.



(a) Forces (in red) contributing to the sliding of the inner slope. Modified from S. Jonkman et al. (2017)

(b) Schematisation of a slide of the inner slope (S. Jonkman et al., 2017)

Figure 2.11: Schematisation of failure due to sliding of the inner slope

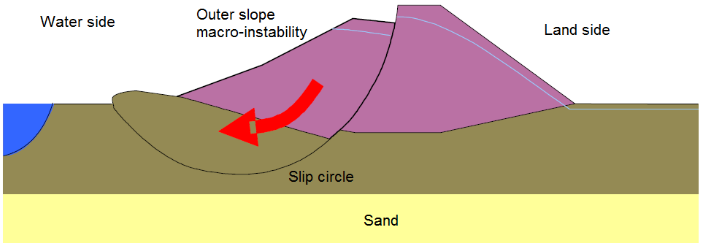


Figure 2.12: Schematisation of failure due to sliding of the outer slope ('t Hart, 2018)

Piping (STPH) When the hydraulic gradient between the water and land sides of a dike is high, water can be pushed out of the soil at the land side of the dike. Next, groundwater can start flowing from the water side to the place where water is pushed out of the soil on the land side. Eventually, pipes can form underneath the dike. These pipes can undermine the foundation of dikes. In order for piping to occur, three preceding conditions have to be satisfied.

- Uplift: Water from the water body enters the sand layer underneath the dike. Behind the dike, this water exerts pressure on the clay layer. If this pressure is larger than the weight of the soil it is lifted up and can eventually rupture.
- Heave: When water starts seeping out of the aquifer behind the dike this does not immediately pose a problem. It does become a problem when individual sand grains start to be washed out because this will start erosion. This will happen if the gradient at the point where uplift occurred is larger than a certain critical gradient. The critical gradient is dependent on soil characteristics.
- Pipe formation: The head difference between the front and the rear of the dike needs to be higher than a critical head difference for pipes to form underneath the dike. The critical head difference is amongst others dependent on soil characteristics and the length between the front and the rear of the dike: the seepage length.

Only if these three conditions occur, piping can occur. A dike has failed according to this failure mechanism when uplift and heave occur. Additionally, the pipe length has to exceed a critical length. Figure 2.13 shows an illustration of the failure mechanism.

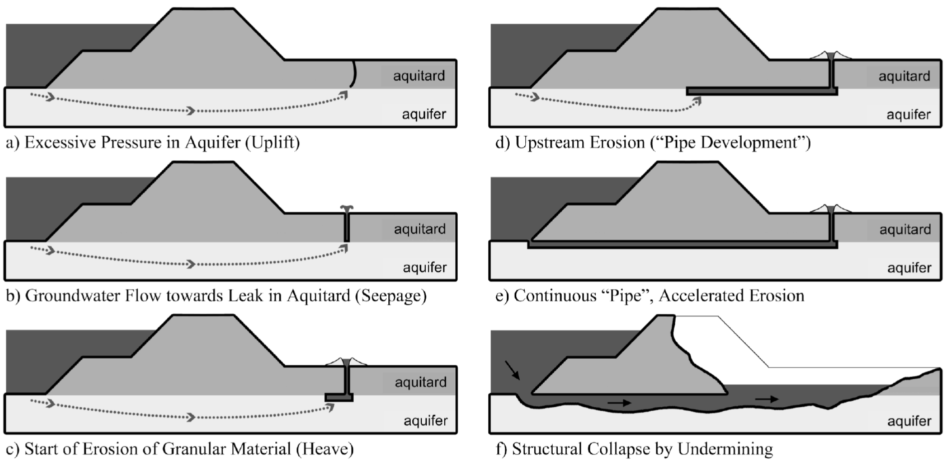


Figure 2.13: Phases of piping (S. Jonkman et al., 2017)

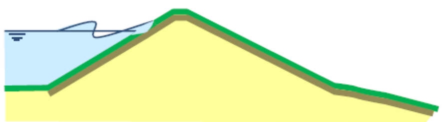
Micro-instability (STMI) Micro-instability is caused by the phreatic line in the dike body increasing to the level of the inner slope. If the layer is permeable the water pressure can cause individual grains to be pushed out and be washed away. A dike has failed according to this failure mechanism when grains on the slope start to be washed away.

Asphalt revetment (AGK and AWO) When an asphalt revetment is located on a dike, two failure modes can occur. Firstly, waves breaking on the asphalt revetment can eventually cause deformation in the asphalt revetment. This can lead to a broken asphalt cover. A dike has failed according to this failure mechanism when cracks start to occur. Secondly, when a storm with high water has passed, the asphalt revetment can be pushed up by the water still present in the dike. The revetment can be pushed up and/or slide, exposing the core of the dike. This can also happen during long loads of high waves, causing pressure differences to be high in the wave impact zone. A dike has failed according to this failure mechanism when the asphalt cover comes loose from the layers underneath.

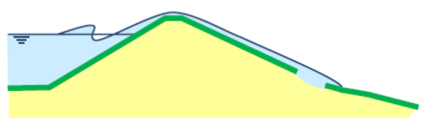
Grass revetment (GEBU, GEKB, GABU and GABI) When a grass revetment is located on a dike, four failure mechanisms can occur.

- The grass cover on the outer slope can erode due to wave impact and/or wave run-up. The forces of the impacting waves or high flow velocities of waves running-up can cause the grass revetment to erode, exposing the core of the dike. A dike has failed according to this failure mechanism when erosion reaches the layers underneath the grass cover.
- The same can happen on the crest or inner slope of the dike when the combination of water level and waves is high enough so that water reaches over the crest of the dike. A dike has failed according to this failure mechanism when the grass cover is damaged.
- During a storm with high waves the grass revetment on the outer slope can slide due to large pressure differences. Water inside the dike can push off the grass revetment after a wave has passed since during that specific moment no water is pushing on the dike from the outside. A dike has failed according to this failure mechanism when the top layer has moved with respect to the underlying layers.
- Finally, also the inner slope can slide. When water pressures inside a dike increase due to a storm, pressure can be so large that eventually the inner grass cover is pushed off. A dike has failed according to this failure mechanism when the grass cover on the inner slope starts to slide.

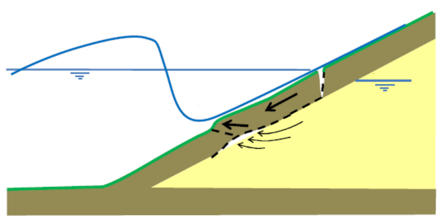
Figure 2.14 shows an illustration of these failure mechanisms.



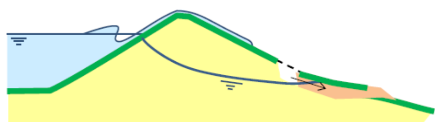
(a) Erosion of the outer slope (GEBU)



(b) Erosion of the crest or inner slope (GEKB)



(c) Sliding of the outer slope (GABU)



(d) Sliding of the inner slope (GABI)

Figure 2.14: Illustration of grass revetment failure mechanisms ('t Hart, 2018)

Stone revetment (ZST) Stone revetments are usually made of concrete and do not break easily due to wave impact. However, the same as with asphalt and grass, the revetment can be pushed off during a combination of high waves and large pore water pressures inside the core of the dike. A dike has failed according to this failure mechanism when the sub-layer is damaged.

Dunes (DA) Dunes are sand bodies which usually have little to no cover to prevent erosion. During a storm, dune erosion will almost always take place. The volume and shape of the dune determine the resistance against dune erosion. If enough sand is present, a storm is not able to breach a dune. During normal circumstances, a dune naturally recovers to its original state. A dune has failed according to this failure mechanism when dune erosion reaches the land side of the first dune body.

Structures When a structure is present other failure mechanisms have to be considered.

- A structure has to have sufficient height. When a structure is not high enough, water can flow over the structure. When this leads to an exceedance of the basin capacity behind the structure or to damage to the bed protection of the structure, a flood defence has failed according to this failure mechanism.
- A structure like a storm surge barrier has to close during severe weather conditions. There is however always a small chance that this closure fails. A flood defence has failed according to this failure mechanism when damage to the bed protection occurs or the basin capacity is exceeded.
- Piping can occur at a structure as it can occur at dikes and dams. The same criteria for a failure apply.
- Loads on a structure can be such that it causes damage or instability. A flood defence has failed according to this failure mechanism when a slide occurs, the basin capacity is exceeded or the bed protection is damaged.

Foreshore At some locations, a foreshore is located in front of a flood defence. A foreshore is usually beneficial for the safety of a flood defence. But also a foreshore can get damaged during storm conditions. Wave attack and currents can erode a foreshore or the foreshore can slide due to unfavourable load combinations, comparable as is possible at regular dikes and dams. Also, liquefaction can cause slides of the foreshore. A flood defence has failed according to this failure mechanism when a slide occurs.

Other Finally, there are some other aspects that can possibly endanger the safety of a flood defence. Objects like trees, cables and pipelines can pose a risk for a flood defence. These aspects will not be discussed in further detail in this report.

2.6. Allowable failure probability per failure mechanisms

As discussed in Section 2.3, all the dike trajectories in the Netherlands have a maximum allowable probability of causing a flood and a signal value. Waterveiligheidsportaal (2018) is an interactive website which shows signal values and maximum allowable failure probabilities per dike trajectory. For example, the dike trajectory in north-east Groningen where the Twin Dike is located has a maximum allowable probability of flooding of 1/3,000 per year. In order to design a safe enough dike, this probability of flooding has to be divided over all the relevant failure mechanisms. Each of these failure mechanisms can lead to a flood.

If a dike trajectory is designed to withstand piping, erosion and stability conditions that have a return period of 3,000 years, failures will theoretically occur with lower average return periods. Because then in those 3,000 years, the dike trajectory will on average fail once for piping, once for erosion and once for stability, resulting in 3 floods in 3,000 years while only one is allowed. This means that the total allowable probability of flooding has to be divided over all the relevant failure mechanisms. Table 2.2 shows contribution factors per failure mechanisms as used by the WBI.

In order to calculate the required maximum probability of failure due to a specific mechanism, the maximum allowable probability of flooding has to be multiplied by the contribution factor for that specific mechanisms. See Equation 2.2.

$$P_{Req,j} = \omega_j \cdot P_{Req} \tag{2.2}$$

Failure mechanism	Dunes	Dikes and dams
Height structure (HTKW) or grass cover erosion crest and inner slope (GEKB)	0	0.24
Piping (STPH)	0	0.24
Macro-stability inner slope (STBI)	0	0.04
Grass cover erosion outer slope (GEBU)	0	0.05
Other revetment outer slope	0	0.05
Reliability closure of structure (BSKW)	0	0.04
Piping at structure (PKW)	0	0.02
Strength and stability structure (STKWp)	0	0.02
Dune erosion (DA)	0.70	0
Other failure mechanisms	0.30	0.30

Table 2.2: Failure mechanisms and their corresponding contribution factor (ω) to the total failure probability (Ministerie van Infrastructuur en Milieu, 2017b)

For this particular dike section (maximum allowable probability of flood 1/3,000 per year) this leads to required protection levels for piping and erosion of the inner slope of:

$$0.24 \cdot \frac{1}{3,000} = \frac{1}{12,500} \tag{2.3}$$

From the equations above it can be seen that the allowable failure probability for erosion of the inner slope and piping is stricter than the total failure probability of the entire dike trajectory. This is due to the fact that it is unknown which failure mechanism will cause a flood beforehand.

Length-effect Not only the different contributions to the total failure probability of individual failure mechanisms are important, but also the length over which they can occur is important. The so-called length-effect causes that a design of a dike on cross-section level differs from a design on trajectory level. For every dike section, the allowable probability of failure is the same. However, if the length of the total dike trajectory increases, the probability of finding a section where the load is larger than the resistance also increases. The probability that a dike breaches somewhere along its trajectory is higher than that it breaches at a specific cross-section (Jongejan, 2016). This is the length-effect. In short, the length-effect is defined as the increase of the probability of failure with the increasing length of a dike stretch (Vrijling et al., 2011). This has multiple reasons which are elaborated in more detail below.

Both the probability of finding a higher load or lower resistance over a longer dike trajectory can cause increasing probabilities of failure. For piping, the conditions of the soil are very important. The longer the dike trajectory the higher the probability of finding an anomaly which increases the probability for piping to occur. Figure 2.15 shows the differing soil characteristics and possible standard deviation of the dike crest over a longer distance. For wave overtopping, the waves and the water levels are the important loads. Water levels will usually not differ much over a dike trajectory since these are quite uniform. But when the orientation of a dike differs a lot within a dike trajectory, overtopping volumes can also differ within this dike trajectory. A small dike trajectory facing only one angle will be affected less by this phenomenon.

In order to cope with the length-effect, the maximum allowable failure probability of a dike section has to be adjusted using the length-effect factor. This can be done using Equation 2.4.

$$P_{Req,i,j} = \frac{\omega_j}{N_J} \cdot P_{Req} \tag{2.4}$$

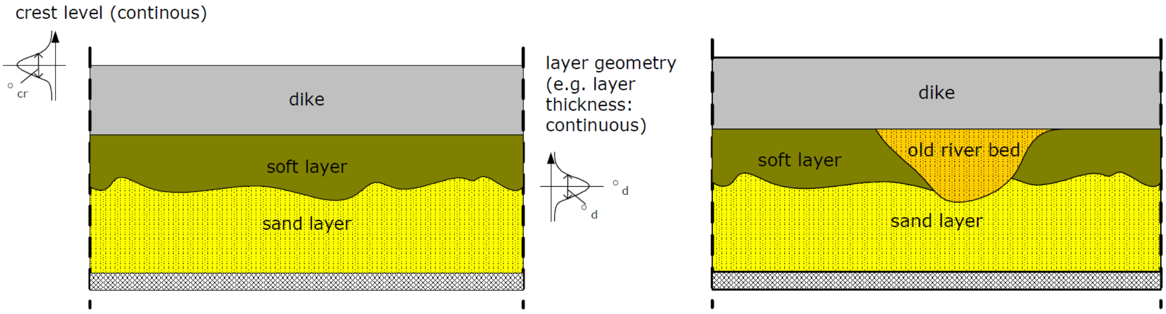


Figure 2.15: Differences in soil characteristics over a dike stretch (Vrijling et al., 2011)

In which:

P_{Req}	=	Maximum allowable probability of failure of the dike trajectory
$P_{Req,i,j}$	=	Maximum allowable probability of failure for failure mechanisms j and dike section i
N_j	=	Length-effect factor for failure mechanism j
ω_j	=	Contribution factor for failure mechanism j

The value for N differs for each failure mechanism and the specific location. Each failure mechanism, however, has a typical range for N. Over a certain length there is usually more difference in the soil, important for piping, than in the crest level, important for erosion of the inner slope. The value of N for erosion of the inner slope is therefore in the range of 1 to 3, while for piping values for N can reach up to 10. The WBI defines methods to obtain the length-effect factors for all the dike trajectories and failure mechanisms.

2.7. Reliability methods

To check if a dike satisfies the safety requirements for any of the above-mentioned failure mechanisms and for the required return period it has to be checked if the resistance is higher than the load. This can be described with the following simple limit state function.

$$Z = R - S \tag{2.5}$$

In this equation, R is the resistance and S is the load. If Z is smaller than zero, meaning the load is larger than the resistance, a failure has occurred. For failure mechanisms related to overtopping, wave height and water level are examples of load parameters. If the waves are larger or the water level is higher more overtopping will occur. Examples of resistance parameters are the crest level and angle of the waterside slope. If these resistance parameters increase, overtopping decreases. For other failure modes, the same distinction can be made. Usually, the water level is a load parameter and soil characteristics and geometrical parameters are resistance parameters. By filling in the different parameters it can be calculated if a failure occurs.

However, these load and resistance parameters usually have a probabilistic distribution. This means that they do not have a fixed value. The density of soil, water level, wave height etc. will never have the same fixed value in an area. Usually, a mean value is expected but a certain deviation around this mean will always be the case. Figure 2.16 shows what distributions of load and resistance can look like. The load and resistance both have a mean value, the most probable value of occurrence, with a spread around this mean. The figure shows that even if the mean value of load and resistance are far apart, there will always be a small chance that the load is higher than the resistance, leading to a failure. In this figure, load and resistance are represented by a normal distribution. A mean value with an equal spread left and right of it. In reality, this distribution can have all sorts of shapes.

There are four groups of methods that can be used to calculate the probability of failure. They can all be used to assess flood defences. The first in the list is the most complicated method, going down the methods get simpler.

- The first group are the so-called level 3 methods. These are the full-probabilistic methods. Using

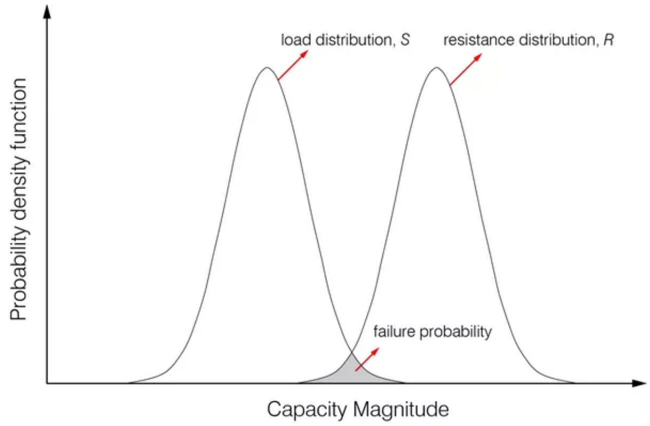


Figure 2.16: Load and resistance parameters shown with corresponding distribution (Milczarek, 2017)

these methods the failure probability is calculated exactly. This can be done by calculating the integral of the grey area in Figure 2.16 exactly. This can be done relatively easily when there is only one load and one resistance parameter. But it will get a lot harder when there are multiple. When there are many parameters a Monte Carlo analysis is a possible level 3 method. With a Monte Carlo analysis, random values are drawn for all variables from their own corresponding distribution. Using these randomly drawn values it is checked if this combination of values leads to failure. If this is repeated many times, a conclusion about the failure probability can be made.

- Secondly, there are level 2 methods. These are an approximation of the full-probabilistic methods. Here probability density functions are approximated by standard normal distributions. Also, the limit state function is linearised in the point with the highest probability density of failure: the design point. This design point is then used to say something about the reliability of a system. This method is called FORM (First Order Reliability Method).
- Next, there are level 1 methods. These are semi-probabilistic methods. Here characteristic values for strength and resistance parameters are used. Characteristic values are dependent on what the parameter is, but is often the value that has a probability of non-exceedance of 5% for resistance parameters and the value that has a probability of exceedance of 5% for load parameters. The characteristic value for load needs to be multiplied by a safety factor and the characteristic value for resistance needs to be divided by a safety factor. This leads to a design load and design resistance. The limit state function is evaluated with the design load and resistance. See Figure 2.17 for a representation of how these values are obtained.
- The final group is level 0 methods. These are deterministic calculations. A deterministic value of the variables is chosen and the limit state function is evaluated. This is a simple method with many limitations but calculations can be performed easily.

2.8. Loads

A flood defence is designed such that a flood may only occur with a certain minimum allowable return period. In order to design a flood defence, the loads corresponding to these return periods have to be known. The main forces exerting on a dike are caused by water. These loads are called hydraulic loads. The loads influence the design and assessment criteria of a flood defence. Chbab and de Waal (2017) define hydraulic loads as:

- Water level h [m + NAP]
- Significant wave height H_{m0} [m]
- Peak wave period T_p [s]
- Spectral wave period $T_{m-1,0}$ [s]
- Average wave direction [°]

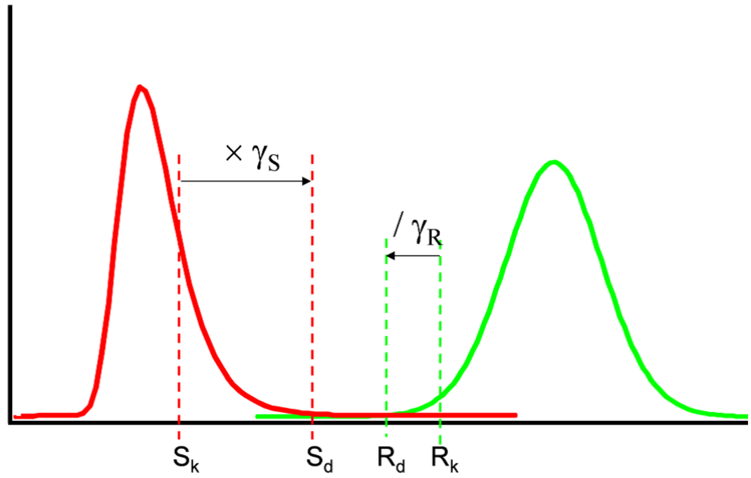


Figure 2.17: Visual representation of how safety factors lead to a design load and resistance (S. N. Jonkman et al., 2015)

The last four hydraulic loads can be grouped together and are called wave characteristics. All the hydraulic loads are dependent on a specific set of basic stochastic variables. The basic stochastic variables are defined as:

- Wind speed [m/s] and wind direction [°]
- River discharge [m³/s]
- Sea water level [m + NAP]
- Lake water level [m + NAP]
- Condition of a storm surge barrier (open or closed)

The hydraulic load is determined by a specific combination of the above-mentioned basic stochastic variables. These differ per location. In coastal areas, the river discharge and lake water level do not influence the hydraulic load. In the upstream riverine areas, sea water levels are not relevant. For this sake, the Netherlands has been split into different so-called *hydraulic region types*. Within each region type, a specific combination of basic stochastic variables is used to determine the hydraulic loads. The hydraulic region types and the stochastic variables used are depicted in Table 2.3. The area of interest for this research is the Twin Dike project area in the north-eastern part of the Netherlands. This is a coastal region with dikes. In this area, sea water levels, wind speed and wind direction are the basic stochastic variables. River discharge, lake water level and storm surge barriers do not influence the hydraulic loads in this area.

The water level is a hydraulic load which is directly related to the same basic stochastic variable. Using statistical data of measuring locations the water levels for different return periods can be estimated. In the Netherlands sometimes over 100 years of wind and water data is available. Data can be analysed and extrapolated to find return periods of water levels for larger return periods. Figure 2.18 shows how historic water levels at Hook of Holland are extrapolated to find water levels for a larger return period.

Wave characteristics are not basic stochastic variables themselves. They need to be calculated from other basic stochastic variables, namely wind direction, wind speed and the water level (river discharge). This means a relationship is needed between the basic stochastic variables (water level, wind- speed and direction) on one side and the local hydraulic loads (wave characteristics) on the other side. This relation is very complex and consumes too much time if this has to be done for every hydraulic load determination along flood defences. That is why this relationship is determined once, up front, for many different combinations of imposed conditions and is stored in a database. Next, the statistics of water level, wind speed and wind direction can be used to find wave characteristics for different return periods.

Basic stochastic variable	Hydraulic region type						
	Upper rivers	Lower rivers	IJssel- & Vechtdelta	Lakes	Coast - dikes	Eastern Scheldt	Dunes
Wind speed and direction	x	x		x	x	x	
Discharge	x	x					
Sea water level		x			x	x	x
Lake water level			x	x			
Condition storm surge barrier		x	x			x	

Table 2.3: Types of hydraulic regions and the basic stochastic variables used to determine the hydraulic loads. Modified from Chbab and de Waal (2017).

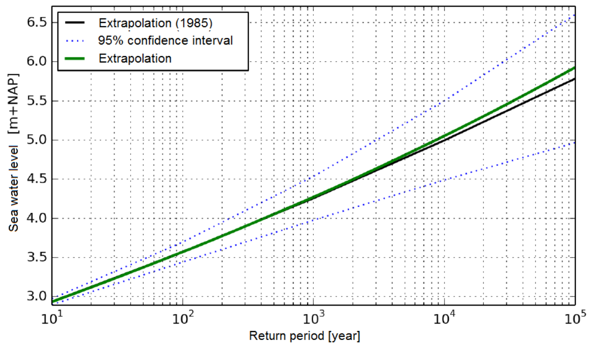


Figure 2.18: Extrapolation of historic water levels at Hook of Holland for longer return periods. Modified from Chbab (2017)

Different software, for example, Hydra-NL, can be used to calculate the hydraulic loads. Hydra-NL can calculate loads on pre-defined Hydra-NL output locations. For more information on the determination and calculation of hydraulic loads see Appendix B.1 and B.2.

2.9. Global aspects of dike design

Dike design is usually an iterative process. Crest levels, for example, are not only dependent on local wave and water level occurrences but also on the dike geometry itself. A dike design process can take many iterations before it is finished. Generally, four factors are taken into account for a regular dike design: sufficient height, stability measures, piping measures and (wave) erosion measures.

Height / overtopping measures The maximum allowable amount of water coming over a dike for a large part determines the shape of a dike. The crest level has to be sufficiently high so that overtopping or overflow will not cause erosion of the inner slope or slides. Also, the water volumes itself may not cause harm to the land behind the dike. Overtopping volumes can be influenced by altering the steepness of the outer slope or the construction of a wave berm. Making the slope steeper will cause less overtopping, as will the construction of a wave berm.

Stability measures A dike also needs to be sufficiently stable. Soil profile, geometry, strength parameters and groundwater conditions all determine the stability of a dike. The steepness of the slope can be altered to increase the stability of a dike. Also, a berm on the inner slope can be added to improve stability.

Piping measures For piping, it has to be checked if uplift, heave and pipe formation can occur. If all these three conditions can occur, additional measures have to be taken. To counteract the uplift pressures and to increase the seepage length, for example, a piping berm can be constructed on the land side of a dike.

(Wave) Erosion measures Waves can cause considerable amounts of damage to a dike. To reduce the wave load, the slope of a dike can be strengthened. This can be done by constructing a hard cover layer on the water side. This cover layer can be constructed out of asphalt, concrete blocks or rip-rap. Additionally, an offshore breakwater can be constructed to reduce the wave load.

Figure 2.19 shows a cross-section of a dike and the different design components. When a dike needs to be designed or reinforced, usually, the aspects above are altered until the dike satisfies the required safety norms.

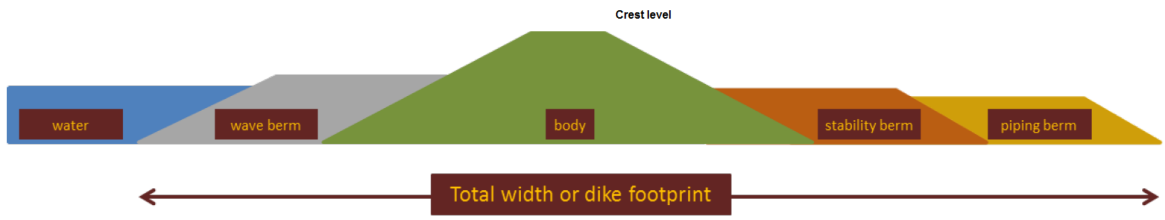


Figure 2.19: Different design components of a dike (S. Jonkman et al., 2017)

Typical dike design Because of all the design measures listed above, dikes located at different locations usually have different characteristics. Good examples are river and sea dikes. Because the conditions they are exposed to are different, sea and river dikes can be distinguished easily. The sea is a large open body where the wind has a large effect on the creation of waves. Rivers, however, are usually just up to a few hundred meters wide. It is much harder for waves to form in this small area. A sea dike therefore experiences much more wave attack and corresponding overtopping than a river dike. This means sea dikes have higher crests compared to the design water level than a river dike (to reduce overtopping). Sea dikes also usually have a berm at the seaside to help wave breaking and reduce overtopping volumes. Also, the outer slope is protected by a hard revetment to prevent damage from wave attack. Usually, waterside berms and strong revetments are not present at river dikes. High water in rivers can, however, last for a significantly longer period than storms from the sea. This means water has more time to infiltrate in the river dike and stability is a larger issue here. To make sure river dikes are stable enough, usually a relatively flat inner slope or a berm on the land side are present.

2.10. Conclusion

This chapter showed the concept of flood protection in the Netherlands. Flood defences are categorised in the primary, regional and other flood defences. It explained that all the primary flood defences are assigned a safety norm: a maximum allowable probability of causing a flood. This safety norm depends on the consequences a possible failure of a dike trajectory has. Flood defences are assessed using the Statutory Assessment Instruments (WBI) and designed using the Design Instruments (OI). These instruments state different failure mechanisms for which a flood defence needs to be assessed. Usually, some residual strength is present between the verdict that a flood defence does not meet the required safety norms according to the WBI and OI, and actually having a too high probability of causing a flood. In order to assess or design a dike, the loads on the dike need to be known. In different regions, different variables determine the loads. Design aspects of dikes consist of the shape of the dike body, the crest height, stability-, piping- and wave berms and hard revetments. In the next chapters, this knowledge will be applied to the case study: the Twin Dike project.

3
Case Study Description

This research investigates the use of multiple lines of dikes as an alternative to regular integral dike reinforcements. In order to assess the applicability of this concept, a case study will be assessed. The case that is going to be investigated is the Twin Dike project in the north-eastern part of the Netherlands which is currently being constructed. This chapter will first present the background of this project. Next, the safety concept and other characteristics of the project are explained. Information from this chapter will be used to perform an assessment in the following chapters.

3.1. Assessment results

In the third national assessment round all the Dutch primary flood defences have been assessed on their safety. This assessment has been performed using the old safety norms (see Section 2.3). The 11.7 km stretch of dike between Eemshaven and Delfzijl did not satisfy the required safety norm (see Figure 1.3). This dike stretch is managed by the local water board; Noorderzijlvest. As can be seen in Figure 2.6a, this area had to be able to withstand design water levels which have an average return period of 4,000 years. The failure mechanisms that did not satisfy the required safety norms were macro-instability of the inner slope, stability of the stone revetment and the grass cover being located below the required height. Table 3.1 shows an overview of all the failure mechanisms for which the required safety norms were not met and the corresponding length over which this was the case.

As mentioned in the paragraph above, the assessment was performed using the old safety norms. As of 2017, the safety norms and assessment instruments have been updated. Figure 2.6b shows that for this dike trajectory has a minimum allowable return period of causing a flood of 3,000 years. Because of these changes, this dike stretch has been reassessed using the current methods. The results are also depicted in Table 3.1. It can be seen that each failure mechanism has an even larger length over which it does not satisfy the required safety norms.

Failure mechanism	**Third assessment round**	**Reassessment**
Macro-instability inner slope	11.5 km	Was not reassessed
Grass cover below required level	8.5 km	-
Erosion grass revetment outer slope	-	11.7 km
Stability stone revetment	3.0 km	3.6 km
Erosion due to overtopping (Height)	-	10.3 km

Table 3.1: Failure mechanisms that did not meet the required safety norms at the Eemshaven-Delfzijl dike stretch and the corresponding length over which this was the case (Waterschap Noorderzijlvest, 2016b).

In Table 3.1, the failure mechanism *erosion of the grass revetment outer slope* is comparable to *grass cover below required level*. Additionally, overtopping is shown in the reassessment column. Dike stretches did not meet the required safety norms for overtopping if overtopping volumes during design conditions were more than 5 l/s/m (Waterschap Noorderzijlvest, 2016a). Then, the overtopping volumes can cause erosion of the inner slope.

3.2. The reinforcement project

Because this dike stretch did not meet the required safety norms, it needs to be reinforced. A large part of the dike will be integrally reinforced the traditional way. The crest will be elevated, a macro-stability berm will be constructed and the revetment will be modified/pulled up (Waterschap Noorderzijlvest, n.d.-a). 2.5 km of the dike will, however, be integrally reinforced in a different way. Along this stretch, the Twin Dike project will be implemented (see Figure 1.3). Instead of integrally reinforcing the dike the traditional way, a smaller second dike will be constructed behind the existing dike (see Figure 1.2). At this particular 2.5 km stretch, stability of the stone revetment was sufficient. So only macro-instability of the inner slope, erosion of grass revetment and overtopping were failure mechanisms that did not meet the required safety norms.

The Twin Dike project consists of two dikes which lie behind each other. The existing dike, next to the sea, will become the outer dike. Except for the construction of a macro-stability berm, this dike will not be altered. The stone revetment on the seaside will not be pulled up and the crest will remain at the current height (Wijermars, 2017). The current height of the crest of the dike is in between 7.9 and 8.6 m + NAP. A second (inner) dike will be constructed up to 500 m behind the existing dike with a height of 4.0 m + NAP. Because the outer dike is not being elevated and strengthened, more water will be able to come over this dike compared to a case where it is integrally reinforced the traditional way. The inner dike will make sure that the water is retained in the basin in between the two dikes and the hinterland is not flooded. The idea is that the two dikes together provide the required safety for the area.

The basin in between these two dikes has a total area of approximately 0.5 km^2. It will be split up into two areas: area B in the north and area C in the south. New land-use will be allocated to these areas to experiment with innovative projects. Since saline water will sometimes overtop the outer dike and enter these areas, the current land use (cultivation of potatoes) will not be optimal for this location.
Area B is therefore planned to house saline agriculture. Here, products will be cultivated that grow well in saline conditions, like sea vegetables and cockles. A study shows that saline agriculture has the potential to be more profitable than the current land-use (Kwakernaak & Lenselink, 2015). This will be an experimental location where this innovation will be stimulated.
Area C is planned to become a so-called silt engine. Under everyday circumstances, water from the Ems-Dollard will be able to deposit its sediment here. The deposition of silt will cause the land to naturally grow (elevate) over time, adding to the flood safety of the area (Kwakernaak & Lenselink, 2015). Alternatively, the silt can be excavated and used for future dike reinforcements in the area, which is predicted to be cheaper than using materials from regular sources (Waterschap Noorderzijlvest, 2016c). Also, it will reduce the amount of sediment in the water of the Ems-Dollard estuary which will have a positive ecological effect (Waterschap Noorderzijlvest & Grontmij Nederland B.V., 2016). The reduction of sediment will however be negligibly small, but also this area is an experimental location to assess the effects of this innovation.
For the realisation of the silt engine, the original dike will contain an inlet structure which will let water in and out of the basin during daily tidal conditions. During storm conditions, a movable gate is able to close this inlet structure. A detailed design of the structure is not available yet. The predicted dimensions vary by source. Waterschap Noorderzijlvest (2016b) mentions a 12 m wide and 2.5 m high inlet, Waterschap Noorderzijlvest (2016a) mentions a 12 m wide and 3 m high inlet, personal communication with Kees de Jong revealed a 6 m wide and 2.5 m high inlet with a sill level of -0.5m + NAP, Sweco (2018) mentions a sill level of -0.25 m + NAP. A detailed overview of this area can be seen in Figure 1.3. Figure 3.1 shows a schematic overview of the situation. The inner dike is divided into three sections.

The overall goal of this project is to investigate the possibility to conduct dike reinforcements in a cheaper way. Constructing a second dike and leaving the need to reinforce the original dike is predicted to be cheaper than performing a traditional reinforcement. Both the Twin Dike and a regular reinforcement protect the area behind the dikes against floods. For the Twin Dike, there are however additional

Figure 3.1: Schematic overview of the Twin Dike project. Blue: original (outer) dike, purple: second (inner dike). The location of the inlet structure is depicted with the red circle.

costs because the area in between the two dikes needs to be bought or in this case leased from the current owners. Experiments are performed with new land uses which have the potential to have higher revenue than the current land use. This way the net costs of a Twin Dike project are reduced. If this project turns out to be profitable, landowners might want to cooperate without the government having to lease the land from current owners, because their land will be more valuable after the construction of a Twin Dike. If this pilot project turns out to be successful it might be implemented on larger scales in the future (Kwakernaak & Lenselink, 2015).

3.3. Safety concept

According to the Statutory Assessment Instruments (WBI), a dike section does not meet the required safety norms when at least one of many failure mechanisms do not meet the required safety norms along this dike section (see Section 2.5). For the Twin Dike project, this is different. Failure of this system can only occur if a specific set of dike sections fail. If the northern dike section of the outer dike fails, also dike section 1 of the inner dike needs to fail to cause a total failure of the system. The failure paths are depicted in Figure 3.2 and show the specific events that need to occur in order to have a total failure of the system.

This event tree, however, only shows which sections need to fail in order to have a total system failure.

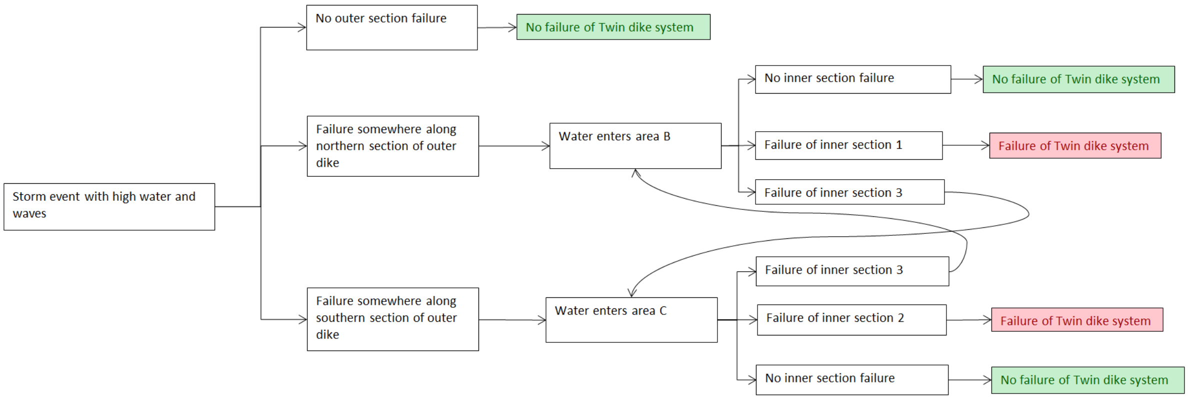


Figure 3.2: Event tree showing steps needed to have a failure of the Twin Dike system

It does not show how this can happen. The event tree depicted in Figure 3.3 shows different ways the outer dike can fail. In order to assess the Twin Dike system it is interesting to see when water enters the basin in between the two dikes. This does not necessarily have to happen when a failure occurs. This can also be seen in the event tree. Additional to all the failure mechanisms, water can also enter the area in between the two dikes due to plain overtopping (without causing erosion). This has an influence on the loads of the inner dike and will therefore be assessed in the next chapter. It is boxed in red in Figure 3.3.

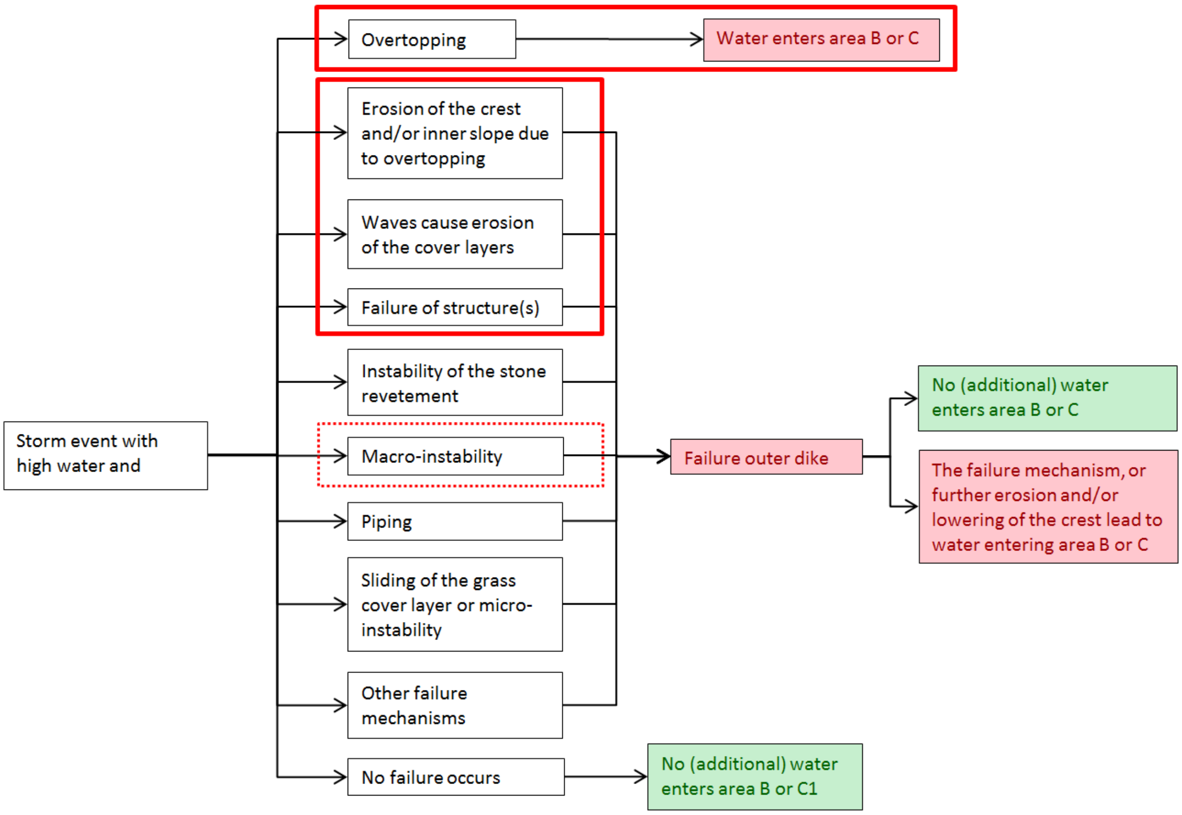


Figure 3.3: Event tree for water entering area B or C. Boxed in red are the governing failure mechanisms for the outer dike. *Macro-instability was a governing mechanism but a berm is constructed. It is therefore not regarded a governing mechanism.*

Out of all the possible failure mechanisms, not all are governing. In Section 3.1 it was discussed that this dike stretch did not meet the required safety norms for the failure mechanisms macro-instability, erosion due to overtopping and erosion of the outer slope. For all the other failure mechanisms the dike satisfied the required safety norms in the last assessment. The outer dike will remain in current condition, except for the construction of an inner berm to ensure macro-stability. Therefore, the governing failure mechanisms of the outer dike are erosion due to overtopping and erosion of the outer slope. These failure mechanisms are also boxed in red in Figure 3.3. If a structure is present, it is possible that this structure can fail too. Because the construction of the Twin Dike will also involve the construction of an inlet structure, failure of structure is also assumed to be a governing failure mechanism. This failure mechanism is therefore also boxed in red in Figure 3.3. All these governing failure mechanisms will be elaborated further in the next chapter.

In order to have a total failure of the Twin Dike system, the inner dike also needs to fail. Figure 3.4 shows the event tree for the inner dike when water is present in the basin. For the inner dike, again some failure mechanisms will be governing. Since no hard revetment or structures are present on the inner dike, these failure mechanisms cannot occur. Piping did not pose a problem at the outer dike. Since the soil surface level in the basin does not reach into the aquifer, this gives an even larger resistance against piping. It is assumed piping is not a governing mechanism. Micro-instability is not a governing mechanism if the entire dike is made from clay (Ministerie van Infrastructuur en Milieu, 2017b), which is the case. This leaves erosion of the crest and inner slope due to overtopping (height), macro-instability, erosion of the outer slope and grass cover sliding as governing mechanisms. These are boxed in red in Figure 3.4.

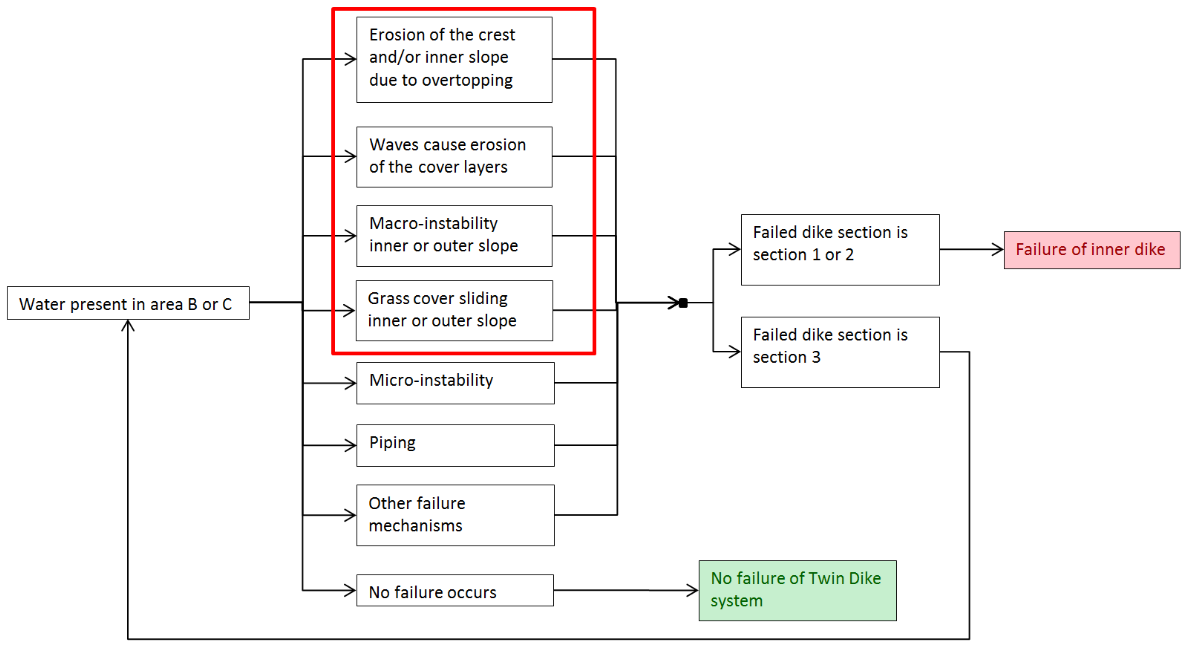


Figure 3.4: Event tree for failure of the inner dike

3.4. Design life

Usually, integral dike reinforcement projects have a design life of 50 years. The design life for the Twin dike project is rather short with only 25 years (2020 till 2045) (Wijermars, 2017). This is done because the Twin Dike is an innovative project and it is not yet known how well it will perform in a real-life situation.

Within the design life, it is expected that sea levels will rise and the land will subside. Gautuer, Schelfhout, and van Meurs (2015); Wijermars (2017) expect local land subsidence of 20 cm in this period. This is in conformity with sources like Nederlands Centrum voor Geodesie en Geo-Informatica (NCG) (2018) which expect subsidence rates up to 6 mm per year in this area. Throughout this report, land subsidence of 20 cm for the period 2020 - 2045 is used.

Also, the sea level is expected to rise within this period. Deltares (2017); Luinenburg (2016); Waterschap Noorderzijlvest (2016c); Wijermars (2017) expect a sea level rise of 30 cm until the end of the design life. Gautuer et al. (2015), however, mentions a sea level rise of 25 cm within the same period. Also Hydra-NL (see Appendix B.1) works with a sea level rise of 25 cm in 2050 compared to 2017 for climate scenario W+ (the scenario with the highest sea level rise). Since calculations will be easier using the pre-programmed climate scenarios in Hydra-NL, a sea level rise of 25 cm is used in this report. Also, the difference in sea level rise over these 5 missing years (2045 to 2050) is expected to be relatively small.

3.5. Dike geometry

In order to perform the assessment, the dike geometry needs to be known. Nine dike profiles have been obtained from Waterschap Noorderzijlvest (2010). This document provides information on dike geometry for all the dikes which are managed by water board Noorderzijlvest. In this area, profile 76 to 84 are located. The locations of these profiles are depicted in Figure 4.1a as red dots with corresponding profile number. The exact profiles can be seen in Appendix D.

The dike profiles reach to the still water line at approximately 0 m + NAP. This area, however, has extensive foreshores which need to be taken into account. An estimate of the geometry of the foreshore has been made using *Actueel Hoogtebestand Nederland* (2009). Also, the orientation of the dike needs to be known, this is calculated using the computer program *QGIS*. This software is able to calculate the angle of elements compared to the north. This way the dike orientation for every profile was obtained.

3.6. Dike composition

Also, the dike composition has to be known to perform an assessment. At the location of the Twin Dike project, the outer slope of the current dike will not be altered. Therefore, the current dike compositions can be used. A detailed cross-sectional composition of the dike was however not found. Therefore, other information was used to determine the composition. The website of the water board shows how the dike composition looks like. This is depicted in Figure 3.5. It shows an original sea dike constructed in 1880, with a reinforcement constructed in 1970.

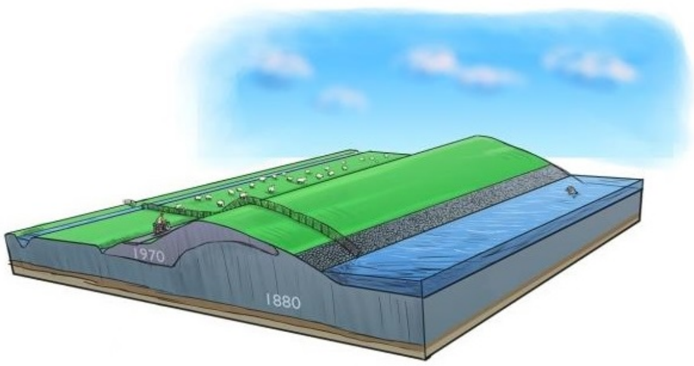


Figure 3.5: Situation of the Eemshaven-Delfzijl dike before the current integral reinforcement (Waterschap Noorderzijlvest, 2017a)

Personal communication with employees of water board Noorderzijlvest confirms that the original dike from 1880 is a clay dike which was reinforced in the 1970s as a sand dike with a 1 m thick clay cover layer. Gautuer et al. (2015) confirms the presence of an old sea dike which was reinforced with a sand core dike with a 1 m clay cover layer. Hofsté (2017) reports that the old sea dike had a height of 6 m + NAP and a crest width of 1.5 m during the construction of the reinforcement in 1970. Wijermars (2017) confirms this and also mentions that the outer slope of old sea dike coincides with the outer slope of the current dike. This is in line with what is depicted in Figure 3.5. It is therefore assumed that the old sea clay dike indeed had a crest height of 6 m + NAP during the 1970s and that the outer slope coincided with the current outer slope. With a land subsidence of 25 cm from 1970 to today, the current height is approximately 5.75 m + NAP. An image of the old design drawings as taken from Hofsté (2017) is shown in Figure 3.6. It confirms that the crest is located at 6 m + NAP.

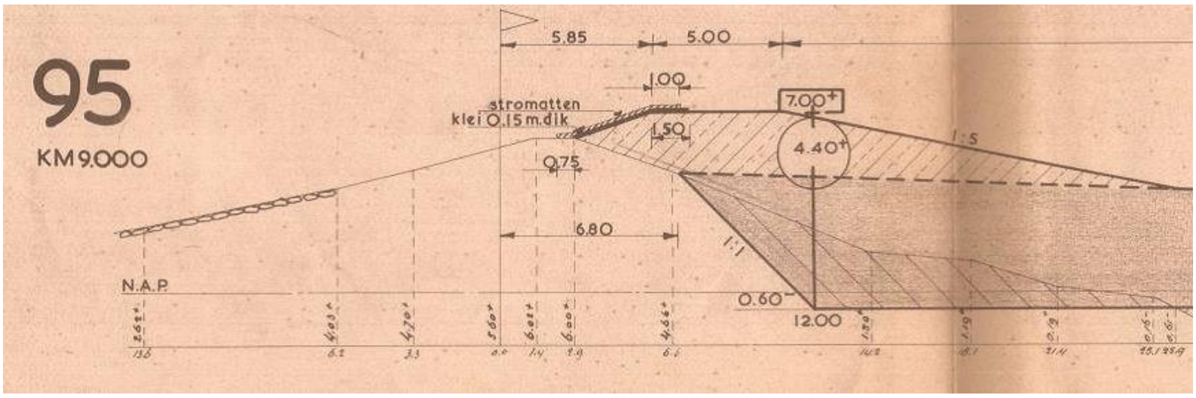


Figure 3.6: Design drawing of the 1970s reinforcement at the current location of the Twin Dike project (Hofsté, 2017)

4

General Safety Assessment Twin Dike

In this chapter, existing instruments will be used to asses the Twin Dike project. This is done in order to find out which failure mechanisms are relevant for the failure of multiple lines of dikes as flood defences. In the Netherlands, the Statutory Assessment Instruments (WBI) are used to assess primary flood defences. The Design Instruments (OI) are used to design future reinforcements. These instruments consist of rules, methods and software which have to be used when assessing or designing a primary flood defence (see Section 2.5). The methods used in this chapter are in accordance with the methods of the WBI and OI.

First, the governing failure mechanisms of the outer dike, as discussed in Section 3.3, are assessed. Next, an attempt is made to find out how the loads on the outer dike influence the loads on the inner dike. Finally, a conclusion is presented about this analysis.

4.1. Outer dike

In this section, the outer dike will be assessed for its governing failure mechanisms. As discussed in Section 3.3, the governing failure mechanisms are erosion of the crest and/or inner slope, erosion of the outer cover layers and failure of the structure. Additionally, overtopping itself is able to cause water to enter the basin in between the two dikes. This water has an influence on the loads on the inner dike. Therefore, this will also be assessed.

The first step is to divide the outer dike into dike sections. As described in Ministerie van Infrastructuur en Milieu (2017b) a dike trajectory can be cut up in several dike sections. Within these sections, the properties of the flood defence and the loads are similar to each other. This way, an assessment can be made per section. Nine dike profiles are known around this stretch of dike and 12 Hydra-NL output locations. A Hydra-NL output location is a location where Hydra-NL (see Appendix B.1) is able to determine hydraulic loads. These hydraulic loads can be determined for specific return periods. The hydraulic loads are water level, and wave characteristics (see Section 2.8). The locations of the profiles and output locations are shown in Figure 4.1a.

Using the information above, the outer dike is split into 15 dike sections. Each section is linked to a specific combination of profile and output location. The dike within a section has a similar orientation, geometry and load acting on it. When specifying the dike sections, the effects of sheltering effects of the small cove and several groynes have been taken into account. Figure 4.1b shows an overview of all assigned dike sections of the outer dike.

Table 4.1 shows the dike section ID with their corresponding profile number and Hydra-NL output location number. It also shows the length of each section. This length will be needed to calculate the average overtopping volumes later in this chapter.

In the following paragraphs, the outer dike will be assessed for the relevant failure mechanisms.

(a) Location of dike profiles (red dots) and Hydra-NL output locations (red crosses)

(b) Sections and their corresponding ID. See Table 4.1 for the corresponding profiles and output locations.

4.1.1. Overtopping

First, plain overtopping (without erosion) will be assessed. One of the main functions of the inner dike is to retain overtopping water of the outer dike and thus preventing a flood of the hinterland. The overtopping volumes will be determined and the resulting increase in water depth in the basin will be determined in this paragraph.

Input Hydra-NL (for more information see Appendix B.1) is used to determine the overtopping volumes. Hydra-NL can calculate overtopping discharge for a given dike profile which is linked to a specific Hydra-NL output location. The combinations of profiles and output locations as mentioned in Table 4.1 are used. Calculations are performed for 2 reference years. Namely, the current situation (reference year 2023) and the future situation at the end of the planning period (reference year 2045). The two reference years differ in the sense that the future situation takes into account sea level rise and soil subsidence (see Section 3.4).

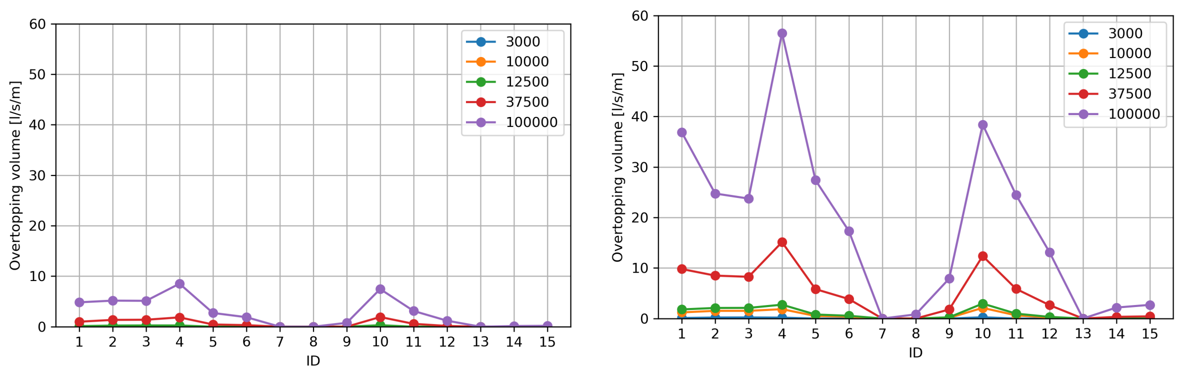
Hydra-NL is able to determine hydraulic loads for specific return periods. Each failure mechanism has a maximum allowable return periods for which it needs to be able to withstand loads. Overtopping itself, however, is not a failure mechanism. There are no safety norms which specify the maximum allowable

ID	Profile	Output location	Length [m]
1	76	WZ_1_6-7_dk_00005	105
2	77	WZ_1_6-7_dk_00006	257
3	77	WZ_1_6-7_dk_00007	240
4	78	WZ_1_6-7_dk_00008	267
5	78	WZ_1_6-7_dk_00009	67
6	79	WZ_1_6-7_dk_00009	54
7	80	WZ_1_6-7_dk_00012	272
8	81	WZ_1_6-7_dk_00012	238
9	82	WZ_1_6-7_dk_00012	199
10	82	WZ_1_6-7_dk_00013	42
11	83	WZ_1_6-7_dk_00013	74
12	83	WZ_1_6-7_dk_00014	207
13	84	WZ_1_6-7_dk_00015	165
14	84	WZ_1_6-7_dk_00016	257
15	84	WZ_1_6-7_dk_00017	172

Table 4.1: Combination of profile and output location and the corresponding length of that section.

return period for overtopping to cause a certain water volume to enter the area behind a dike. Erosion of the inner slope and crest due to overtopping have a governing return period of 37,500 years (see Section 4.1.2). The return periods assessed for plain overtopping are therefore also chosen in this range. A total amount of nine return periods is assessed.

Results Figure 4.2 shows the governing average overtopping volumes per return period for the different combination of profile and output location. ID 1 on the left of the graph, corresponds with ID 1 in the most northern part of Figure 4.1b. See Table 4.1 and Figure 4.1b for the definition and location of the IDs. All the results can be seen in table format in Appendix C.1.



(a) Current situation

(b) Future situation (2045)

Figure 4.2: Average peak overtopping volumes per return period and ID, see Table 4.1 for the corresponding profiles and output locations.

Looking at Figure 4.2 some interesting things can be noted. Overtopping volumes in 2045 are a factor

5 to 6 higher than in the current situation. This means that sea level rise and soil subsidence have a significant impact on the overtopping volumes. This effect will be analysed further in the sensitivity analysis in chapter 6. In the 2045 situation only return periods 3,000 years and higher are causing overtopping volumes of more than 0.1 l/s/m. It is also noted that ID 7 and 8 do not have significant overtopping in any of the return periods. This is most probably due to their sheltered location. Waves and wind come into the Ems-Dollard estuary from the north, this makes that ID 7 and 8 are not directly exposed to these waves (see Figure 4.1b). The thing that is, however, most interesting, is the effect on these overtopping volumes on the water level inside the basin

In order to calculate the expected rise in water level in the basin due to the overtopping volume, it is assumed that the calculated overtopping volume is present over the entire length of the corresponding dike section. A peak duration of the storm of 4 hours is assumed. This is the time the peak of a storm in the Wadden Sea lasts according to Ministerie van Infrastructuur en Milieu (2018). It is assumed that the maximum overtopping volume is present for the entire peak. In reality, the peak will only last for a moment, so the overtopping volume is overestimated. Table 4.2 shows the total amount of water entering the basin in between the two dikes after a storm.

Return period	3,000	10,000	12,500	37,500	100,000
Current situation					
Water depth basin B [m]	0	0.01	0.01	0.06	0.24
Water depth basin C [m]	0	0.01	0.01	0.01	0.09
Future situation (2045)					
Water depth basin B [m]	0.01	0.06	0.09	0.42	1.44
Water depth basin C [m]	0.01	0.01	0.03	0.18	0.79

Table 4.2: Increase in water depth in the basin after a 4 hour storm with peak overtopping volume being present over the entire length of each section per return period

Table 4.2 shows that the rise in water level in each of the basins is lower than 25 cm for the current situation. For the future situation (2045) the increase in water depth in the basin is larger. The increase is still, however, less than 10 cm for a 1/12,500 per year or more frequent situation. For a 1/37,500 per year situation, which is the required safety level for erosion due to overtopping, the increase in water level is around half a metre. For a return period of 100,000 years, the increase is up to one and a half metres. This return period is however very extreme.

The results have also been calculated by interpolating the known overtopping volumes over the distances in between the known points instead of assuming the overtopping volume over the entire stretch. The resulting depth fluctuates a few centimetres above or below the calculated depth in Table 4.2.

It is interesting to see that the resulting increase of water depth in the basin is relatively low. Without the presence of the inner dike these overtopping volumes would most probably not lead to a flood in the first place. A flood is an event which leads to significant damage or casualties (see Section 2.5.1). Water depths caused by the overtopping water are probably in the range of a few centimetres. This area is sparsely inhabited and mostly consists of farmland. A few centimetres of water will not lead to significant damage or casualties. This would suggest that the construction of the inner dike is not necessary for the counteraction of overtopping volumes.

4.1.2. Grass cover erosion crest and inner slope (GEKB)

Next, the failure mechanisms *grass cover erosion of the crest and inner slope* is assessed. The original dike at the Twin Dike location did not meet the required safety norm for this failure mechanism. In order to have a more detailed view of this failure mechanism, it is assessed again in this section.

Due to overtopping, fast flowing water on the inner slope or crest can damage the grass cover causing the core of the dike to be exposed. Figure 4.3 illustrates the failure mechanism.

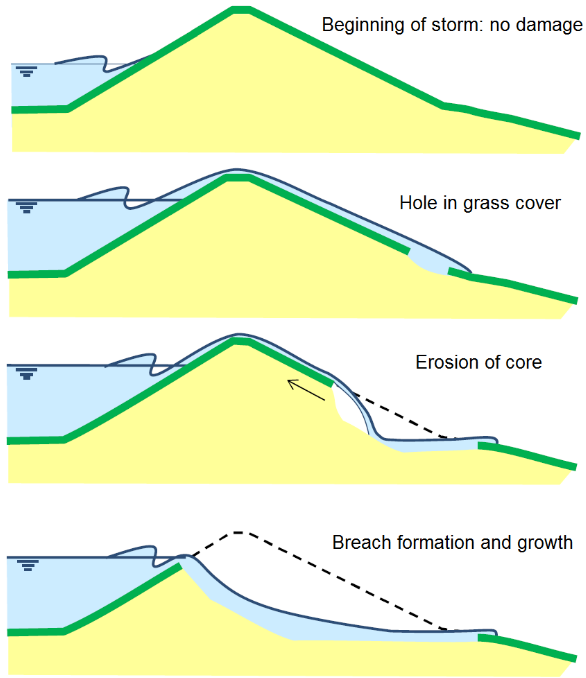


Figure 4.3: Step by step failure due to erosion of the inner slope or crest. Modified from 't Hart (2018)

Input In the Twin Dike region, the maximum allowable probability of causing a flood is 1/3,000 per year (see Section 2.3). Equation 2.4 can be used to calculate the maximum allowable probability of failure for this specific mechanism. The contribution factor for *grass cover erosion of the crest and inner slope* is 0.24 (see Table 2.2). With a length effect factor of 3 (Ministerie van Infrastructuur en Milieu, 2018), this leads to a maximum allowable probability of failure of 1/37,500 per year for this failure mechanism.

RisKeer is a computer program which can calculate the safety for erosion due to overtopping for current situations. It performs calculations in a probabilistic manner. It is assumed that the inner slope and crest can withstand a certain critical overtopping volume. This volume depends on the quality of the grass, the wave height class and has a certain mean and standard deviation. Next, it checks what the probability is that the occurring overtopping volume is larger than the critical overtopping volume. More background on RisKeer and its overtopping erosion assessment can be found in Appendix B.5.

In RisKeer, again a Hydra-NL output location can be assigned to a profile. For this combination, the assessment is performed. The combinations from Section 4.1.1 are again used. The wave heights at a return period of 37,500 years are in the range of 2-3 m and a closed sod quality is assumed. This leads to the inner slope withstanding overtopping volumes with a mean of 70 l/s/m and a standard deviation of 80 l/s/m (see Appendix B.5).

Results The resulting failure probabilities per combination of profile and output location are given in Figure 4.4. It can be seen that for all the dike sections the probability of failure is well below the maximum allowable probability of failure 1/37,500 per year (depicted with a red line in the figure).

As mentioned in chapter 3, the dike did not meet the required safety norms for overtopping (reference year 2012). It is then strange to see that during this assessment the dike satisfies the required safety norms for this failure mechanism. This can be explained by the recent change in assessment techniques for this failure mechanism. In the last assessment of the Eemshaven-Delfzijl dike stretch, a dike

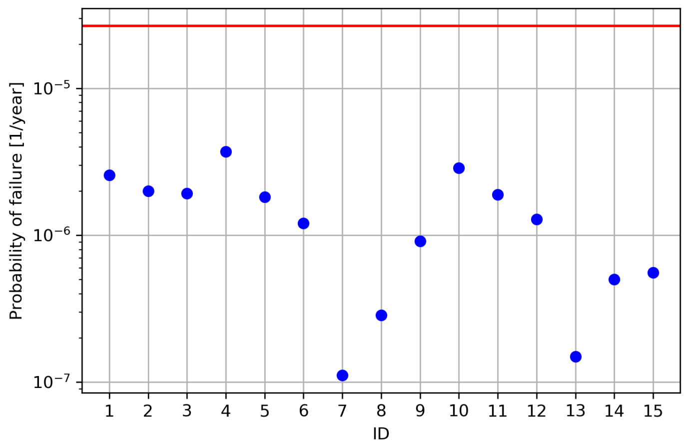


Figure 4.4: Probability of failure [1/year] per section of erosion of the crest and/or inner slope due to overtopping as calculated with RisKeer (current situation)

did not meet the required safety norms if the average overtopping volume exceeds 5 l/s/m (Waterschap Noorderzijlvest, 2016a). In the current assessment, the resistance of a grass cover has a probabilistic distribution instead of being a fixed value of 5 l/s/m. Figure 4.2a, however, shows that overtopping volumes for a 1/37,500 per year situation are less than 5 l/s/m everywhere. The last assessment also used an older version of Hydra-NL where overtopping volumes in this region were overestimated. The combination of a new Hydra version and the probabilistic distribution for the grass cover lead to a difference in results.

The results above are for the current situation. For this project, it is more interesting to see the probability of failure at the end of the planning period. RisKeer is however not able to compute for future reference years. The Design Intruments (OI) states that if the significant wave height is below 3 m, the sod quality is closed and the clay layer has a thickness of at least 0.4 m, a critical overtopping volume of 15 l/s/m can be used (Ministerie van Infrastructuur en Milieu, 2016a). This is all true for this case. Figure 4.2 shows that in reference year 2045, for return period of 37,500 years the overtopping volumes are below 15 l/s/m everywhere except for one location. At this location, the overtopping volume is 15.1 l/s/m. This would mean that at this location the required safety is not met. However, the difference is only 0.1 l/s/m. Also, tests using the overtopping simulator show that well-closed grass covers without open holes can sometimes resist overtopping volumes of 100 l/s/m (EurOtop, 2016). This means that an overtopping volume of 15.1 l/s/m should pose no problems, provided the inner slope is of good quality.

4.1.3. Grass cover erosion outer slope (GEBU)

The next failure mechanism that will be assessed is erosion of the grass cover on the outer slope due to wave impact or run-up. During a storm with waves, the impact of the waves or the run-up can damage the grass revetment, eventually exposing the core and causing a breach. Figure 4.5 illustrates an overview of the failure mechanism.

Using assessment methods of the WBI and OI, calculations are performed semi-probabilistically with a cumulative overload method. During an entire storm, the loads on the grass revetment are added up and it is checked whether this value is lower than a certain critical overload value. If this is the case the revetment is strong enough, otherwise not. The calculation can either be performed for a run-up load or a wave impact load. If the grass revetment is present below the design water level, wave impact is governing and only this mechanism has to be assessed. Otherwise, wave run-up is governing and only this mechanism has to be assessed.

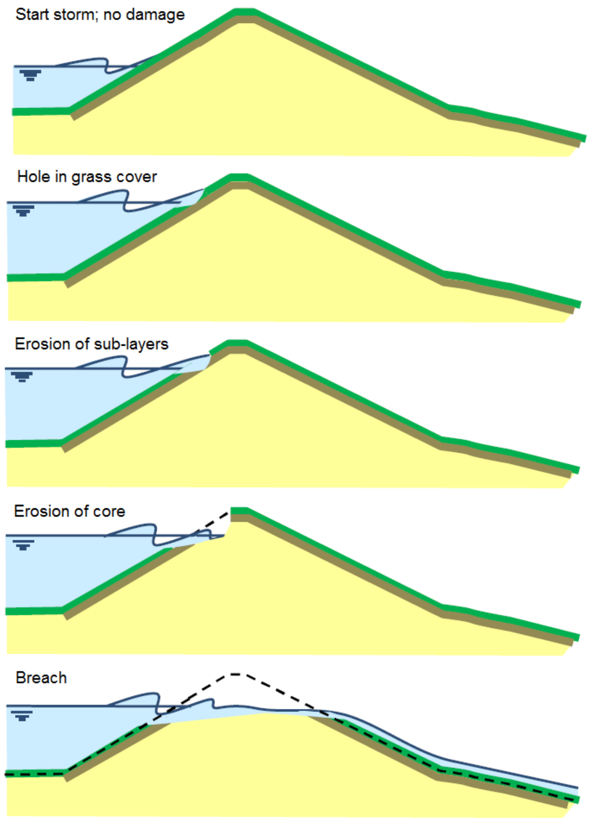


Figure 4.5: Step by step failure due to erosion of the outer slope. Modified from 't Hart (2018)

Input
In the Twin Dike region, the maximum allowable probability of causing a flood is 1/3,000 per year (see Section 2.3). Equation 2.4 can be used to calculate the maximum allowable probability of failure for this specific mechanism. The contribution factor for *grass cover erosion of the outer slope* is 0.05 (see Table 2.2). With a length effect factor of 3 (Ministerie van Infrastructuur en Milieu, 2016a), this leads to a maximum allowable probability of failure of 1/180,000 per year for this failure mechanism.

Return periods This failure mechanism is assessed using the computer program *BM Gras Buitentalud*, see Appendix B.3 for more information. The program can check whether a grass revetment is safe enough for a single return period event. The output of *BM Gras Buitentalud* is a single safety factor per specified return period. The program will not calculate the actual probability of failure itself. Therefore, it has to be known for which return periods the assessment has to be performed. The return periods that are going to be used are the inverse of the five failure probability limits used for the assessment verdict categories at dike section level (Ministerie van Infrastructuur en Milieu, 2017b). For this dike trajectory and this failure mechanism, this results in failure probability intervals as given in Table 4.3.

The return periods used are thus $18,000,000$; $600,000$; $180,000$; $3,000$ and 100 years. However, large intervals now exist between the different return periods. That is why return periods of 10; 300; $1,000$; $10,000$ and $60,000$ years are also added.

Water level As input *BM Gras Buitentalud* needs the water levels over the course of the storm. The calculations are performed for the same set of different combinations of Hydra-NL output locations and profiles as in section 4.1.1. In accordance with the WBI and OI, the water level course over time will be obtained in the following way.

Cat.	Description	Interval for Twin Dike area
I	Easily satisfies signal norm	$P_f < 1/18,000,000$
II	Satisfies signal norm	$1/18,000,000 < P_f < 1/600,000$
III	Satisfies lower bound	$1/600,000 < P_f < 1/180,000$
IV	Maybe satisfies lower bound	$1/180,000 < P_f < 1/3,000$
V	Does not satisfy lower bound	$1/3,000 < P_f < 1/100$
VI	Does not satisfy lower bound by far	$P_f > 1/100$

Table 4.3: Categories for failure mechanism assessments [1/year], dike section level (Ministerie van Infrastructuur en Milieu, 2017b)

The first step is to calculate the governing water level per location per return period. This will be the maximum water level over the course of a storm. The water level is obtained with Hydra-NL. Hydra-NL is unfortunately not able to calculate with return periods larger than $1,000,000$ years. For now, this means the calculation with a return period of $18,000,000$ years is discarded.

Next, the course of the water level over time has to be known. Ministerie van Infrastructuur en Milieu (2018) gives a typical storm surge duration in the Wadden Sea of 45 hours. During that time, the shape of the storm surge is as given in Figure 4.6.

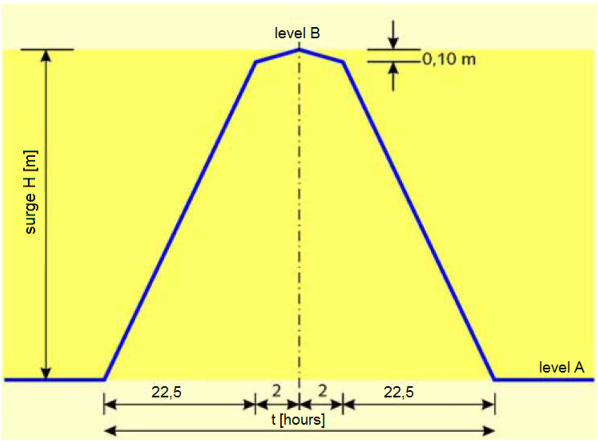


Figure 4.6: Storm surge course during a typical 45 hour storm at the Wadden Sea. Modified from Ministerie van Infrastructuur en Milieu (2018)

It has to be kept in mind that at this location a tide is present. The tide is found by averaging a regular tide series of Eemshaven and Delfzijl (Rijkswaterstaat, n.d.). The peak of the tide has to coincide with the peak of the storm surge so that the maximum water level during a storm for a specific return period is equal to the governing water level as calculated with Hydra-NL. This means level B in Figure 4.6 is equal to this water level minus the amplitude of the tide. Level A is equal to the average water level (approximately 0 m + NAP at sea). Figure 4.7 shows an example of a resulting water level course over time.

This approach is conducted for all return periods and for all locations. The imposed tide will always be the same, as well as the surge shape. Only the surge height will differ per location and return period.

Wave load Next, the wave loads during the storm have to be known. Hydra-NL can compute wave loads on grass cover revetments for a given water level and return period. In the most extreme case (reference year 2045 and return period 600,000 years), the water levels reach till 7.79 m + NAP. Hydraulic wave loads are thus needed for water levels in the same range. A step size needs to be defined for the interval of the evaluated water levels. A smaller interval gives more precise results but more

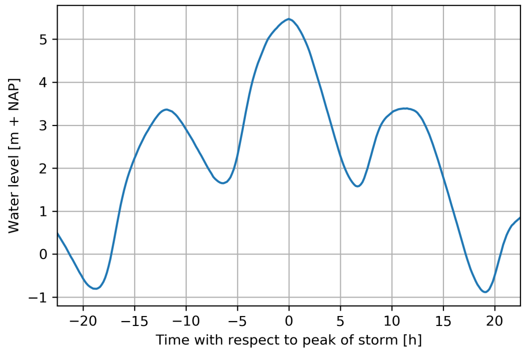


Figure 4.7: Water level course at Hydra-NL output location 15 for 1/1,000 per year situation, current situation (without sea level rise).

computations are needed, which are time-consuming. Hydra-NL can only compute loads for minimum water levels of 0 m + NAP. Due to the tide, water levels are sometimes below 0 m + NAP. However, this is only for short moments before and after the worst of the storm. So the difference in wave height will not have a large impact on the results. Loads are thus calculated for water levels ranging from 0 m + NAP till 7.8 m + NAP and a step size of 0.1 m is chosen.

Governing calculation: wave impact or wave run-up The next step is to see which calculation is governing: wave impact or wave run-up. This is dependent on the location of the grass revetment. If a grass revetment is present below the still water line, wave impact is governing. If it is only present above the still water line, wave run-up is governing. The computer program QGIS is used to find the elevation of the transition between stone revetment and the grass cover slope. Using satellite images and the *Actueel Hoogtebestand Nederland* (2009), this elevation can be found. The results are depicted in Figure 4.8. The figure shows that the grass-stone transition is located at approximately 3.5 m + NAP, except for the first 200 m. Here an asphalt cover is located till 5.5 m + NAP. This elevation has also been confirmed with old design drawings (see Figure 3.6) and correspondence with the local water board. However, a small beach is also present in the project area. At this location it was not possible to determine the elevation of the stone revetment from satellite images (if a stone revetment is present at all at this location).

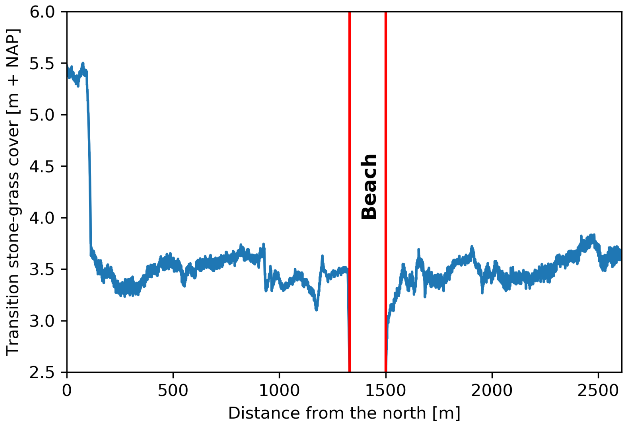


Figure 4.8: Level of transition stone-grass revetment

The lowest water level evaluated is 3.895 m + NAP for output location 3, profile 76, a return period of 10 years and for the current situation. The elevation of the stone-grass transition is lower everywhere except for the first 200 m. Here an asphalt revetment is present till an elevation of 5.5 m + NAP. This means that for all calculations wave impact is governing, except for a few lower return periods at the first 200 m, which is represented by ID 1. These will be discarded for now, and only wave impact will be assessed.

Finally, a few other variables have to be given.

- The top of the grass revetment has to be given. Since grass covers the entire upper outer slope, crest and inner slope this value is equal to the crest level of each profile that is assessed.
- Coefficients a, b and c. Dependent on sod quality. See Table 4.4.
- Vertical step size for calculations. Default value of 0.1 m.
- Minimum and maximum water level to evaluate. These are the minimum and maximum occurring water levels which occur during the course of the storm.
- Minimum and maximum wave heights to evaluate. Maximum = maximum occurring wave height. Min = default value, coefficient c (see Table 4.4).
- Clay layer thickness of the grass cover. The residual strength of a clay layer with a thickness of up to 0.5 m can be evaluated. The thickness is around 1 m at this location, so 0.5 m is chosen as value.
- Sand content in clay. This has to be determined with laboratory tests. According to Ministerie van Infrastructuur en Milieu (2018), regular firm clay has a maximum sand content of 0.4. The higher the sand content, the lower the strength. For now, a value of 0.4 is assumed.

	Closed sod	Open sod
a	1.0	0.8
b	-0.035	-0.070
c	0.25	0.25

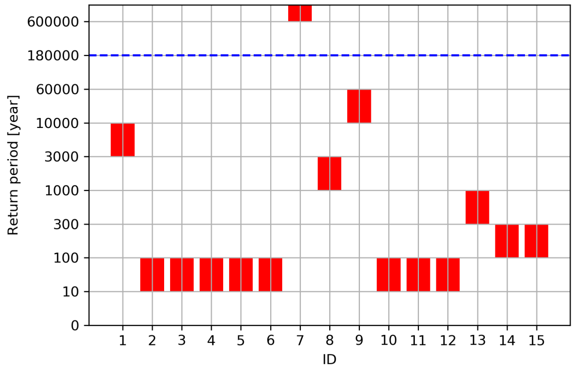
Table 4.4: Coefficients a, b and c for different kind op sod qualities (Ministerie van Infrastructuur en Milieu, 2018)

The calculations were performed for both open and closed sod qualities, for the current situation and for reference year 2045 (end of design life period).

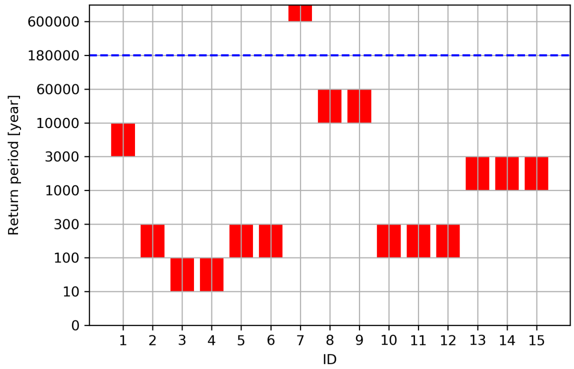
Results

Summarising, the calculations have been performed for nine return periods, two reference years, two sod qualities and on 15 locations (see Table 4.1). This results in a total of 540 cases and corresponding safety factors. The return period for which the grass cover fails is in between the return periods where the safety factor is above 1 for the last time and below 1 for the first time. Figure 4.9 shows the interval where failure occurs based on these criteria. The full results can be seen in Appendix C.2.

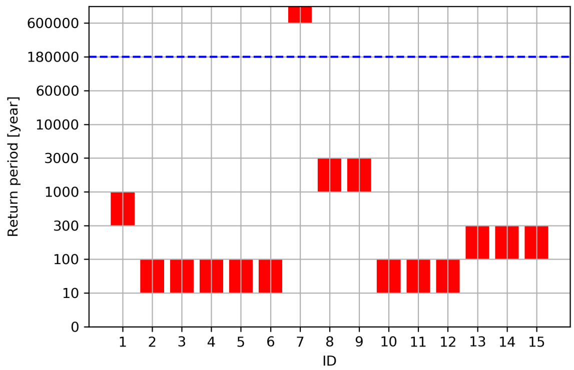
Figure 4.9 shows that the probability of failure is much higher for erosion of the outer slope than for erosion of the inner slope. Using Equation 2.4, a regular dike has to withstand conditions with a return period of at least 180,000 years for this failure mechanism. The figure shows that only one section meets this requirement. In reference year 2045, for a closed grass quality, more than 80% of the total length of the dike is not able to withstand conditions with a return periods of 1,000 years. Around 30% of the total length cannot withstand conditions with a return period of 100 years. If this were a regular single dike, the required safety for this failure mechanism is not met by far according to the WBI and OI. This is, however, a system of multiple lines of dikes. Failure of the outer dike does not immediately imply failure of the inner dike and thus a total system failure. Nevertheless, the outer dike fails at relatively low return periods. The effect of this failure mechanism will, therefore, be further analysed in the next chapter.



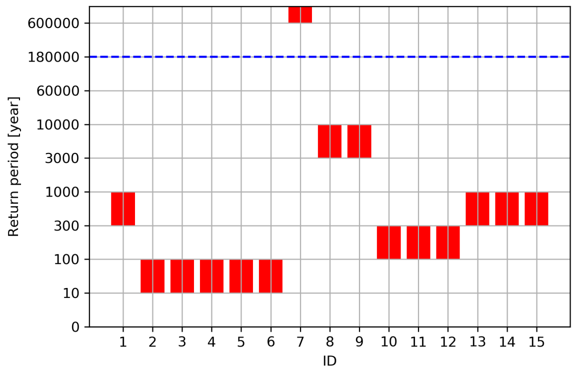
(a) Failure interval for the current situation, grass quality open



(b) Failure interval for the current situation, grass quality closed



(c) Failure interval for reference year 2045, grass quality open



(d) Failure interval for reference year 2045, grass quality closed

Figure 4.9: Return period interval for which the grass cover fails for different combinations of reference year, grass quality and ID. The blue line indicates the required safety level for this failure mechanism if this was a regular dike.

The hydraulic loads which were used to calculate the results in Figure 4.9 were determined with Hydra-NL. Also the program RisKeer (see Appendix B.5) can be used to determine these loads. RisKeer uses a slightly different calculation method than Hydra-NL. To see the influence of this difference in calculation method, some calculations have also been performed with RisKeer. This showed that the hydraulic wave loads calculated with RisKeer were slightly lower than calculated with Hydra-NL for the higher ranges of return periods. The wave heights, which are in the range of 1.5 m to 2.0 m, are approximately 10 cm lower in the cases with higher return periods (180,000 years). It is expected that this does not influence the results, since the loads in the higher return periods are so large that a failure is guaranteed. RisKeer is also only able to calculate hydraulic loads for the current situation, and not for future reference years. It is therefore decided to not use the hydraulic loads calculated by RisKeer for this analysis.

4.1.4. Structure

The final failure mechanism for the outer dike is failure of the inlet structure. The predicted location of the structure is depicted in Figure 3.1 with a red circle. As described in Section 3.2, the exact dimensions of the inlet structure are not yet known. The width is either 6 m or 12 m, the height is between 2.5 m and 3 m, and the sill level is between -0.25 m and -0.5 m + NAP. Since the exact design of the structure is unknown and assuming that after the construction of the inlet it satisfies the safety norms, the failure mechanism is not discussed in depth. It is only discussed briefly.

A structure has multiple possible failure mechanisms. For this structure these are an insufficient height (HTKW), failure of closure (BSKW), piping (PKW) or the strength and stability can be insufficient (STKW). These four additional failure mechanisms will be discussed below.

Reliability closure Assuming the smallest inlet opening mentioned above, a 6 m wide, 2.5 m high inlet with a sill level around -0.25 m, is capable of letting in a large amount of water when a closure fails. The inner dike is only 4 m + NAP high and area C behind the tidal inlet structure has an area of 0.16 km². This means the water level in the basin will probably level out fast with the outside seawater level. Figure 4.10 shows the water level at hydra location 15 (close to the inlet) versus the return period for reference year 2050. It can be seen that already at a return period of 10 years a water level of 4 m + NAP is exceeded. This means that if closure fails the complete Twin Dike system will already fail at conditions with an average return period of 10 years.

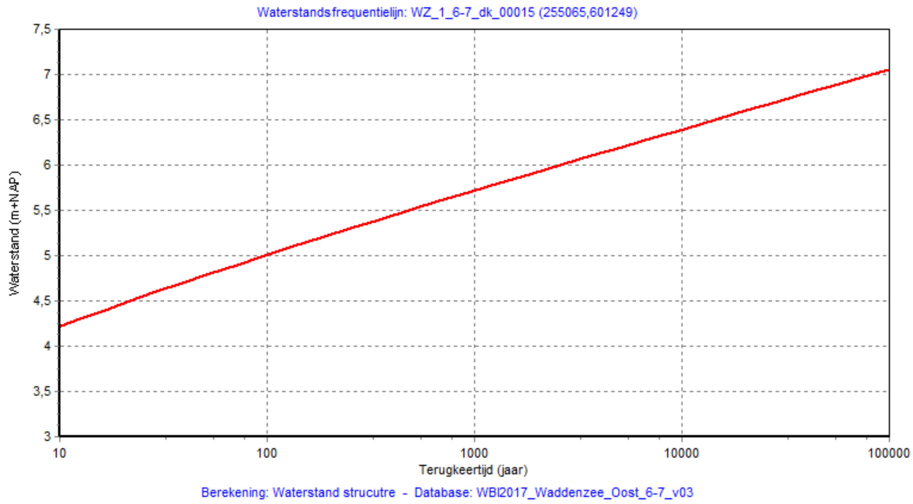


Figure 4.10: Frequency line for the water level in front of the planned location of the tidal inlet.

According to Table 2.2, failure of closure of a structure has a failure contribution factor of 0.04. This means the allowable probability of failure without the length effect is equal to 1/3,000 * 0.5 = 1/75,000 per year. Due to the fact that the inner dike does not provide much additional safety against failure of closure, it is advisable that the inlet structure has sufficient reliability. The construction two gates behind each other can, for example, provide the needed redundancy.

Height The total height of the structure is not known. Assuming it is embedded in the dike, and the original dike continues on top of the structure, or the top of the structure has the same height as the surrounding dike, the overtopping volumes are not more than calculated in Section 4.1.1.

Piping and strength Also, the strength and resistance against piping are hard to discuss without a detailed design. But during the construction, the structure can be made such that it is strong enough and does satisfy the required norms. The same applies to piping. Additional ground works can be performed to ensure the required safety.

4.2. Inner dike

In this section, the inner dike will be assessed on its relevant failure mechanisms. As discussed in Section 3.3, the relevant failure mechanisms are erosion of the crest and/or inner slope due to overtopping, macro-instability, erosion of the outer slope and grass cover sliding.

First, it will be analysed what the governing loads (waves and water levels) in the basin can become. Next, the relevant failure mechanisms will be assessed. The inner dike is assessed for both the northern part as well as the southern part (dike section 1 and 2 in Figure 3.1).

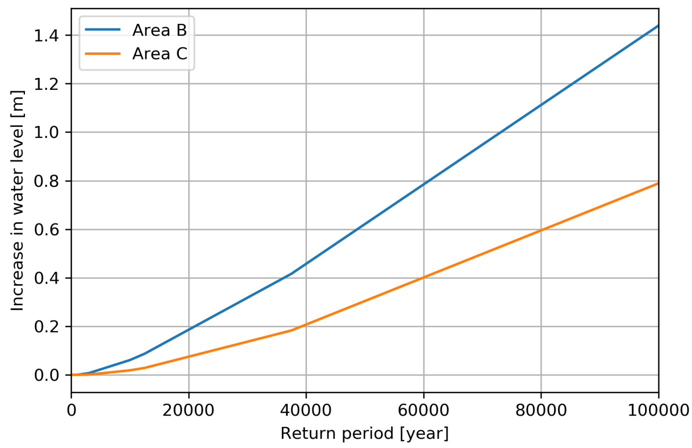


Figure 4.11: Increase in water depth per return period in the basin due to overtopping, reference year 2045

4.2.1. Loads inner dike

Water level The water level inside the basin is determined by the regular tidal in- and outflow through the inlet structure. Additionally, during storm conditions, the water level can rise due to overtopping and failure of closure of the structure. The rise in water level due to failure of closure of the structure is hard to predict since the exact probability of failure of the structure is unknown. If the structure has a maximum allowable probability of failure of 1/75,000 per year (as required, see Section 4.1.4), the chances of water levels of higher than 4 m + NAP are 1/10 (probability of 4 m + NAP water level, see Figure 4.10) * 1/75,000 = 1/750,000 per year. The reliability of closure of the structure is deemed outside the scope of this report and will not be discussed further.

The increase in water depth in the basin due to overtopping was discussed in Section 4.1.1. Figure 4.11 shows the increase in water depth in each basin per return period due to overtopping. It can be seen that the water depth increases several decimetres up to 1.5 m for a return period of 100,000 years.

The total amount of water inside the basin is dependent on the closure regime of the inlet structure. According to different sources the inlet structure closes at an outside water level of 2.0 m + NAP (Luinenburg, 2016; Vranken & Sinoo, 2016; Waterschap Noorderzijlvest, 2016a), 1.75 m + NAP (Hofsté, 2017) or 2.5 m + NAP (Gautuer et al., 2015). However, more recently de Jong (2019) mentions a more detailed closure regime where the closure level is dependent on the storm predicted. This results in the closure levels per return period as mentioned in Table 4.5. Summing the closure levels from Table 4.5 and the fragility curve from Figure 4.11 leads to a result as depicted in Figure 4.12.

Return period [year]	200	400	1,000	2,000	4,000	10,000	20,000	37,500
Closure level [m + NAP]	3.5	3.5	3.4	3.4	3.3	3.0	2.3	1.2

Table 4.5: Closure levels per return period as mentioned in de Jong (2019)

Additionally, the water levels inside the basin can also increase due to wind set-up. This can be calculated with Equation 4.1. The set-up is dependent on the wind speed, water depth and the fetch. The exact value of the set-up will be determined per relevant failure mechanism in the next section.

$$\frac{dS}{dx} = C_2 \frac{u^2}{gd} \tag{4.1}$$

In which:

S = total wind set-up [m]
C_2 = constant $\approx 3.5 \cdot 10^{-6}$ to $4.0 \cdot 10^{-6}$
d = water depth [m]
u = wind velocity [m/s]

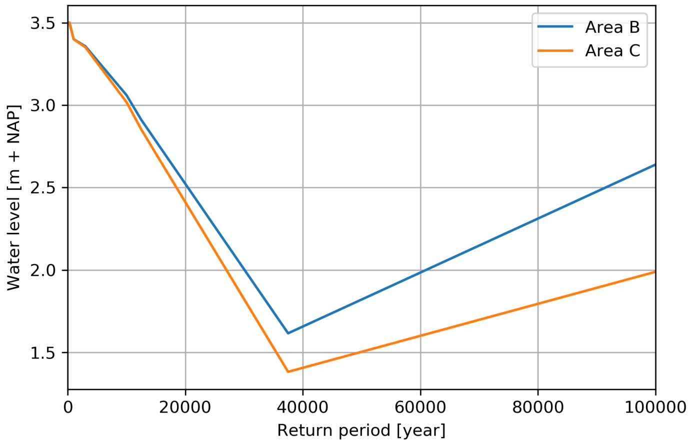


Figure 4.12: Water depth inside basin per return period due to overtopping, including the already present water, reference year 2045

Waves Also, waves can form inside the basin. Since the basin is closed off from the sea, the waves can only be generated locally in the basin due to the wind. Using the Bretschneider formulas shown in Equations 4.2 and 4.3 an estimation of the wave height and period can be made.

$$\tilde{H} = \tilde{H}_{\infty}\left(tanh(0.343\tilde{d}^{1.14}) \cdot tanh\left(\frac{4.41 \cdot 10^{-4}\tilde{F}^{0.79}}{tanh(0.343\tilde{d}^{1.14})}\right)\right)^{0.572} \tag{4.2}$$

$$\tilde{T} = \tilde{T}_{\infty}\left(tanh(0.10\tilde{d}^{2.01}) \cdot tanh\left(\frac{2.77 \cdot 10^{-7}\tilde{F}^{1.45}}{tanh(0.10\tilde{d}^{2.01})}\right)\right)^{0.187} \tag{4.3}$$

In which:
$\tilde{H}$ = $\frac{g \cdot H_{m0}}{U_{10}^2}$
$\tilde{T}$ = $\frac{g \cdot T_p}{U_{10}^2}$
$\tilde{d}$ = $\frac{g \cdot d}{U_{10}^2}$
$\tilde{F}$ = $\frac{g \cdot F}{U_{10}^2}$
$\tilde{H}_{\infty}$ = dimensionless wave height at deep water = 0.24
$\tilde{T}_{\infty}$ = dimensionless wave period at deep water = 7.69
F = fetch [m]
H_{m0} = significant wave height [m]
T_p = peak wave period [s]
d = water depth [m]
U_{10} = wind velocity at an altitude of 10 m [m/s]

The exact values of the wave height and wave period will be determined per relevant failure mechanism in the next section.

4.2.2. Assessment inner dike

In this section, the inner dike will be assessed on its relevant failure mechanisms. These relevant failure mechanisms are erosion of the crest and/or inner slope due to overtopping, macro-instability, erosion of the outer slope and grass cover sliding (see Section 3.3).

In order to perform a proper assessment, the role of the inner dike within the Twin Dike system has to be known. From Section 4.1 it is known that overtopping and erosion of the outer slope are governing mechanisms for the outer dike (additionally failure of the structure, but this is deemed outside the scope of this research). Overtopping is not able to damage the inner slope of the outer dike for the required return periods, but it causes an increase in water level in the basin in between the two dikes. This water will cause a load on the inner dike. Section 4.1 also showed that a regular dike at this location has to be safe against overtopping for 1/37,500 per year conditions, and for erosion of the outer slope for 1/180,000 per year conditions. This means that overtopping on the outer dike, and all the consequent failure mechanisms for the inner dike may not lead to a failure in a 1/37,500 per year condition. Figure 4.13 shows a schematic representation of this concept.

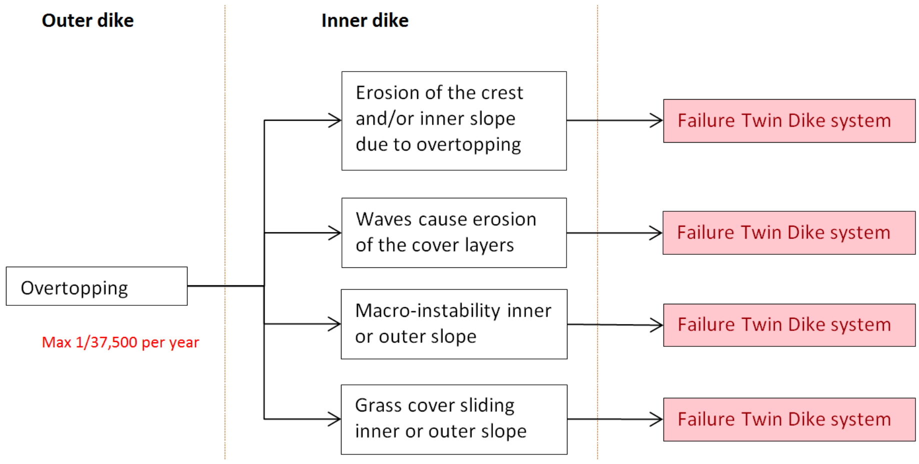


Figure 4.13: If overtopping occurs at the outer dike, the inner dike can consequentially fail due to all the relevant mechanisms of the inner dike. The Twin Dike has to be safe for 1/37,500 per year conditions for overtopping, taken into account the failure of the inner dike.

The paragraphs below will assess if the relevant failure mechanisms of the inner dike are able to withstand loads caused by overtopping volumes of the outer dike. The effects of erosion of the outer slope of the outer dike on the loads of the inner dike will be assessed in the next chapter.

Erosion of the crest and/or inner slope due to overtopping Overtopping occurring at the inner dike is dependent on the direction of the wind (and the consequent fetch), the wind velocity and the amount of water present in the basin. These factors determine the wave characteristics in the basin and thus the resulting overtopping volume. Every wind direction has a certain probability of occurrence, and for each wind direction, the probabilities of occurrence of specific wind velocities differ. Caires (2009) performed an analysis on the occurring wind conditions for multiple locations in the Netherlands. The closest location to the Twin Dike project where the analysis was performed is Lauwersoog. The analysis was performed for conditions with a return period up to 10,000 years. Table 4.6 shows the maximum wind speed and fetch (northern basin) per wind direction in a 1/10,000 year situation at Lauwersoog. Note that only directions between north, east and south are included. These are directions such that the wind blows in direction of the inner dike, otherwise the wind is directed to the sea and not to the inner dike. Additionally, Table 4.6 shows the resulting wind set-up, wave heights, peak wave periods and overtopping volumes which occur at the inner dike for these situations. Wave heights and periods were calculated using the Bretschneider equations (Equations 4.2 and 4.3). The wind set-up is calculated with Equation 4.1. As depicted in Figure 4.12, the water depth in a 1/10,000 year situation is 3 m + NAP. The resulting overtopping volumes were calculated with Hydra-NL (see Appendix B.1).

Table 4.6 shows that the overtopping volumes with a return period of 10,000 years is lower than 1.8 l/s/m for all situations. As found out in Section 4.1.2, overtopping volumes below 15 l/s/m will not damage the slope, provided it is of good quality. These conditions will, therefore, most probably not lead to

Direction	N	NNE	ENE	E	ESE	SSE	S
Fetch	1000	700	500	400	500	700	1000
Wind speed [m/s]	30	26	26	24	22	24	26
Wind set-up [m]	0.06	0.03	0.02	0.02	0.02	0.03	0.05
Wave height [m]	0.77	0.56	0.49	0.40	0.40	0.52	0.66
Peak period [s]	2.70	2.27	2.07	1.88	1.91	2.19	2.50
Overtopping [l/s/m]	1.8	0.2	< 0.1	< 0.1	< 0.1	0.1	0.8

Table 4.6: 1/10,000 per year wind speed per direction at Lauwersoog (Caires, 2009) and the consequent wind set-up, wave height, wave period and overtopping volume

damage to the inner dike.

As mentioned before, the inner dike has to be able to withstand 1/37,500 per year conditions due to the loads in the basin resulting from overtopping of the outer dike. Because of this, it also has to be checked if the inner dike can withstand conditions with return periods which are higher and lower than 10,000 years.

At return periods less than 10,000 years, the water level in the basin rises due to the closure regime. This can have a negative effect on the overtopping volumes. The worst situation in Table 4.6 is a fetch of 1000 m with a wind speed of 30 m/s. Quick calculations with Hydra-NL show that overtopping volumes will be higher than 15 l/s/m when the depth is more than 3.4 m. However, according to Table 4.5, a depth of 3.4 m is only reached in a 1/2,000 per year situation. Then the wind speeds have dropped to approximately 28 m/s (Caires, 2009). This results in a lower wave height as calculated with the Bretschneider formulas, which results in overtopping volumes less than 15 l/s/m. This is also the case for a water depth of 3.5 m + NAP and the corresponding wind speed, which is the largest water depth reached in the basin.

For return periods higher than 10,000 years, the wind velocity is unfortunately not available in Caires (2009). But, assuming a fetch of 1000 m, water depth of 3 m + NAP and a wind speed of 40 m/s the resulting overtopping volumes are 4.9 l/s/m. The wind speed is extremely exaggerated and is most probably not reached for return periods up untill 37,500 years. Also, the water level reduces to less than 3.0 m + NAP after a return period of 10,000 years (see Figure 4.12). This will lead in even more reduced overtopping volumes. It is, therefore, safe to assume that the critical overtopping volume of 15 l/s/m is never reached for the relevant return periods.

The results are even very conservative. All the results assume the northern basin (area B), fetches in basin C are even less and result in even less overtopping. The inlet structure is located in basin C, so these water depths will only be reached in this area. Area B will probably have even lower water depths. Also, the dominant wind direction for overtopping over the inner dike differs for overtopping over the outer dike. The past paragraphs showed that the wind needs to come from the north for maximum overtopping over the inner dike. The largest overtopping volumes over the outer dike occur when the wind originates from the north-west. Finally, the maximum water depths inside the basin are reached after the peak of the storm. At that moment the wind speeds have most probably reduced compared to the wind speeds used in the calculations above. Concluding, the inner dike satisfies the safety requirements for erosion of the crest and/or inner slope due to overtopping volumes over the outer dike.

Erosion of slope Erosion of the outer slope is also dependent on the wave height. Since the inner dike is covered entirely by grass, wave impact is governing over wave run-up. See Section 4.1.3. Using the resistance-duration curves as used by *BM Gras Buitentalud* (see Appendix B.3), it is possible to estimate how long a grass cover is able to resist a certain wave height. The maximum wave height in

Table 4.6 is 0.77 m for a return period of 10,000 years. A closed sod grass slope is able to withstand this wave height for almost 19 hours. The peak of a storm will however not last this long.

In more extreme cases it is hard to make an estimate of the erosion resistance of the slope. Wind velocities for higher return periods are not available. If again a fetch of 1000 m and an extremely exaggerated wind speed of 40 m/s is assumed, waves are able to form in the basin up to a height of 1.04 m. In this case, a grass slope will be able to withstand this for approximately 7 hours. This can pose a problem, however, a peak of a storm is likely to last for less than 7 hours. Again, as mentioned in the previous paragraph, the calculations are very conservative. Fetches of basin B were assumed in combination with water levels of basin C. Water levels in basin B are actually lower. In basin C the fetches are smaller than in basin C. These aspects will reduce the occurring wave height. Concluding, the inner dike satisfies the safety requirements for erosion of the outer slope due to overtopping volumes over the outer dike.

Macro-instability Since there can be water inside the basin during a storm, the inner dike is assessed on macro-instability. The computer program *D-Stability* is used to assess the dike for this failure mechanism. See Appendix B.6 for more information about this program. The program requires the subsoil layers and the dike geometry as input. Given a certain water level, the phreatic line can be determined and the corresponding factor of stability is calculated. The dike geometry and subsoil layers are respectively obtained from van der Wijk (2018) and Leusink (2018). The daily condition on the water side (the basin) is a water level of 1 m + NAP, approximately the maximum water level reached by the tide. The polder water level is approximately -1 m + NAP (Waterschap Noorderzijlvest, n.d.-c). The water level already present in the middle of the body of the dike is assumed at 1/3 of the total height of the dike: approximately 1.33 m + NAP. The evaluation is performed for water levels ranging from 0 m + NAP to 4 m + NAP in steps of 1 m, using the UpliftVan model.

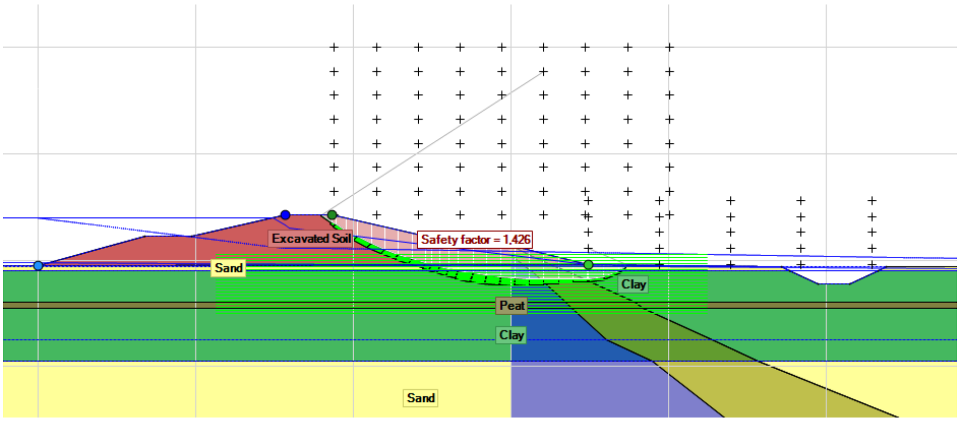


Figure 4.14: Overview of the governing stability calculation

The governing calculation is with profile *Omme-CPTU8 DWP1* (van der Wijk, 2018) with a water level of 4 m + NAP. The resulting stability factor is 1.426. Figure 4.14 depicts the calculation in *D-Stability*. Using Equation 4.4 from Ministerie van Infrastructuur en Milieu (2017b), the stability factor can be translated to a probability of failure per year.

$$P_{f;i} = \Phi\left(-\frac{\frac{F_{d;i}}{\gamma_d} - 0.41}{0.15}\right) \tag{4.4}$$

In which:

$P_{f;i}$	=	Failure probability of scenario i [1/year]
Φ	=	Standard (cumulative) normal distribution [-]
$F_{d;i}$	=	Stability factor for scenario i [-]
γ_d	=	Model factor (1.06 for UpliftVan) [-]

This formula is not meant to be used in this situation since, in this case, there is not a single primary dike. It is, however, used because it gives insight in the probability of failure due to macro-instability of the inner dike.

Using the above mentioned equation the resulting probability of failure for this case is $2.26 \cdot 10^{-10}$ per year. This is very small. For a complete overview, the probability of failure has to be multiplied by the probability of occurrence of this particular situation. This has to be done for all possible situations. Figure 4.12 shows that the probability of reaching a water level of 4 m + NAP due to overtopping is less than 1/100,000 per year. For cases with water levels lower than 4 m + NAP, the resulting probability of failure will be even lower. Multiplying the probability of failure with the probability of occurrence will still lead to negligible failure probabilities. The inner dike thus satisfies the safety requirements for macro-stability due to overtopping over the outer dike.

Sliding of the grass cover slope Sliding of the grass cover on the outer slope is not relevant if the core of the dike consists of clay (Ministerie van Infrastructuur en Milieu, 2017b). Sliding of the grass cover on the inner slope can, however, pose a problem. This can be checked with the Edelman and Joustra criteria depicted in Equation 4.5 (Ministerie van Infrastructuur en Milieu, 2018).

$$SF_{EJ} = \left[\frac{1}{\gamma_d \cdot \gamma_n}\right] \cdot \frac{\frac{tan\phi'}{\gamma_{m,\phi}}\left(\frac{\rho_g}{\gamma_{m,\rho}} \cdot g \cdot cos\alpha - \frac{\rho_w}{\gamma_{m,\rho}} \cdot g \cdot cos\alpha\right) + \frac{c'}{\gamma_{m,c} \cdot d}}{\frac{\rho_g}{\gamma_{m,\rho}} \cdot g \cdot sin\alpha} \tag{4.5}$$

In which:

SF_{EJ} = Safety factor
$tan\phi'$ = Tangent of effective angle of repose [°]
$\gamma_{m,\phi}$ = Safety factor for $tan\phi$ (=1.1) [-]
c' = Effective cohesion [Pa]
$\gamma_{m,c}$ = Safety factor for c' (=1.25) [-].
ρ_g = Volumetric weight wet soil [kg/m³].
ρ_w = Volumetric weight water [kg/m³]
$\gamma_{m,\rho}$ = Safety factor volumetric weight (=1.0) [-]
α = Slope angle [°]
g = Gravitational acceleration [m/s²].
d = Layer thickness [m]
γ_d = Model factor (=1.1) [-]
γ_n = Damage factor (=1.1) [-]

According to Ministerie van Infrastructuur en Milieu (2018), the first metre of cover is eligible for sliding. The slope of the dike is 1:2.5. Assuming exaggerated values for clay (density 2,000 kg/m³, cohesion 10,000 Pa, angle of repose 20 degrees) leads to safety factors above 1. The inner dike thus satisfies the safety requirements for sliding of the grass cover on the inner slope.

4.3. Conclusion

The results above show that the outer dike meets the safety requirements for most failure mechanisms. The dike was assessed in the third assessment round and was reassessed recently using current assessment instruments. The dike did not meet the required safety norms for the failure mechanisms of overtopping, erosion of the outer slope and macro-instability of the outer slope. Assuming that the failure mechanisms that did meet the required safety norms in the original assessment still meet the safety norms, overtopping, erosion of the outer slope and macro-instability are the governing failure mechanisms of the outer dike. Since an additional berm is constructed to provide the needed safety, also macro-instability is assumed not to be a governing failure mechanism. This can however not be checked since a detailed design of the to be constructed berm is not yet available. This leaves overtopping and erosion of the outer slope as remaining governing mechanisms.

The Twin Dike system has to be able to withstand failure due to overtopping for return periods of 37,500 years. The analysis showed that overtopping volumes are low enough such that they do not cause erosion of the crest and/or inner slope. This was the case for the current, as well as the future (2045) situation. However, this only holds if the grass is of good quality. Overtopping volumes itself cause an increase of the water level in the basin of around 0.5 m for return periods of 37,500 years. It is interesting to see that the increase of water level in the basin due to overtopping is relatively low. These volumes would most probably not lead to a flood in this area if the inner dike was not present. This would suggest that the construction of the inner dike is not necessary for the counteraction of overtopping volumes.

Erosion of the outer slope, however, is a very concerning mechanism. For return periods less than 100 years the slope already fails according to the assessment instruments. A regular single dike has to be able to withstand conditions with a return period of at least 180,000 years.

Also, a structure is present in the outer dike. This structure is a vital link in the flood safety of the Twin Dike system. If the structure fails to close, the inner dike will not be able to deal with the loads. The water level at sea will level with the water level in the basin due to the large size of the inlet structure. A seawater level which has an average return period of 10 years is higher than the crest of the inner dike. The closure reliability of the structure, therefore, needs to be sufficient. However, due to the scope of this research, the structure will not be elaborated in more detail in this report.

The inner dike does not have any concerning failure mechanisms resulting from overtopping water over the outer dike. Waves generated in the basin, in combination with the closure regime of the structure and overtopping volumes cause negligible loads on the inner dike. Additionally, the water level in the basin can be further reduced by modifying the closure regime of the structure.

The loads on the inner dike due to erosion of the outer slope of the outer dike were not yet be assessed. The assessment of the outer dike only includes the erosion resistance of the grass cover and the 0.5 m of clay underneath the grass. This does not tell anything about the influence of the loads on the inner dike. For this reason, the effects of erosion of the outer slope of the outer dike will be elaborated in more detail in the next chapter.

5 Detailed Assessment Erosion

Chapter 4 showed that a large part of the outer dike of the Twin Dike project does not meet the safety requirements for the failure mechanism erosion of the outer slope. It is, therefore, interesting to take a deeper look into this failure mechanism and the influence it can have on the loads on the inner dike. Without erosion, the only loads acting on the inner dike were generated by overtopping water over the outer dike. The event tree in Figure 4.13 was evaluated. The overtopping volumes were not large enough to cause a failure of the inner dike, see Section 4.1.1. But, when taking into account erosion, the hydraulic loads inside the basin can change. The fault tree shown in Figure 5.1 will be evaluated. Section 4.1 showed that a regular dike at this location has to be safe for the failure mechanism erosion of the outer slope for 1/180,000 per year conditions. This means that erosion of the outer slope of the outer dike, and all the consequent failure mechanisms for the inner dike may not lead to a failure in a 1/180,000 per year condition.

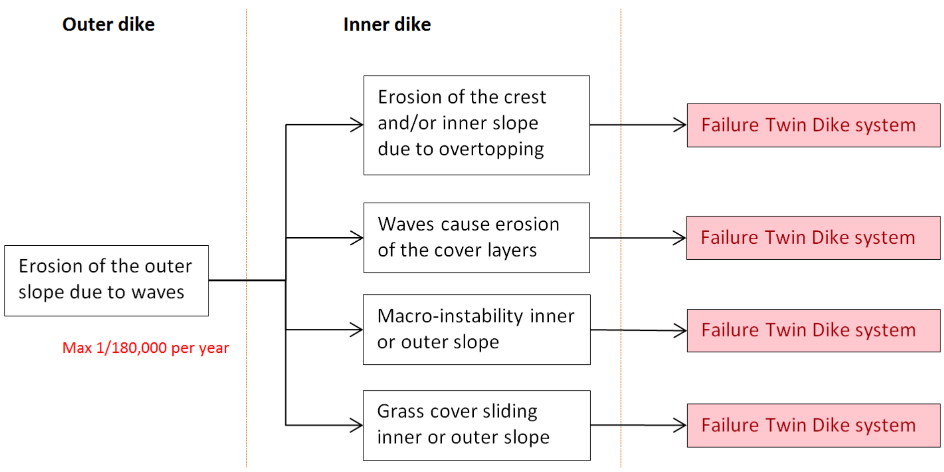


Figure 5.1: If erosion of the outer slope occurs at the outer dike, the inner dike can consequentially fail due to all the relevant mechanisms of the inner dike. The Twin Dike has to be safe for 1/180,000 per year conditions for erosion of the outer slope, taken into account the failure of the inner dike.

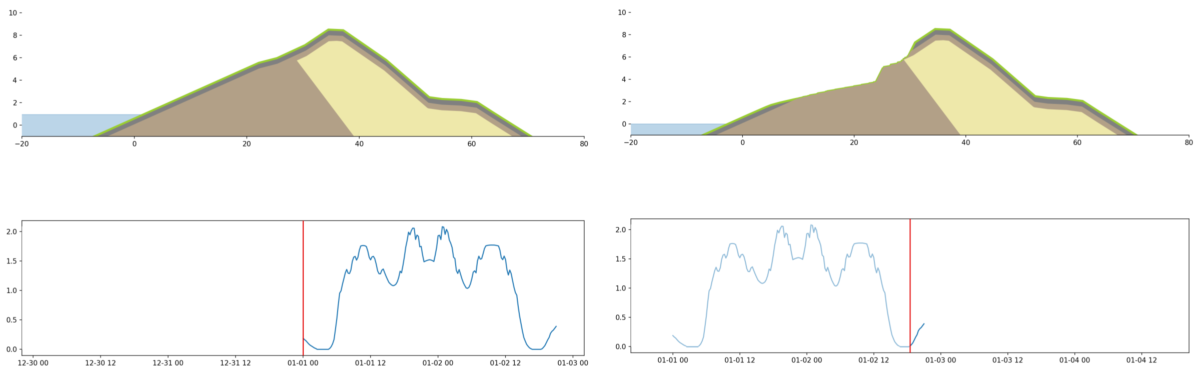
This chapter will use a prototype model to further assess the erosion of the outer slope due to wave attack. First, the model and its input will be discussed. Next, the results and the consequent impact on the safety of the Twin Dike system will be analysed.

5.1. Erosion model input

A prototype erosion model developed by HKV and Deltares is used to investigate if the occurring hydraulic loads cause erosion to the outer dike and possibly increase the amounts of water entering the basin in between the two dikes. Given the geometry of the dike, and the hydraulic loads acting on this dike over time, erosion- and overtopping volumes are calculated per specified time step.

The geometry of the dike was discussed in Chapter 3. The outer shape of the dike is obtained from drawings of the water board, see Appendix D. The inner composition is an old clay sea dike, which coincides with the outer slope of the current dike. The crest of the old sea dike is located at 5.75 m + NAP. The rest of the dike consists of a sand body, covered with a 1 m clay layer with a grass cover.

The hydraulic load inputs are wave characteristics and water levels acting on the dike over time. A representative storm for a certain return period has to be given as input. This means that again, only one return period at a time can be calculated. The representative storms were also used as input for *BM Gras Buitentalud* in Section 4.1.3, and can be used again here. Per time step, the model uses the water level and wave characteristics to calculate if damage to the grass revetment occurs. After this has occurred, the dike can start to erode. The model also calculates the overtopping and erosion volumes per time step. The overtopping is calculated using the same method as Hydra-NL (see Appendix B.1). Erosion is calculated using various empirical formulas. For more details about the background of this model see Appendix B.4. Additionally, the model is also able to visualize the erosion of the dike over time. Figure 5.2 shows an animation from the model. The left figure shows the situation at the beginning of a calculation. The upper left part shows how the profile looks like and the lower left part shows the wave height development over time. The right figure shows the same situation at the end of the calculation: the storm has eroded part of the dike.



(a) Situation at beginning of calculation

(b) Situation at end of calculation

Figure 5.2: Animation from erosion model depicting the erosion state of a dike (ID 3, return period 6,000 years, reference year 2045)

Erosion can also be more severe and cause a breach of the dike. Figure 5.3 shows the situation at the end of the calculation due to a storm with a larger return period. It can be seen that the dike is largely eroded.

The model is run for two reference years, namely, the current situation and the situation at the end of the period (2045). The same combinations of output locations and profiles as depicted in Table 4.1 are used. Because the calculations are time-consuming, only a closed sod quality is considered in these calculations. The impact of choosing either a closed or open sod quality is assessed in the sensitivity analysis in Chapter 6. Instead of nine return periods, the model is run with 21 different return periods. This is done to get more detailed results. The total amount of calculations thus becomes 315. For all the calculations the total overtopping- and erosion volume over a storm (m^3/m) can be obtained. The full results can be seen in Appendix C.3.

From the animations, it can also be seen when a breach occurs. The occurrence of a breach has been defined as a total amount of overtopping- or erosion volume being exceeded. If the total amount of overtopping exceeds 10.000 m^3/m, or the total amount of erosion volume exceeds 40 m^3/m, a breach has occurred. These amounts have been verified with the animations and give correct answers. With this information, it can be seen at what return periods the hydraulic loads are such that the dike was

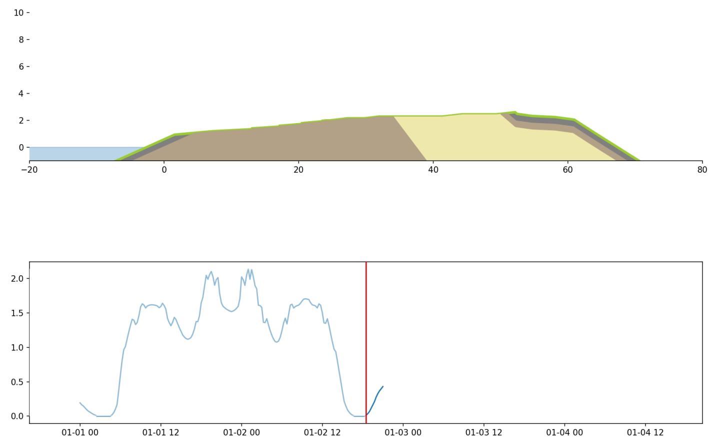


Figure 5.3: Animation from erosion model depicting a breach (ID 3, return period 8,000 years, reference year 2045)

breached.

5.2. Results

The model showed that minor erosion has only little influence on the overtopping volumes. It is therefore decided to look at what return periods breaches occur. A breach, namely, has a large influence on the overtopping volumes. Figure 5.4 and 5.5 respectively shows the results for the current situation and the 2045 situation. It depicts the intervals for which a breach in the outer dike occurs. Only the interval can be depicted, and not the actual probability of failure. This is because the model requires a simulated storm as input. If for a certain return period a breach does not occur, and for the next return period a breach does occur, it is known that a breach occurs for a return period within this interval.

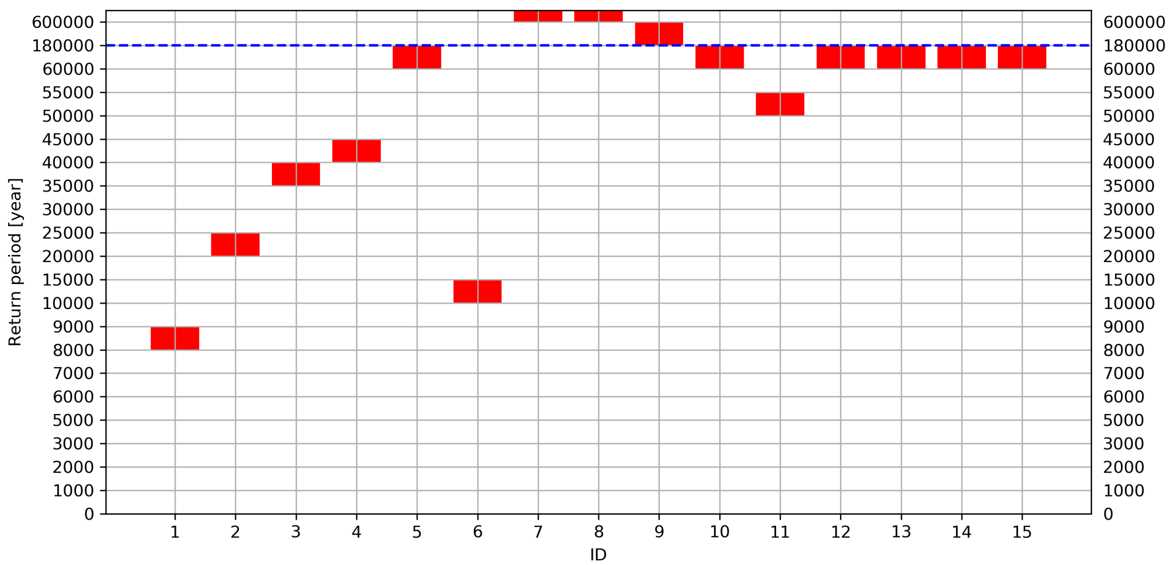


Figure 5.4: Average return period interval for which a breach occurs in the outer dike (current situation). The blue line show the required safety level for this mechanism were this a regular single dike.

When comparing Figures 5.4 and 5.5 with Figures 4.9b and 4.9d it can be seen that return periods for which a failure occurs are much less frequent in Figure 5.4 and 5.5 (the red bars are located 'higher' in this figure). This means that, according to this model, these dike sections are actually safer than first estimated in Section 4.1.3. This can be explained by the fact that residual strength is taken into

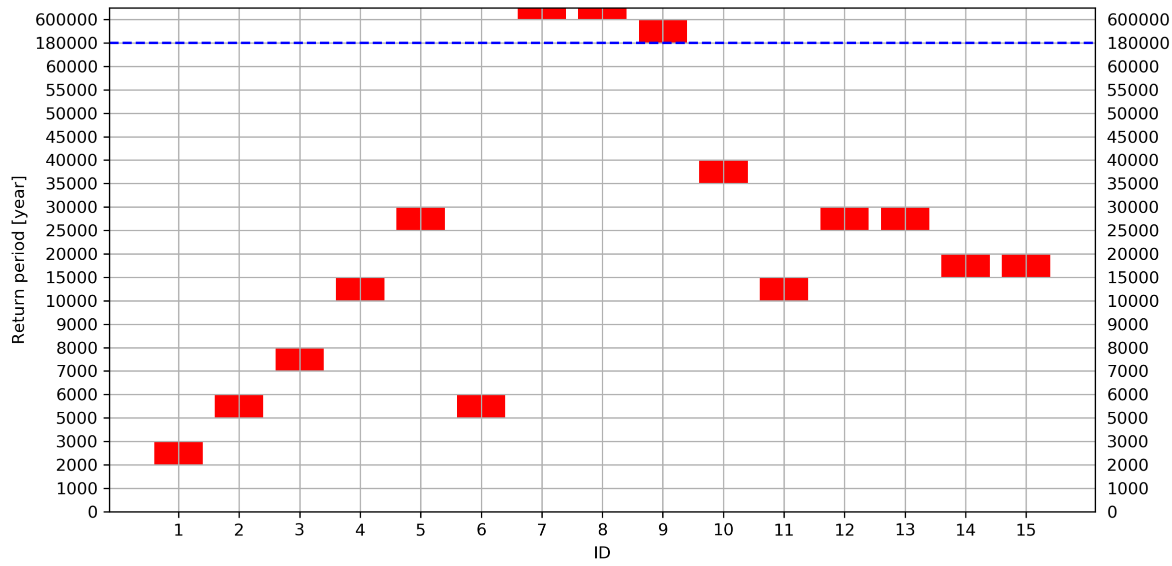


Figure 5.5: Average return period interval for which a breach occurs in the outer dike (reference year 2045). The blue line show the required safety level for this mechanism were this a regular single dike.

account in this chapter. In section 4.1.3 a slope did not meet the required safety level if the grass cover and the first 0.5 m of the underlying clay layer were eroded. In this chapter, a failure occurs if a breach occurs. Additional to the grass cover and 0.5 m underlying clay layer, most of the rest of the dike body also has to erode before a failure occurs. This results in failure intervals sometimes being more than a factor 10 less in this chapter compared to Section 4.1.3.

Besides the fact that failure intervals are a lot more favourable now than in Section 4.1.3, they are still relatively frequent. A regular dike at this location has to be able to withstand return periods of 180,000 years for this failure mechanisms (see Section 5.1). Figure 5.5 shows that, for reference year 2045, at 12 of the 15 sections a breach occurs at return periods less than 180,000 years. For a regular dike, this means that the required safety norm for this mechanism is not met. However, in this case, an inner dike is present. It has to be analysed if the inner dike is able to cope with the loads caused by a possible breach.

Figure 4.7 shows the shape of the water level course over the duration of a storm. It has three distinctive peaks, a major one at the middle of the storm and two minor ones before and after. The results of the erosion model showed that in all the cases where breaches occurred, this always happened on or before the major middle peak. The model also showed that during the third (minor) peak, water overflowed the residual profile of the outer dike. Assuming a conservative breach width of 20 m, the duration and amounts of water were always sufficient to level the water level inside the basin in between the two dikes with the water level at sea. This means that the water level at this third minor peak is governing for the safety of the Twin Dike system. If it is near or more than 4 m + NAP , the inner dike will most probably not be able to provide the required safety since the crest level of the inner dike is 4 m + NAP.

The breach with the lowest return period (so also with the lowest water levels) occurs at ID 1 for a return period of 3,000 years in reference year 2045. In this situation, the water level at the third (minor) peak is 3.77 m + NAP. This leaves only 23 cm below the crest of the inner dike. The wave heights during this peak are 1.78 m, the spectral wave period is 4.62 s and the wave direction is 17.9°. Hydra-NL's dam- and foreshore module (see Appendix B.1) can be used to perform a quick calculation on the expected overtopping over the inner dike. The residual profile is assumed to function as a breakwater. The area in between the two dikes is assumed to function as a foreshore. The height of the residual dike profile is 3 m + NAP, comparable to Figure 5.3). The results are overtopping volumes over the inner dike in the order of hundreds of l/s/m. This is way above the safe value of 15 l/s/m. The inner dike cannot withstand this. For all the other IDs and return periods in reference year 2045 for which breaches occur, water levels and waves are even higher. This causes even higher overtopping volumes over the inner dike. A total system failure thus occurs for the same return period intervals as depicted in Figure 5.5. For this failure mechanism, the inner dike does not provide any additional safety in addition to the outer dike. The highest load location is ID 15 (reference year 2045). In order for the inner dike to experience overtopping volumes of less than 15 l/s/m at the required return period of 180,000 years, the height of the inner dike has to be at least 6.4 m + NAP.

For the current situation, the results are less negative. The construction height of the inner dike is 4.25 m + NAP, which is higher than the expected crest level in 2045. Still, this additional 25 cm will not change the conclusions. For situations where a breach occurs, the water levels and wave heights are high enough to cause overtopping volumes of 80 l/s/m and up. A total system failure will thus also occur in 12 of the 15 sections for return periods which are more frequent than required by the safety norms.

5.3. Conclusion

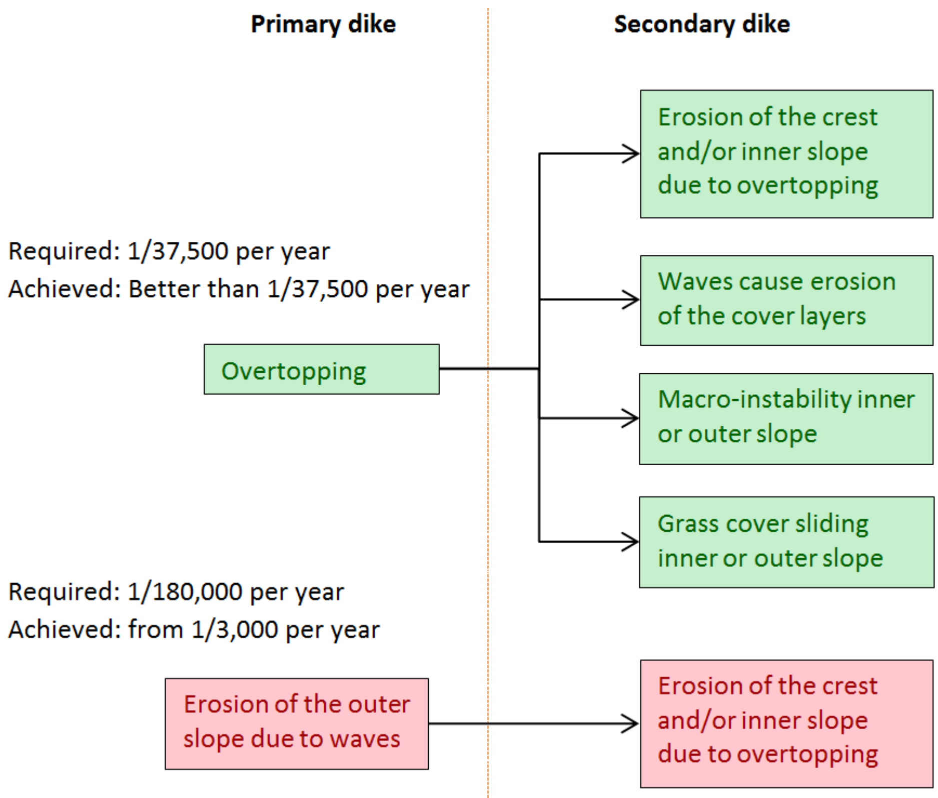


Figure 5.6: Safety verdict (reference year 2045) of the Twin Dike system taking into account the residual strength. Based on the situation that overtopping and erosion of the outer slope are the only governing mechanisms of the outer dike.

The model shows that the dike has quite some residual strength. Section 4.1.3 showed that failure of the outer slope already occurred from return periods of 100 years. Including the residual strength of the dike showed that a breach occurs at return periods from 3,000 years in reference year 2045. During a breach, so much water enters the basin that the inner dike is not able to prevent the hinterland from floods. The inner dike does not provide additional safety for this mechanism. This means that the required safety level for this failure mechanism is not met. The Twin Dike has to be able to withstand failure related to erosion of the outer slope for conditions with an average return period of 180,000 years. Now, breaches occur for return periods less than 180,000 years for 12 of the 15 dike sections. Figure 5.6 shows a schematic overview of the conclusions of the safety requirements. These are based on overtopping and erosion of the outer dike, the two governing failure mechanisms of the outer dike.

There can, however, be some discussion on the results of this model. The model uses empirical formulas to determine the erosion of the soil. The model is a prototype, which means that some of the coefficients which were used might not be optimally configured for this situation. The sensitivity of the results on these coefficients will be analysed further in the next chapter.

Also, Figure 5.2 shows the wave height development over time. During the peak of the storm, when the water levels are the highest, there is a dip in the occurring wave heights. These loads are calculated using Hydra-NL (see Appendix B.1). The dip in wave height occurs due to the calculation method of Hydra-NL. So-called hydraulic load combinations are calculated. For a specified return period and water level, the corresponding wave characteristics are found. For most ranges of water levels and return periods, the wind speed and water level are correlated. Meaning that for a certain return period, a higher water level also results in larger wave height. For higher ranges of water levels, their probability of occurrence is very low. The probability that such an event coincides with large wave heights gets smaller. In other words: for water levels that occur with a low probability, less probability is 'available' for large wave heights. This statistical phenomenon can be seen in Figure 5.2. This is of course not a realistic situation. Removing these dips manually is a lot of work since these loads are determined statistically. Smoothing the dips will lead to overestimated loads. Also, from personal communication with employees from the water board, it became clear that that observed wave heights near the shore are smaller than predicted by Hydra-NL for specific return periods. Another way has to be found to better represent the loads over time. This will also be analysed in the next chapter.

Furthermore, the model calculates the occurrence of a breach. In Section 2.5 it was mentioned that all the primary Dutch flood defences have a maximum allowable probability of causing a flood. A breach does not necessarily mean that a flood takes place. The results show that a breach sometimes occurs on or just after the peak of the storm. At this moment the highest loads have already passed. The amounts of water coming through the breach and over the inner dike might be low enough not to cause a flood. In order to completely assess the safety of this concept, it has to be analysed if an actual flood occurs in governing situations. This is deemed outside the scope of this research.

Finally, the model does not take into account 3D effects. The erosion is only calculated in 2D and extrapolated over the entire dike section. To know with more certainty how much water exactly enters the basin, a 3D breach growth model has to be used. This is however outside the scope of this research.

6

Sensitivity Analysis

The results obtained in the previous two chapters all contain uncertainties. Some uncertainties are more relevant for the outcome than others. This chapter will analyse which uncertainties may alter the conclusions. Next, a sensitivity analysis is performed to assess the impact of the uncertainties on the results in more detail.

6.1. Relevant uncertainties

In Chapter 4 it was concluded that erosion of the inner slope and/or crest due to overtopping, overtopping itself and erosion of the outer slope are governing failure mechanisms for the outer dike.

Erosion of the inner slope and/or crest did not pose a problem for the considered conditions. Also, overtopping volumes were not large enough to cause considerable loads on the inner dike. However, there is uncertainty in the calculated overtopping volume. Overtopping volumes are dependent on the wave characteristics, water level, dike geometry and model coefficients. Uncertainties in these variables can influence the amount of water which is coming over the outer dike. These uncertainties will, therefore, be assessed in this chapter.

Also, the assessment methods for the inner dike have uncertainties. However, the analysis showed that the relevant failure mechanisms were not able to damage the inner dike by a large margin. Therefore, it is first assessed how large the uncertainties of the loads on the inner dike due to overtopping are. Small differences in overtopping volumes can relatively easily be counteracted by altering the closure regime of the inlet structure. Uncertainties in the assessment of the inner dike are not considered for now.

The second relevant failure mechanism for the outer dike is erosion of the outer slope due to wave attack. This was assessed using *BM Gras Buitentalud* in Section 4.1.3, and using a prototype model in Chapter 5. Erosion of the outer slope of the outer dike is able to cause large loads on the inner dike. The erosion model is the same as *BM Gras Buitentalud* but additionally, it takes into account the residual strength. Because of this, only the uncertainties of the erosion model are discussed. The erosion model uses different input variables and formulas to determine the erosion of the dike body and the consequent overtopping volumes. One of the input variables for the model is a storm. The storm contains data on wave characteristics and water levels over its entire duration. As discussed in Section 5.3, the wave height input is generated by Hydra-NL and is not a good representation of real circumstances. There is also uncertainty in the different model factors, the dike geometry and soil parameters. These parameters can influence the results. Since erosion of the outer slope plays a dominant role in the safety verdict of the Twin Dike system, the above-mentioned variables are considered as relevant uncertainties.

6.2. Sensitivity analysis overtopping

In this section, the sensitivity of different parameters on the overtopping volumes will be assessed. Appendix B.1 shows that the calculated overtopping volumes are dependent on the wave conditions, water level, geometric parameters of the dike and a model coefficient. The outer geometry of the dike was determined with relative high certainty due to the drawings depicted in Appendix D. There is, however, uncertainty in the wave height, water level and the model coefficient. These parameters are preferably all assessed. Hydra-NL, however, does not easily allow to alter the model coefficient. A few quick calculations using Equation B.1 show that the impact on the results due to altering the model coefficient is lower than altering the wave characteristics and water level. Because of this, the model coefficient is not included in this analysis.

6.2.1. Set-up

The effect on the results by altering the wave parameters and the water level will be assessed. The first parameter for which the sensitivity will be analysed is the sea water level. This water level is uncertain because the exact value of the sea level rise is unknown. In this report, a sea level rise of 25 cm in the period from 2020 to 2045 was assumed. Haasnoot et al. (2018) predicts that values can reach up to 50 cm within this period. However, if it is assumed that the sea levels will continue to rise at the current rate, the sea level rise within this period will only be 5 cm (Baart et al., 2018). These extreme values will be used in the sensitivity analysis.

Also, soil subsidence is uncertain. In this report, soil subsidence of 20 cm was assumed for the period from 2020 to 2045. This was a safe value. NAM B.V. (2015) predicts that soil subsidence till 2080 will only have a maximal value of 10 cm in this region. Both values will be used in the sensitivity analysis.

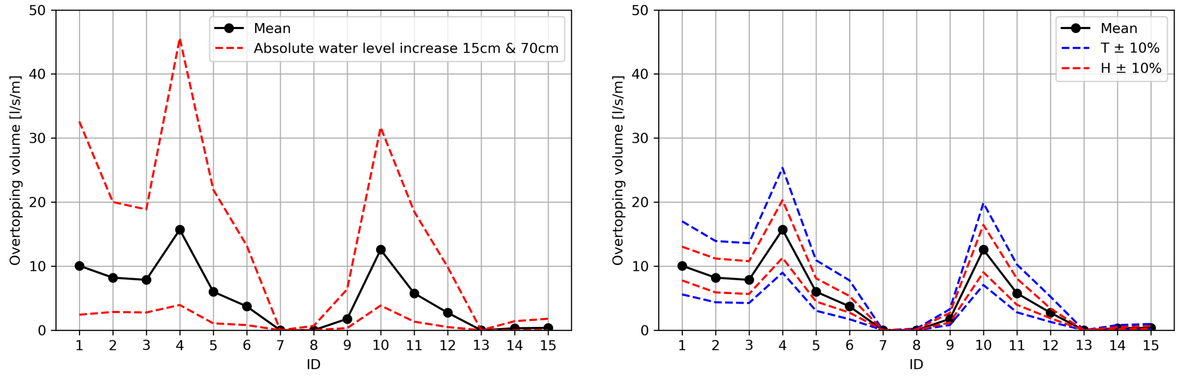
The Twin Dike has to be able to withstand conditions with an average return period of 37,500 years for overtopping at the end of the design life. Conditions with an average return period of 37,500 years with mean values for all parameters and reference year 2045 will, therefore, be used as a base run. This is slightly different than the run depicted in Figure 4.2b. The results depicted in that figure already contain safety factors to take into account the uncertainties. A total of seven cases will be assessed. Firstly, a base run with mean values for all parameters. Next, one case with maximum sea level rise and soil subsidence (20 + 50 = 70 cm) and another with minimal sea level rise and soil subsidence (5 + 10 = 15 cm). Additionally, four runs are performed to see what the influence on the results is of varying the wave height and period by $\pm$10%. An overview of the different cases that are going to be assessed is given in Table 6.1.

Variable	**Case 1**	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**	**Case 7**
Sea level rise [m]	0.25	0.50	0.05	0.25	0.25	0.25	0.25
Soil subsidence [m]	0.20	0.20	0.10	0.20	0.20	0.20	0.20
Wave height [m]	-	-	-	+10%	-10%	-	-
Wave period [s]	-	-	-	-	-	+10%	-10%

Table 6.1: Cases assessed in the overtopping sensitivity analysis

6.2.2. Results

The results of the overtopping sensitivity analysis are depicted in Figure 6.1. Figure 6.1a depicts the base run together with case 2 and 3. It can be seen that on some locations, overtopping volumes can get more than twice as large with extreme sea level rise. Figure 6.1b shows the overtopping volumes when the wave height and wave period are varied by 10%. It can be seen that the impact of the wave period on the overtopping volumes is larger than the wave height. However, the impact is still considerably lower than in the case of extreme sea level rise. The results show that the overtopping volumes are larger than 15 l/s/m in a few cases. This was the safe limit for which damage will not occur due to overtopping (see Section 4.1.1). This does not necessarily mean that the dike is not able to withstand this. Section 4.1.1 mentioned that slopes of good qualities can sometimes withstand overtopping volumes of 100 l/s/m (EurOtop, 2016). If the inner slope is of good quality it can possibly still withstand these overtopping volumes.



(a) Varying sea level rise and soil subsidence

(b) Varying wave parameters

Figure 6.1: Sensitivity overtopping volumes

It is also interesting to look at the increase in water level inside the basin in between the two dikes due to these overtopping volumes. As in Section 4.1.1, a peak storm duration of four hours is assumed. Also, the entire overtopping volume is assumed to be present over the entire dike section. The resulting increase in water level is depicted in Table 6.2. It can be seen that in the most extreme case water levels can increase up to 1.17 m. This is still relatively low. In basin B this water level is not able to damage the inner dike (see Section 4.2). In basin C the largest increase is only 0.60 m. Combined with the water already present in the basin the total water level is 1.5 m + NAP. This will not lead to considerable loads on the inner dike. Additionally, the closure regime of the inlet structure can relatively easily be adapted such that water levels in the basin will decrease.

	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**	**Case 6**	**Case 7**
Basin B	0.42	1.17	0.12	0.55	0.30	0.70	0.23
Basin C	0.18	0.60	0.04	0.25	0.12	0.33	0.09

Table 6.2: Increase in water depth [m] in the basins after a 4 hour storm for different cases

6.3. Sensitivity analysis erosion model

In this section, the sensitivity of the erosion model will be assessed. The entire erosion model consists of tens of different variables and model factors. For the sake of simplicity and time, not all of these parameters are going to be used in the sensitivity analysis. Below, an elaboration will be made for the relevant variables.

The model calculates erosion and overtopping volumes over time. Before erosion can occur, a grass cover section of the slope first has to fail. This failure can either occur due to wave impact or wave run-up. As discussed in Section 4.1.3, wave impact is governing for almost the entire area. Because of this, it is decided not to include the model parameters for run-up in the sensitivity analysis.

Model parameters are also used to calculate the overtopping- and erosion volumes. The formulas used for overtopping have been used for more than 15 years to determine overtopping volumes in Dutch flood defence assessment (TAW, 2002). Also, they have been validated with many experiments (EurOtop, 2016). The erosion volume formulas are, however, obtained from recent reports and researches and are not widely used for the assessment of Dutch flood defences (see Appendix B.4). Also, they have not been validated as extensively as the overtopping volumes. This means that the limitations and uncertainty margins are better known for the overtopping formulas. Furthermore, the eroded volume and consequent breach formation have a much larger effect on the overtopping volumes than the overtopping model parameters. The occurrence of a breach causes more than 100 times more water to come over the dike than in regular situations. The model parameters for overtopping influence the overtopping volumes by a maximum of 10%. Because of these reasons, it is decided not to include the overtopping model parameters in the sensitivity analysis, but only the erosion volume model parameters.

Besides the model parameters, a few other parameters are going to be assessed in the sensitivity analysis. They all have an effect on the erosion volume as well but relate to the shape and structure of the dike. Also, the sea level rise and soil subsidence as mentioned in Section 6.2.1 are included in the analysis. All the variables used in this sensitivity analysis are depicted in Table 6.3.

Parameter	Description	Unit
a	Resistance-duration curve coefficient	[m]
b	Resistance-duration curve coefficient	[1/h]
cm_{grass}	Model coefficient erosion grass and clay grass root	[-]
cm_{clay}	Model coefficient erosion volume clay	[-]
cm_{sand}	Model coefficient erosion volume sand	[-]
$cm_{terrace}$	Model coefficient erosion terrace depth	[-]
F_{sand}	Sand content in clay	[-]
h_{sea}	Absolute sea level rise	[m]
D_{clay}	Thickness of clay cover layer on dike	[m]
h_{clay}	Height of the original clay sea dike, see 3.6	[m + NAP]

Table 6.3: Parameters that are going to be assessed in this sensitivity analysis

Most parameters that are going to be assessed have a mean and standard deviation which was obtained from models and test results. The distributions, means and standard deviations of coefficients *a* and *b* (Klerk & Jongejan, 2016), cm_{sand}, $cm_{terrace}$ (Klein Breteler, Capel, Kruse, Mourik, & Kaste, 2012), cm_{clay} (Mourik, 2015) and cm_{grass} (Breteler, 2015) are given in Table 6.4.

Variable	Distribution	Mean	Standard Deviation
cm_{sand}	Normal	1	0.19
cm_{clay}	Normal	0.55	0.25
cm_{grass}	Normal	0.59	0.32
$cm_{terrace}$	Normal	1	0.15
a_{closed}	Log-normal	1.82	0.62
a_{open}	Log-normal	1.4	0.5
b_{closed}	Deterministic	-0.035	
b_{open}	Deterministic	-0.07	

Table 6.4: Parameter variability

6.3.1. Set-up

Performing a sensitivity analysis for all the 15 combinations of profiles and hydraulic output locations as mentioned in Table 4.1 would be very time-consuming. That is why the sensitivity analysis is only performed for one combination of dike profile and hydraulic output location, namely for ID 6. This combination of profile and output location is chosen because large erosion already occurs for this combination in relative low return periods. This means that results from the sensitivity analysis will be visible in this same range of return periods, making it easier to analyse the results.

Instead of the nine return periods used in Chapter 4, 22 return periods are used, see the vertical axis in Figure 6.8. This way the sensitivity of different parameters can be investigated in more detail. In this chapter, it is not of interest any more to look at different reference years, since the only difference was the sea level rise which is now incorporated in the sensitivity analysis itself. The analysis will only be performed for reference year 2045. The difference between closed and open sod qualities is also incorporated in the sensitivity analysis since the difference is implemented by using different values for model parameters a and b.

The sensitivity analysis will be performed by conducting one base run where all the parameters are equal to their mean values. In the runs in Chapter 5 the parameters were equal to their 95% value (a semi-probabilistic assessment). Next, all of the parameters mentioned in Table 6.4 will be varied by $\pm$ 1 and 2 times their standard deviation. Additionally, the thickness of the clay cover layer and the crest of the old sea dike will be varied. According to design drawings, the clay cover is 1 m thick and the crest of the old sea dike was located at 6 m + NAP when the last reinforcement was constructed in the 1970s. Previously in this report, it was assumed that the clay cover had a thickness of 1 m and that the crest of the old sea dike has settled for 25 cm to 5.75 m + NAP. However, no measurements have been made, so a certain degree of uncertainty exists. To assess the sensitivity of these uncertainties the sensitivity analysis is also performed with a varying clay cover thickness of $\pm$ 20 cm. The crest height of the old sea dike is varied by $\pm$ 25 cm. Also, an absolute sea level rise of 15 cm and 70 cm will be analysed next to the mean value of 45 cm (see Section 6.2.1). The sand content in the clay was assumed to be 0.4. This content does not affect the erosion rate up until values of 0.7. To assess the sensitivity also sand contents of 0.7, 0.8, 0.9 and 1.0 will be assessed.

Results are the overtopping- and erosion volumes over an entire storm for different return periods. By analysing the difference between the different runs, the sensitivity of different parameters can be assessed.

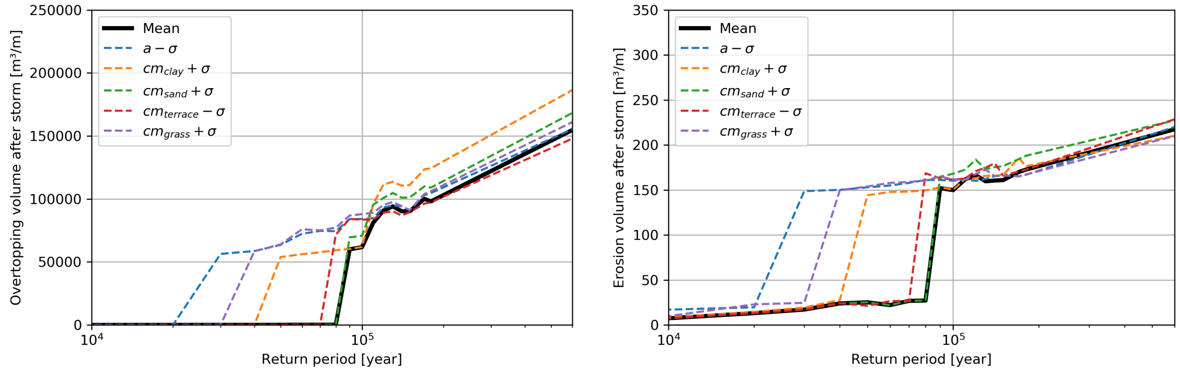
6.3.2. Results

Figure 6.2 shows the total amount of overtopping and erosion after a storm per return period. The black line indicates the base run where all the parameters have their mean value. The dashed lines are different scenarios where the specified coefficients are increased or decreased by their standard deviation in such a way that they have an increasing effect on the overtopping and erosion rates.

In most runs, also the one with the mean values, a clear jump can be seen where overtopping and erosion rates increase suddenly between two return periods. The jump indicates that the dike has breached during a storm. When a breach occurs, much more erosion and overtopping occurs than in a case without a breach. The further the dashed lines are away from the base run, the larger the sensitivity of these parameters on the results. If a dashed line is located above the black line, this indicates that for the same storm per return period, more overtopping and/or erosion occurs.

Of all the parameters assessed in Figure 6.2, coefficient a has the largest impact on the occurrence of a breach. It can be observed that a dike breach will occur at a lower average return period than in the base run (at a return period of 30,000 years instead of 90,000 years). cm_{grass}, cm_{clay} and $cm_{terrace}$ also have this effect but this is a lot less. Varying the sand erosion coefficient does not cause a breach in lower return periods. It can also be seen that for higher return periods the increase in total overtopping volume is influenced most by cm_{clay}.

The terrace coefficient has an interesting effect on the overtopping volumes. Increasing this coefficient

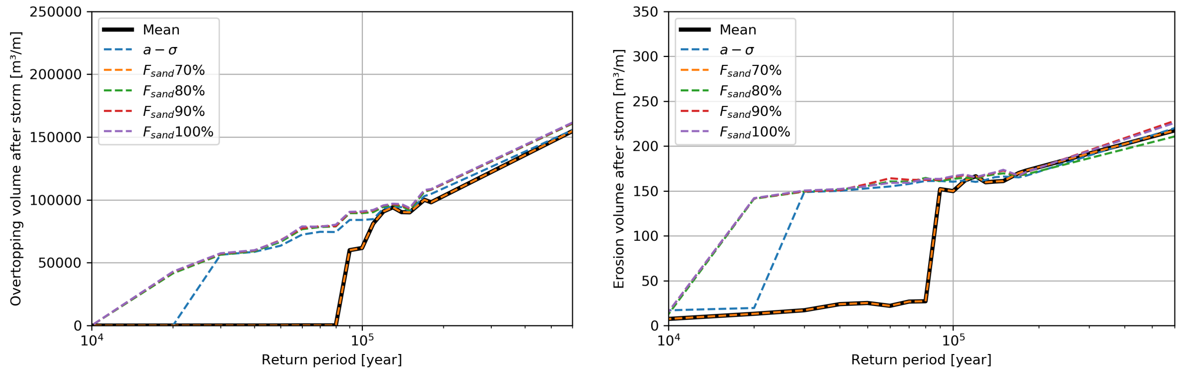


(a) Overtopping (b) Erosion

Figure 6.2: Sensitivity of soil model coefficients. Parameters increased or decreased with σ so that they have an increasing effect on the overtopping and erosion rates.

has the effect that a breach will occur at lower return periods. But at the higher return periods, it will decrease the total overtopping volumes. This can be explained in the following way. Per time step waves can erode a part of the dike. This part has a specific shape and volume. The terrace coefficient relates to the depth to width ratio of this erosion shape. Decreasing the terrace coefficient causes the erosion shape to become relatively wider which causes a breach faster. But after a breach has formed, the vertical erosion will go slower and decrease the overflow volumes.

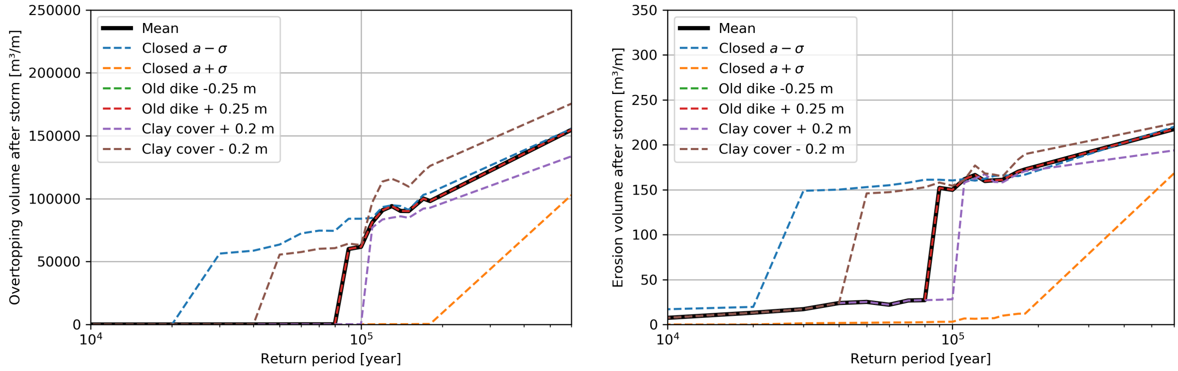
The calculations have also be performed by altering the parameters by 2 times their standard deviation, and in such a way that they have a decreasing effect on the overtopping and erosion volumes (changing +σ by -σ and vice versa). The results are similar: a, and cm_{grass} have the largest effects, followed by cm_{clay} and $cm_{terrace}$. The full results can be seen in Appendix C.4.



(a) Overtopping (b) Erosion

Figure 6.3: Sensitivity of the sand content

Figure 6.3 shows the sensitivity of the sand content parameter. As mentioned in 6.3.1, a sand content up to 0.7 does not affect the results. This can also clearly be seen in the figure. A sand content of 0.7 gives the exact same results as the base run (sand content 0.4). See Appendix B.4 for how the sand content is incorporated into the model. When the sand content is further increased, this has large effects on the results. For comparison, also the sensitivity of coefficient a is shown in the graph (since this was the most sensitive coefficient in the previous graph). Increasing the sand content has similar effects as decreasing coefficient a by one standard deviation. It is interesting to see that increasing the sand coefficient from 0.7 to 0.8 has large effects. But increasing it further to 1.0 only has a minor effect on the results. From this, it can be concluded that the sand content also has a large sensitivity, provided that the sand content has a possibility of reaching values above 0.7.



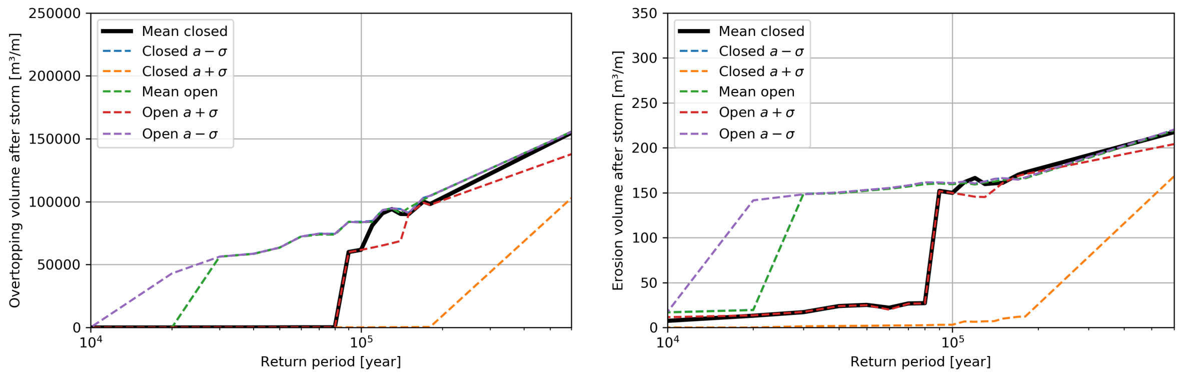
(a) Overtopping

(b) Erosion

Figure 6.4: Sensitivity of other dike geometry parameters

Figure 6.4 shows the sensitivity of several other dike parameters in which a certain uncertainty exists. For comparison again the sensitivity of coefficient a is shown. It can be seen that only the clay layer thickness has an effect on the results. The elevation of the old clay sea dike does not have an observable effect on the results. This can be explained by the fact that altering the height of the old clay sea dike only has a minor effect on 25 cm of the slope while increasing the thickness of the clay layer has an effect over the entire slope above the crest of the old sea dike. This stretch is most governing for the erosion of the entire dike body. The effect on the results of the clay layer thickness is not as severe as coefficient a.

Next, the effects of considering an open or closed grass sod are assessed. This can be done by altering the parameter distribution of coefficients a and b. The results are depicted in Figure 6.5. The previous figures showed results for closed sod qualities. This figure shows both results for open and closed sod qualities. The figures show that the results for an open sod quality are all within the range of a close sod quality. It is interesting to see that the open sod quality minus one standard deviation has the same results as a closed sod quality minus one standard deviation. That indicates that with those values the sod quality is so bad that when a certain wave height is reached the slope fails within a single time step, thus not further changing the results.

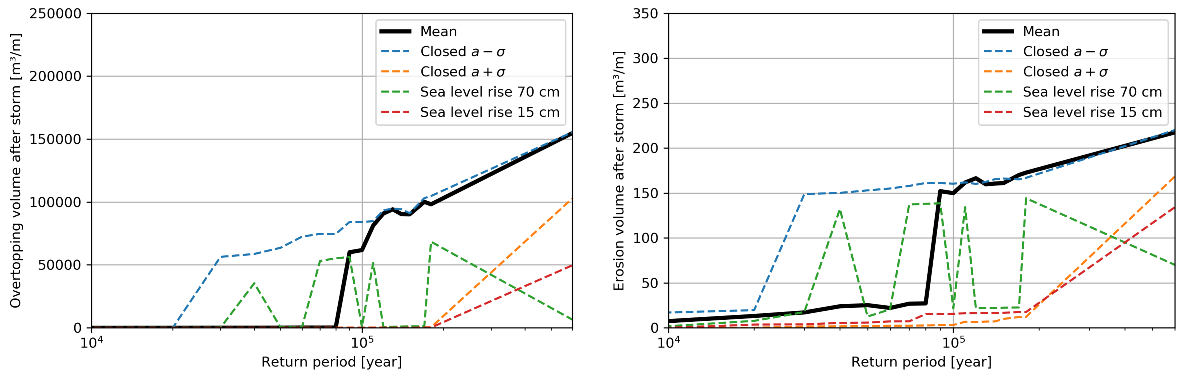


(a) Overtopping

(b) Erosion

Figure 6.5: Sensitivity of coefficient a and b

Finally, the effect of sea level rise is assessed. The results can be seen in Figure 6.6 and are rather strange. It can be seen that for an absolute sea level rise of 70 cm a breach occurs for lower return periods than for the base run. However, for higher return periods the breaches do not occur any more. This is due to the fact that Hydra-NL works with a combined chance for the occurrence of water levels and wave heights. This was explained in Section 5.3. The larger a water level for a specific return period, the lower the probability on a large wave height. This was already the case for the base run. With extreme sea level rise, these effects are even larger. The effects are so large that for higher water levels (and higher return periods) the wave heights get so low that they do not cause a breach any more.



(a) Overtopping (b) Erosion

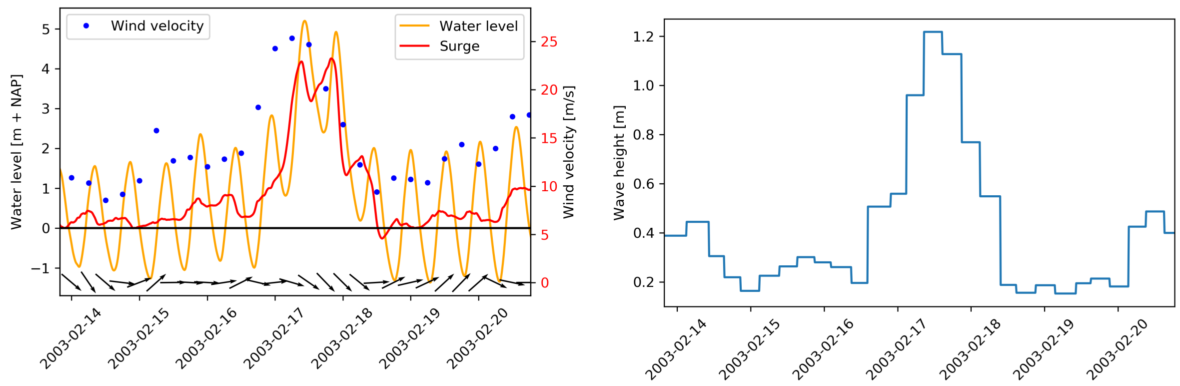
Figure 6.6: Sensitivity of absolute sea level rise

The figure shows that a breach occurs earlier for the extreme sea level rise scenario than for the base run. The effects for extreme sea level rise are less than for coefficient a.

6.4. Hydraulic loads

As mentioned in Section 5.3, the shape of the wave height over time generated by Hydra-NL is unrealistic. In this section, the effect of a more realistic wave height development over time is assessed.

From the European Centre for Medium Range Weather Forecasts (ECMWF) a dataset was obtained which contains simulated wind- speeds directions in the Wadden Sea area over a period of 6,500 years. From the KNMI a dataset was obtained which contains the corresponding water levels in this region. From this dataset, the most severe storms within this 6,500 year period are obtained. Figure 6.7a shows an example of the most extreme storm at Delfzijl which was obtained from this dataset.



(a) Water level and wind speed during the storm. The arrows indicate the wind direction. (b) Wave height during the simulated storm

Figure 6.7: Storm with a return period of 6,500 years generated with ECMWF data

Next, the corresponding wave characteristics have to be obtained. The SWAN dataset (see Appendix

B.2) which was used to determine the hydraulic loads in the Wadden Sea can be used to obtain the wave characteristics. Per combination of wind- velocity, direction and water level the wave characteristics can be obtained. This way, for each storm, the water level and wave characteristics over time are known. This is the input which is needed for the erosion model. The wave height development over time corresponding to the storm depicted in Figure 6.7a is depicted in Figure 6.7b. Now that all the parameters are known the erosion model can be run with this storm. This will be done for profile 79 since this is also the profile which was used in the rest of this chapter. The data used belongs to the current climatology. This means it is representative for the current situation and not for a future reference year.

The results are interesting. With this storm (the most severe storm in 6,500 years), the erosion of the outer slope is three times less than for a storm with a return period of 6,000 years generated by Hydra-NL. There is a little difference in the water level (just 0.5 m), but the wave height has large differences. The highest wave height reached in the simulated storm is 1.2 m, while the storm with an average return period of 6,000 years generated by Hydra-NL reaches values of 1.86 m. This has a significant impact on the erosion.

This is, however, for a single storm with an average return period of 6,500 years for the current situation. These results can be roughly extrapolated to reference year 2045 and for larger return periods using the same proportions as in the Hydra-NL results. For example, reference year 2045 experiences roughly 50% more erosion than in the current situation in the Hydra-NL results. When this is done, breaches still occur for return periods which are lower than stated in the safety norms. The usage of this source of data thus has a significant impact on the results. The overall conclusion that the safety norms are not satisfied, however, still holds.

6.5. Conclusion

In this chapter, the sensitivity of different parameters related to overtopping and erosion of the outer slope was analysed. The sensitivity analysis for overtopping showed that differences in sea level rise can have a large impact on the consequent overtopping volumes, less than varying wave characteristics. In some cases, overtopping volumes were more than three times as high. In the most extreme case, the increase in water level in the basin in between the two dikes due to these overtopping volumes was 1.17 m in the northern basin and 0.60 m in the southern basin. The inner dike is still able to resist these loads. The overtopping volumes can, however, damage the outer dike. Still, reports show that a dike can still be able to resist these overtopping volumes if the slope is of good quality. The influence of sea level rise thus does not influence the conclusions. The Twin Dike still satisfies the required safety norms for overtopping.

A sensitivity analysis was also performed on the erosion of the outer slope of the outer dike. Figure 6.8 shows a summary of the sensitivity of the different parameters assessed. From the figure, it is clear that coefficient a and cm_{grass} have the most impact on the results. On one hand, these factors can cause a breach to occur for lower return periods. On the other hand, they can cause that a slope will never fail (at least not on this scale). To a lesser extent cm_{clay}, $cm_{Terrace}$ and h_{sea} also have this influence, followed by F_{sand} and the other parameters.

Parameter a relates to the strength the top of the grass cover. If this is very strong, the grass cover will never fail and thus never exposing the core, never causing a breach. To gain more knowledge on this parameter in this project, the grass quality has to be inspected further. But to really get a good understanding, the slope has to be tested on wave impact. However, local poor grass quality can cause the waves to damage the grass and expose the core. Parameter cm_{grass} tells something about the strength of the clay where the roots of the grass cover are still present. For this parameter, the same applies as for coefficient a. To get a better local understanding of this parameter, test and analysis have to be performed. But there is always the uncertainty a weak spot exists.

The other parameters also influence the erosion and overtopping volumes. However, their sensitivity is a lot less. Also, a local dip in the clay or sand quality will not cause large consequences. It will

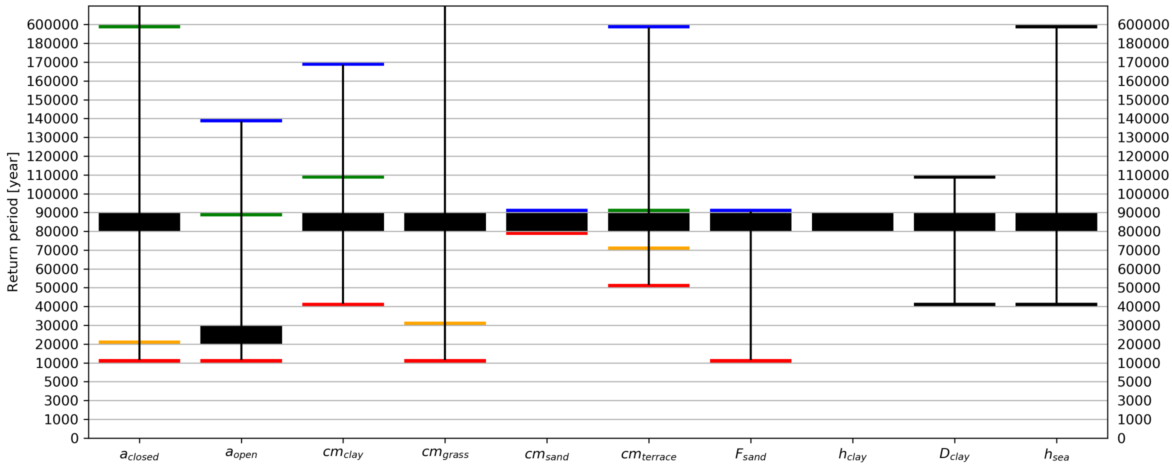


Figure 6.8: Return period of breach occurrence for different variations of various parameters

just cause that part to erode quicker, but if 20 cm later a strong core is again present it will most likely not have large consequences. For F_{sand} it is useful to know in what range the sand content is. In the previous chapter, it was based on a mean value of 0.4. But cases of 0.7 or even 0.9 are known along some of the Dutch dikes (Breteler, 2015). To be more certain of the erosion rate of the clay it will be very useful to know more about this parameter. Also, the height of the old sea dike and the clay layer thickness can be better estimated with a few cone penetration tests.

Finally, hydraulic loads influence the erosion rate significantly. Sea level rise affects the return period of breach occurrence significantly. However, the results get more unreliable with higher water levels due to the way how Hydra-NL determines the hydraulic loads (see Section 5.3). Also, from personal communication with employees from water board Noorderzijlvest, it turned out that observed wave heights next to the shore are smaller than predicted by Hydra-NL. Simulated data from the ECMWF was used to see the impact on the results by using another source for the hydraulic loads. Performing a run with the most severe storm in the 6,500 years dataset led to three times less erosion than a storm with an average return period of 6,000 years generated with Hydra-NL. When these results are extrapolated, a breach still occurs for return periods lower than required by the safety norms. Nevertheless, the analysis shows that different methods used to obtain the loads can result in a large difference in the outcome.

The sensitivity analysis showed that the return period for which breaches occur due to erosion of the outer slope vary greatly with the uncertainty of several parameters. However, using mean values for all the parameters in the erosion model still leads to breaches occurring at return periods which are lower than allowed by the safety norms. Hydraulic loads generated with data from the ECMWF lead to erosion volumes which are a factor three lower than with loads generated from Hydra-NL. When the results are extrapolated this also leads to breaches occurring at lower return periods than allowed by the safety norms. The extrapolation of these results is, however, a very rough method. Before this sensitivity analysis, the conclusion was that the Twin Dike does not meet the required safety norms for the failure mechanism erosion of the outer slope. These uncertainties mean this cannot be determined with 100% certainty. A more detailed analysis of the occurring loads in this region will help to better understand these uncertainties. This is one of the reasons the local water board is conducting a multi-year measuring program in this area (Waterschap Noorderzijlvest, n.d.-b).

7
Cost-Benefit Analysis

In order to assess if multiple lines of dikes are a viable alternative for traditional integral dike reinforcements, a cost-benefit analysis is performed on the Twin Dike project. First, the total costs of the project will be summarised and explained. Secondly, the costs of the Twin dike project will be compared to a regular dike reinforcement. Next, possible alternatives for the Twin Dike project will be discussed and compared. Finally, a conclusion is presented on the costs and benefits of this project.

7.1. Costs of the Eemshaven-Delfzijl reinforcement project

It first has to be mentioned that data regarding the costs has been gathered from various sources. These are documents published by engineering companies, the province, water board, Flood Protection Programme (FPP) and other parties that are involved in financing these projects. These documents have been published in the last 3-4 years. Within these years, certain costs of certain aspects have been altered. Costs mentioned in this chapter are obtained from the most recent and most reliable documents. A document was deemed reliable if it was an actual application for a fund stating the required costs for various aspects. Design reports where estimations were given on various costs were deemed less reliable. When there is a large difference in costs between certain documents, this will be elaborated further.

The Eemshaven-Delfzijl dike reinforcement is one large integral reinforcement project where the entire 11.7 km dike in between Eemshaven and Delfzijl is being reinforced integrally. The construction of the Twin Dike project is part of this total reinforcement project.

7.1.1. Flood protection costs

The total costs of the safety aspects for the Eemshaven-Delfzijl dike reinforcement project are given in Table 7.1. These costs are the total costs for all the different phases of the project. Namely, the reconnaissance phase, the planning phase and implementation phase.

Project	**Costs**	**Paid by**
Safety Regular	€ 95,500,000	FPP (90%) and water board (10%)
Innovation Twin Dike	€ 12,100,000	FPP (100%)
Soil subsidence measures	€ 8,600,000	Soil subsidence commission
Earthquake measures	€ 41,600,000	NAM
Total	**€ 157,800,000**	

Table 7.1: Total costs for safety issues of the Eemshaven-Delfzijl reinforcement project (Waterschap Noorderzijlvest, 2016b)

The different aspects of the costs are further explained below.

- **Safety Regular** These are the regular costs of increasing the safety level of the dikes, excluding special projects like the Twin Dike project. This includes costs for ground works (increasing crest and constructing berms), (re)construction of stone and asphalt revetments but also the costs for buying and altering land when additional space is needed to strengthen the dike. Throughout the country, these costs are paid for 90% by the Flood Protection Programme (FPP) and for 10% by the local water board. In this case, this amount was needed to strengthen 9.2 km of the dike (the total length excluding the Twin Dike project).
- **Innovation Twin Dike** The above-mentioned costs are for the stretches where a regular integral dike reinforcement is conducted. Additionally, there are costs for innovations. The FPP pays 100% of the costs for innovations, provided these projects satisfy certain innovative criteria. The Twin Dike project is also an innovative project which satisfies these criteria. These costs include all the costs related to safety issues at the Twin Dike. These are the costs for constructing the inner dike and reinforcing the outer dike. The area between the inner and outer dike is leased from the current owners by the province. This way, if the Twin Dike fails as a pilot project, the land is given back to these owners. On this area, different pilot projects are conducted. The owners of these projects (private companies) have to pay rent to the province for the use of this area. The net lease costs (lease costs minus lease income) are paid by the FPP and are included in these costs.
- **Soil subsidence measures** Due to gas extraction in the province of Groningen this area suffers additional soil subsidence. Measures needed to counteract the effects of soil subsidence are paid by a special commission (Soil Subsidence Commission). In this case, the commission pays for the countermeasures of 0.45 m of soil subsidence. These are the extra costs for groundworks, revetments and the purchase of land.
- **Earthquake measures** Additionally, gas extraction also causes earthquakes. Extra costs have to be incurred to make a dike able to withstand earthquakes. These are for example the costs to implement a stability berm, soil improvements, drainage and sheet pile walls. These costs are paid by the company which is responsible for the gas extraction, the NAM. The NAM and the Soil Subsidence Commission have agreements to specify if a certain measure is needed due to soil subsidence or earthquakes.

Table 7.1 shows that the total costs for the safety issues of the Eemshaven-Delfzijl reinforcement project are €157.8 million. However, at the end of 2017, it turned out that the costs were overestimated by roughly € 35 million. Some costs depicted in Table 7.1 turned out to be higher but some were also lower. The highest reduction in costs was due to a reduction in the need for earthquake measures. Taking into account these differences leads to the following costs as depicted in Table 7.2. The total costs for the safety issues of the Eemshaven-Delfzijl reinforcement project are then € 123.2 million. These costs are the total costs for all the different phases of the project. Namely, the reconnaissance phase, the planning phase and implementation phase.

Project	**Costs**	**Paid by**
Safety Regular	€ 96,900,000	FPP (90%) and water board (10%)
Innovation Twin Dike	€ 12,100,000	FPP (100%)
Soil subsidence measures	€ 10,000,000	Soil subsidence commission
Earthquake measures	€ 4,200,000	NAM
Total	**€ 123,200,000**	

Table 7.2: Updated total costs for safety issues of the Eemshaven-Delfzijl reinforcement project Waterschap Noorderzijlvest (2016b, 2017a, 2017b)

Even more recently (halfway 2019), members of water board Noorderzijlvest mentioned that the *Earthquake measures* costs were reduced even more. The only earthquake measures conducted are the placement of a few clay depots as spare reparation material in the vicinity of the dike. The earthquake measures are reduced to approximately € 300,000. Taking this final update into account leads to total costs as depicted in Table 7.3. The total costs for the safety issues of the Eemshaven-Delfzijl reinforcement project are then € 119.3 million. These costs are the total costs for all the different phases of the project. Namely, the reconnaissance phase, the planning phase and implementation phase.

Project	Costs	Paid by
Safety Regular	€ 96,900,000	FPP (90%) and water board (10%)
Innovation Twin Dike	€ 12,100,000	FPP (100%)
Soil subsidence measures	€ 10,000,000	Soil subsidence commission
Earthquake measures	€ 300,000	NAM
Total	**€ 119,300,000**	

Table 7.3: Most recent total costs for safety issues of the Eemshaven-Delfzijl reinforcement project (Waterschap Noorderzijlvest, 2016b, 2017a, 2017b)

For the Twin Dike project, however, some additional costs have to be incurred that are not directly related to the safety issue. These are discussed in the next section.

7.1.2. Additional costs Twin Dike project

The previous section described the costs needed to provide the required safety to the area. However, for the Twin Dike project, additional costs have to be made. The Twin Dike project consists of two separate areas in between the two dikes. In these areas, two pilot projects will be conducted. Namely, a silt engine and saline agriculture (see Section 3.2 for more information). Additional costs have to be made for the preparation of this area. An overview of the costs is given in Table 7.4.

Project	Costs	Paid by
Inlet structure	€ 9,900,000	Province
Adaptation of waste water pipe	€ 800,000	Province
Spatial planning of the area	€ 2,000,000	Province
Total	**€ 12,700,000**	

Table 7.4: Costs for preparing the area in between the dikes at the Twin Dike project (Provincie Groningen & Waterschap Noorderzijlvest, 2016; Waterschap Noorderzijlvest, 2016c). Excluding the costs for the safety measures (see Table 7.3)

The costs are further elaborated below:

- **Inlet strucutre** These are the costs needed to realise an inlet structure which connects the sea with the area in between the two dikes. The structure is needed to supply salt water in and outflow for the silt engine and saline agriculture.

- **Adaptation of wastewater pipe** A wastewater pipeline is located in the northern part of the Twin Dike project. The pipeline needs to be altered at the location of the outer and inner dike. These costs are paid for by the FFP under the safety issues. The pipeline also crosses the area in between the two dikes. At this location, the pipeline also needs to be altered. These are the costs depicted here.

- **Spatial planning of the area** These costs are needed to make the area suitable for saline agriculture and the silt engine.

7.1.3. Additional costs extra projects

There are also additional costs made by the government, province, local water board or other parties to implement certain additional projects. Examples are a bicycle path on top of the dike and the construction of windmills. These costs are not directly related to the costs for the safety issues of the total reinforcement project or the Twin Dike project itself. Therefore, these costs are not relevant to the conclusions of this chapter. However, in order to have a complete view of the total costs, they will be discussed in this section. An overview of the additional costs is depicted in Table 7.5.

Project	**Costs**	**Paid by**
Innovation multi-year field measurements	€ 9,600,000	FPP
POV Wadden Sea	€ 200,000	FPP
Rich Dike	€ 880,000	Province
Bird breeding island	€ 1,500,000	Province
Bicycle path	€ 1,080,000	Province
City beach Delfzijl	€ 2,270,000	Municipality Delfzijl
Administration windmills	€ 30,000	Energy Company
Pilot clay loam	€ 260,000	RWS
Grass-concrete tiles	€ 130,000	Water board
Total	**€ 15,950,000**	

Table 7.5: Additional costs for extra projects related to the Eemshaven-Delfzijl dike reinforcement (Waterschap Noorderzijlvest, 2016b)

The costs are further elaborated below:

- **Innovation multi-year field measurements** Like the Twin Dike project, this is an innovation fully paid by the FPP. In this project, measurements are being performed in and around the Ems-Dollard estuary for the next 12 years. This is done in order to get a better understanding of the occurring hydraulic conditions in this area. Currently, there is a lot of uncertainty in these loads, probably resulting in over-dimensioned flood defences. The results of this programme can lead to a reduction in costs for future dike reinforcements.
- **POV Wadden Sea** In recent years, studies have been performed on the effects of climate change and the need to keep the Wadden Sea area safe while maintaining the unique values of the area. Different innovations are being realised within the area of this dike reinforcement. These are the Twin Dike, the Rich Dike and the Overtopping Resilient Dike. These costs are for POV Wadden Sea to conduct pilot tests on these projects.
- **Rich Dike** The Rich Dike is a project where the hard transition between the sea and the asphalt revetment of the dike is being softened in order to make it more attractive for birds. Examples are the construction of tidal pools, vertical poles and groynes.
- **Bird breeding island** The breeding island is part of the Rich Dike. An artificial island is created offshore to increase the attractiveness of the area for birds.
- **Bicycle path** A bicycle path is constructed on top of the dike over the entire stretch of the dike reinforcement programme. This is done to increase the recreational attractiveness of the area.
- **City beach Delfzijl** The municipality of Delfzijl wants to use this opportunity to construct a beach in order to increase recreational attractiveness.
- **Administration windmills** Windmills are constructed on the dike by the energy company RWE. The windmills are funded by the energy company itself. These are the administrative costs made by the water board and the province which are compensated by RWE.

- **Pilot bolder-loam** Rijkswaterstaat investigated the usage of bolder-loam in future dike reinforcement programmes. The pilot has already ended and found that the usage of bolder-loam is not an attractive option.
- **Grass-concrete tiles** The water board places grass-concrete tiles near/on the dike in order to increase accessibility and safety for maintenance.

7.2. Expected benefits

The area in between the two dikes will be used for new purposes. A so-called silt engine is constructed in the southern part and the northern part is sub-leased and will become a saline agricultural area (see Section 3.2). These areas are expected to have certain benefits which will be discussed in this section.

7.2.1. Silt engine

Based on Sweco (2018) an estimated amount of 10,000 tons of silt can settle in the area between the two dikes each year. Using the price of regular sand and clay, and the costs to excavate the deposited silt, it can be calculated how much money can be saved. Waterschap Noorderzijlvest (2016c) estimates that € 830,000 can be saved on dike reinforcements when all the silt settled in the area is used for dike reinforcements nearby. In this calculation the costs of sand are € 10 per cubic meter, the costs of clay are € 18 per cubic meter and the silt can be excavated with the expected costs of € 3.10 per cubic meter. If this is the case, the net revenue of the area will be larger than if it were regular grassland. Alternatively, if the silt is not used as material for future dike reinforcements, the silt settled in the basin will cause the land to 'naturally' elevate. This will add to the safety of this area.

7.2.2. Saline agriculture

Currently, the area is being used for the cultivation of potatoes. This has an average income of € 10,000 per hectare per year (Waterschap Noorderzijlvest, 2016c). The northern area is planned to house saline agriculture. The cultivation of saline crops has the potential to have a larger revenue than the current cultivation of potatoes. The exact composition of what crops will be cultivated is unknown. Table 7.6 shows a proposed usage of the area and its revenue. It can be seen that the total revenue per year has a potential value of little over € 1 million. In the current situation, with 30 hectares of potato cultivation, the revenue per year is € 300,000.

Crop type	**Revenues [€/ha/y]**	**Proposed ha**
Cockles	40,000	22
Samphire	32,000	1
Ice plant	28,000	1
Sea-lavender	40,000	1
Seaweed	50,000	1
Saline potato	0	2
Asparagus	0	1
Algae	10,000	1
Total revenue	**€ 1,040,000 /year**	30

Table 7.6: Saline crop types, their net revenue and the proposed amounts of hectares in the Twin Dike project (Waterschap Noorderzijlvest, 2016c).

7.2.3. Sub-leasing

The area in between the two dikes is being leased from the current owners by the province. The net lease costs (lease costs minus lease revenue) are approximately €3.8 million. The lease revenues are expected to be €3 million. Which makes the lease costs €6.8 million (Waterschap Noorderzijlvest, 2016c). The area which is leased in order to construct the Twin Dike project has an approximate area of 50 hectares. The design life of the Twin Dike project is 25 years (see Section 3.2). Assuming the area is leased for a few years longer than 25 years (due to construction time of the Twin Dike project), the area is leased from the current owners for an approximate amount of € 5,000 per ha per year.

The northern area will be sub-leased to businesses which will use the area for saline agriculture. The saline agricultural area has a surface area of approximately 27 hectares. Over a period of 25 years, this means the businesses will sub-lease the area for approximately € 4,000 per ha per year. Regular lease costs for agricultural areas are € 2,500 per ha per year. The net costs for leasing the area are paid by the FPP, and is part of the safety issue of the Twin Dike project.

The province will be responsible for sub-leasing the area to a business for the purpose of saline agriculture. Because this is an innovative project, success is not guaranteed. Having the province lease the area, places the risks in the hands of the province, stimulating the saline agriculture. Table 7.6 shows that the potential net revenue of saline agriculture is larger than the current net revenue of the area. Studies show that over time, owners of the saline farms can pay a lease of up to €7,000 per ha per year (Waterschap Noorderzijlvest, 2016c). If this will indeed be the case, current agricultural owners might be willing to cooperate with a multiple line of dike concept in the future without the government having to lease the area from them. This might be an attractive revenue model since the land of the current agricultural owners will become worth more, or they can lease it to saline agricultural businesses for high profits themselves. If this will be the case, this can save € 3.8 million of lease costs for future projects.

On the other side, if this project fails, the area needs to be returned to the original state. This will further increase the costs of the Twin Dike project. The FPP will pay the costs for the removal of the inner dike and strengthening of the outer dike. The area in between the dikes will have to be returned to its former state before it can be given back to the owners. These costs, as well as the costs for the removal of the inlet structure, will be paid by the province.

7.3. Comparison Twin Dike and regular reinforcement

For this research, it is interesting to find out if a system of multiple lines of dikes is an attractive alternative for a regular dike reinforcement with regards to costs. For this sake, the costs for the Twin Dike project are compared to a regular reinforcement. The regular reinforcement will be the reinforcement conducted north and south of the Twin Dike in the Eemshaven-Delfzijl reinforcement project. This comparison will be made with the use of the information given in the previous sections.

Table 7.3 shows that the costs for the total 11.7 km dike reinforcement are € 119.3 million. The Twin Dike project has a length of approximately 2.5 km. This means that the rest of the reinforcement project is conducted over 9.2 km. € 12.1 million is needed to construct the Twin Dike project. €96.9 million is needed to reinforce the remainder of the flood defences in this reinforcement project. Additionally, the Twin Dike project requires an extra €12.7 million for preparation of the area (see Table 7.4). This results in costs per km as depicted in Table 7.7.

Type	**Average costs per km**
Regular reinforcement	€ 10.53 million
Twin dike (Safety measures only)	€ 4.84 million
Twin dike (Including area preparation)	€ 9.92 million

Table 7.7: Costs per km of the regular reinforcement and Twin Dike reinforcement with average lease profit excluding silt engine profit and earthquake- and soil subsidence measures (Waterschap Noorderzijlvest, 2016b, 2017a, 2017b).

Table 7.7 shows that the regular reinforcement has an average cost of little over € 10 million per km. Costs for the Twin Dike project more than twice as low when only the safety issues are taken into account. However, if costs for the preparation of the area in between the two dikes are included, the costs per km for the Twin Dike project are only marginally less than for a regular reinforcement.

For a proper comparison also the costs for soil subsidence measures (€ 10 million) and earthquake measures (€ 300,000) have to be included. Unfortunately, it is unclear what part of the soil subsidence and earthquake budget is used for the Twin Dike area compared to the rest of the project. However, an estimation can be made. The original dike at the location of the Twin Dike project is not elevated. It will, therefore, be assumed that soil subsidence measures were not implemented at the Twin Dike project. Earthquake measures are assumed to be conducted over the entire 11.7 km stretch with costs evenly distributed per km. This results in costs per km as depicted in Table 7.8.

Type	Average costs per km
Regular reinforcement	€ 11.65 million
Twin dike (Safety measures only)	€ 4.87 million
Twin dike (Including area preparation)	€ 9.95 million

Table 7.8: Costs per km of the regular reinforcement and Twin Dike reinforcement with average lease profit excluding silt engine profit including earthquake- and soil subsidence measures (Waterschap Noorderzijlvest, 2016b, 2017a, 2017b).

Table 7.8 shows that the costs per km are a little higher than depicted in Table 7.7. Still, costs for the Twin Dike project are less than half compared to a regular reinforcement if only safety issues are taken into account. Taking into account the costs for the preparation of the area in between the two dikes leads to higher costs per km for the Twin Dike project. The Twin Dike solution is still 15% cheaper per km compared to the regular reinforcement.

Section 7.2 showed that the area in between the two dikes has the potential to reduce the net costs of Twin Dike project. The silt engine can potentially save € 830,000 on future dike reinforcements. In the results above, this was not taken into account. The saline agricultural area will be sub-leased to businesses. The tables above show the results when the net lease costs of the Twin Dike project are € 3.8 million (the expected costs). However, in potential, these costs can be reduced to zero. But on the other side, the lease revenue can also be zero if this project fails. For these different scenarios, the costs per km are depicted in Table 7.9.

Type	Average costs per km
Regular reinforcement	€ 11.65 million
Zero net lease costs	
Twin dike (Safety measures only)	€ 3.35 million
Twin dike (Including area preparation)	€ 8.43 million
Zero lease revenue from saline agriculture	
Twin dike (Safety measures only)	€ 6.07 million
Twin dike (Including area preparation)	€ 11.15 million
Including silt engine revenue	
Twin dike (Safety measures only)	€ 4.53 million
Twin dike (Including area preparation)	€ 9.61 million

Table 7.9: Costs per km of the regular reinforcement compared to the Twin Dike reinforcement, with different net lease revenues. Including earthquake and soil subsidence measures (Waterschap Noorderzijlvest, 2016b, 2017a, 2017b).

Table 7.9 shows that the costs per km can differ greatly per scenario. In the case where the net lease costs are zero, the Twin Dike project will be more than 25% cheaper per km than a regular reinforcement. This is including the additional costs for the preparation of the area in between the dikes. On the other side, however, if the project fails and the lease revenue is zero, the Twin Dike project approximately costs the same as a regular reinforcement. The revenue of the silt engine does not have a significant impact on the costs per km. The costs are reduced by only 3%.

7.4. Possible improvements to reinforcement

Chapter 5 showed that the Twin Dike project did not satisfy the required safety norms for the failure mechanism erosion of the outer slope. This means that modifications have to be implemented in order to satisfy the required safety. This can either be done by strengthening the inner dike or by strengthening the outer dike.

From the analysis in Chapter 5 it followed that the inner dike has insufficient elevation to cope with the loads induced by a breach in the outer dike. In order to be able to cope with the loads, the elevation of the inner dike can be increased. The inner dike has to be elevated by at least 2.3 m to a level of 6.3 m + NAP in order for the inner dike not to experience overtopping volumes of more than 15 l/s/m in the most extreme condition. This most extreme situation is ID 15 for a return period of 180,000 years (see Chapter 4). The computer program KosWat is used to make an estimation of the required costs. This program can make a rough estimation of reinforcement costs for flood defences. Using key figures for the costs of sand and clay and a supplement factor of 1.953 the additional costs for increasing the elevation of the inner dike can be calculated. Increasing the elevation by 2.3 m will approximately costs an additional € 3.5 million per km (Rijkswaterstaat, 2016).

Another option is to reinforce the outer dike such that a breach will not be able to form in the first place. This can be done by pulling up the hard revetment on the outer slope. Communication with a cost expert from Rijkswaterstaat resulted in expected average costs of €55/m² for the construction of a stone revetment. The stone revetment will need to be increased to a level of approximately 7.5 m + NAP to satisfy the required safety norms. This means that the block revetment has to be placed over an approximate width of 25 m to 30 m. With a supplement factor of 1.902, as mentioned in Rijkswaterstaat (2016), this will lead to additional costs of approximately € 3 million per km.

The paragraphs above show that in order to meet the required safety norms the costs per km for the Twin Dike increase considerably. Even when the most optimistic scenario from Table 7.9 is chosen (zero lease costs including silt engine revenue), the additional costs to meet the required safety norms for the Twin Dike project will make the costs per km larger than for a regular reinforcement.

7.5. Conclusion

This analysis showed that the Twin Dike in its current shape has the potential to be cheaper than a regular reinforcement. Compared to a regular dike reinforcement the Twin Dike project is € 0.5 million to € 3.5 million per km cheaper. This includes the additional costs needed to prepare the area in between the two dikes. When one looks only to the safety measures, the Twin Dike is roughly € 6.5 million per km cheaper than a regular reinforcement. Additionally, it provides space for saline agriculture which can turn out to be a more durable and profitable form of agriculture in this area. Also, the silt engine can result in cheaper dike reinforcements in the future and will increase the elevation of the land 'naturally' which adds to the flood safety of the area.

However, Chapter 5 showed that the Twin Dike does not satisfy the required safety norms for the failure mechanism erosion of the outer slope. Measures which can be taken in order to meet the required safety norms cost at least an additional several million Euro per km. The costs per km for the Twin Dike project will then be higher than for a regular reinforcement when the additional costs needed to prepare the area in between the two dikes are included. For only the safety issues, the Twin Dike project will still be cheaper per km.

However, when one looks solely at the safety issue, there are other solutions which can be even cheaper. This report showed that overtopping volumes are most probably low enough such that these will not cause a flood by themselves. This means that only erosion of the outer slope of the outer dike is a concerning failure mechanism. This can be fixed by constructing a hard revetment on the outer slope of the outer dike. The previous section showed this can be done for approximately € 3 million per km. This solution by itself will already provide the required safety entirely. In the most optimal case, the Twin Dike project costs only € 3.35 million per km (see Table 7.9). This means that the construction of a hard revetment only is cheaper than the Twin Dike and will also probably satisfy the required safety. Although, the expected benefits from the silt engine and saline agriculture will then not be present.

Furthermore, the design life of the Twin Dike project is only 25 years. This means that after 25 years this project area has to be altered to again meet the required safety norms. A more extreme case is if the pilot project is deemed unsuccessful and the entire area has to be returned to its former state. Also, the outer dike will have to be reinforced the regular way. A regular reinforcement has a design life of 50 years. The additional costs that have to be made for the Twin Dike project after 25 years will most probably lead to such an increase in costs that over 50 years a regular reinforcement is cheaper per km. Since this is a pilot project, it is only conducted for a period of 25 years. In order to better compare the costs of the Twin Dike project with a regular reinforcement the Twin Dike project would also have to be implemented for 50 years. However, given the information above, it is safe to assume that the Twin Dike project is not an optimal solution regarding costs.

There can, however, be other situations where this concept might be more favourable than a regular dike reinforcement. An example is a location where a second inner dike is already present in the landscape. Provided this dike has sufficient dimensions, it can reduce the costs so that a Twin Dike concept becomes an attractive solution. Also, performing this concept on a larger scale can be favourable. The main costs for the Twin Dike project are the costs of the inlet structure. Using a larger area with still one inlet structure can reduce the costs per km. A wider area can also cause the loads on the inner dike to become less. When a breach occurs the water will spread over a larger area. This way the loads acting on an inner dike can become lower such that it does not immediately overflow when a breach occurs in the outer dike. This can be beneficial for this concept.

8
Conclusions and Recommendations

8.1. Conclusion

In this research, the use of multiple lines of dikes behind each other was investigated. It was analysed if this is an attractive alternative for traditional integral dike reinforcements regarding flood safety and costs. The Twin Dike project was analysed as a case. The conclusion is that, in this case, multiple lines of dikes are not an attractive alternative for traditional dike reinforcements regarding flood safety and costs.

Flood defences in the Netherlands are assessed using the Statutory Assessment Instruments (WBI). Reinforcements or new flood defences are designed using the Design Instruments (OI). In the Netherlands, primary flood defences protect the land from the sea, main rivers and main lakes. The primary flood defences are split up in dike trajectories, which each have to satisfy a safety norm that is fixed by law. The safety norm states the maximum allowable probability of causing a flood per dike trajectory. The Twin Dike area has a safety norm of 1/3,000 per year. The WBI and OI state rules and methods that need to be followed when assessing or designing a primary flood defence. The WBI defines many different failure mechanisms. Each of these can cause the failure of a flood defence and consequently a flood. When a primary flood defence does not satisfy the required safety norms for one or more of these mechanisms, it will still have residual strength before an actual flood occurs. This residual strength is not taken into account in current instruments (or only to a lesser extent).

Loads on a flood defence are dependent on the average return period. The higher the return period, the higher the loads. Flood defences are assessed per failure mechanism. Each failure mechanism has a maximum allowable probability of failure which, when added up for all the different failure mechanisms, equals the total allowable probability of causing a flood of a dike trajectory. The hydraulic loads can be determined with Hydra-NL. Hydra-NL is a computer program which can calculate hydraulic loads in a probabilistic manner per return period along all the primary Dutch flood defences.

Erosion of the outer slope due to wave impact is the governing failure mechanism at the Twin Dike. The original dike was assessed in the third national assessment round in between 2006 and 2011. In this assessment, the original dike did not meet the safety requirements for the failure mechanisms overtopping (height), macro-instability and erosion of the outer slope. In the Twin Dike reinforcement project, the outer dike (the original existing dike) will only be altered such that macro-stability is ensured. To analyse the total safety of the Twin Dike system the outer dike was assessed again on overtopping and erosion of the outer slope in this research. It was assumed all the other failure mechanisms are not governing since these satisfied the safety norms in the original assessment. The Twin Dike has to be able to resist overtopping conditions which occur with an average return period of 37,500 years. It was found that overtopping volumes were too low to cause damage to the outer dike for return periods specified by the safety norms. The overtopping volumes cause water levels to increase about 0.5 m in the area in between the two dikes for the considered return period. These amounts of water are not able to cause significant loads on the inner dike. The increase of water level in the basin due to overtopping is relatively low. These volumes will most probably not lead to a flood in this area if the inner dike was not present. This suggests that the construction of the inner dike is not necessary for the counteraction of the overtopping volumes. Erosion of the outer slope of the outer dike due to wave impact is, however, troublesome. At some parts of the outer dike, the slope was not able to withstand conditions with an average return period of 100 years. If this area was protected by a single regular dike it has to be able to withstand conditions with an average return period of 180,000 years. This method does, however, not take into account the residual strength of the dike. The impact of the erosion on the loads acting on the inner dike can, therefore, not be determined with these methods. An erosion model was used to further analyse the effects of the residual strength.

Using a prototype erosion model it was determined that the Twin Dike project does not satisfy the required safety norms. The model was used to assess the effect of erosion of the outer slope of the outer dike on the loads acting on the inner dike. This model takes the residual strength of the underlying clay- and sand layers into account and depicts the damage and erosion of a dike body over time. The model showed that the residual strength of the dike is significant. Breaches formed at conditions with an average return period of 3,000 years. Compared to the regular assessment instruments, where the slope was not able to withstand conditions with an average return period of 100 years, the dike can withstand average conditions which are 30 times less frequent. The model showed that for all the cases where a breach formed, hydraulic loads were such that after a breach the inner dike experiences overtopping volumes of at least several hundreds of l/s/m. Since this is a 2D model, a conservative breach width of 20 m was assumed. The inner dike cannot withstand these amounts of overtopping volumes. When a breach is formed in the outer dike, the inner dike also fails. Breaches due to erosion of the outer slope and thus total system failures occur for conditions with average return periods from 3,000 years and higher. This dike stretch has to be able to withstand conditions with an average return period of at least 180,000 years. Following the rules and methods of the WBI and OI, this dike stretch does not satisfy the required safety norms using this particular erosion model.

A sensitivity analysis showed that the return periods for which breaches occur due to erosion of the outer slope vary greatly within the uncertainty of several parameters. The current instruments and the prototype erosion model take various parameters as input. A sensitivity analysis was performed to assess the effects of the uncertainty of those parameters on the results. The impact of sea level rise, soil subsidence and wave characteristics was analysed for the overtopping results. The results showed that sea level rise has a significant and the largest impact on the results. In some cases, overtopping volumes were three times as large. This, however, does not influence the conclusions mentioned in the previous paragraph. The overtopping volumes are still not large enough to cause failure of the outer or inner dike. Also, the volumes are still too low to lead to a flood in this area if the inner dike was not present. The influence of different model coefficients, geometrical dike parameters, sea level rise and hydraulic loads were analysed for the erosion model results. The model- and strength coefficients for grass have a large effect, both in a positive and negative sense, on the exact return period for which a breach occurs. The model coefficient for clay and sea level rise also have this effect but to a lesser extent. The remaining parameters (sand and geometrical parameters) have an even smaller effect on the results. However, when mean values for all the parameters in the erosion model are used this still leads to breaches occurring at return periods which are lower than allowed by the safety norms. Also, the effects on the results by using different hydraulic loads were analysed. The sensitivity analysis showed that hydraulic loads generated with data from the ECMWF lead to erosion volumes which are a factor three lower than with loads generated from Hydra-NL. When the results are extrapolated this still leads to breaches occurring at lower return periods than allowed by the safety norms. Combining all the above-mentioned uncertainties leads to the conclusion that the exact average return period for which breaches occur due to erosion of the outer slope of the outer dike cannot be determined using the methods in this report.

A cost-benefit analysis was performed to analyse the viability of multiple lines of dikes as an alternative for traditional dike reinforcements. The cost-benefit analysis showed that the Twin Dike in its current shape has the potential to be 25% cheaper per km than a regular reinforcement if the pilot project in between the dikes turn out to be successful. This includes additional costs for the preparation of the area in between the dikes and the construction of an inlet structure. For only the flood safety aspects, costs for the Twin Dike project are more than twice as low compared to a regular reinforcement. From the safety analysis, however, it followed that the Twin Dike system does not satisfy the required safety norms using current assessment instruments. When measures are taken to satisfy the required safety norms, the costs for the Twin Dike per km become more than for a regular reinforcement. When looking only at the safety issues, the Twin Dike will still cost less per km than a regular reinforcement. But, in order to only satisfy the required safety norms, there are other options which cost even less than a Twin Dike solution. Also, the Twin Dike has a design life of only 25 years compared to 50 years which is the norm for a regular reinforcement. This fact makes the Twin Dike not an optimal solution regarding costs. This is, however, a pilot project. There can be other situations where this concept might be more favourable than a regular reinforcement. Examples are locations where a second inner dike of sufficient strength is already present in the landscape. Also, performing this concept on a larger scale can make multiple lines of dike a more favourable solution. Both examples can potentially decrease the costs per km making this concept a more attractive alternative to traditional dike reinforcements.

Based on the analysis of the Twin Dike project, multiple lines of dike are not an attractive alternative to traditional dike reinforcement with respect to costs and flood safety. The Twin Dike project does not satisfy the required safety norms as stated by the current design- and assessment instruments. The inner dike is not able to withstand loads for required conditions. A total system failure occurs at return periods more frequent than allowed. There is, however, a large uncertainty in these results. Also with respect to costs, multiple lines of dike are not an attractive alternative to provide the required flood safety.

8.2. Discussion

The regular design- and assessment methods already showed that the outer slope of the outer dike did not satisfy the safety requirements. The usage of the prototype model confirmed that this was the case. As expected, it showed that the outer dike had significant residual strength, but still too little to prevent a breach for the required return periods. The model is, however, a prototype and is not widely used for the assessment of flood defences. From the sensitivity analysis, it followed that some parameters can have a large influence on the exact return period for which a breach occurs. This has to be taken into account when interpreting the results. In order to fully validate these results, one has to compare different erosion models and validate them with actually occurred erosion. Also, this model only takes 2D effects into account. To better predict the water entering through a breach a 3D model has to be used. Finally, the model does not take into account the presence of a hard revetment. For most of the locations, this does not have a large influence. The stone revetment reaches up to 3.5 m + NAP on these locations. Erosion often occurs above this level. One location, however, has a stone revetment which reaches up to 5.5 m + NAP. For better accuracy of the results, this has to be taken into account.

Also, the hydraulic loads used in the erosion model are probably not a good representation of the loads occurring in reality. Hydra-NL calculated wave loads over the course of a storm. During the peak of the storm, there is a dip in the wave heights. This is not a realistic scenario. These dips become larger the higher the return periods get. Using a simulated storm generated from another source (ECMWF), it was already showed that the occurring wave height and consequent erosion are a lot lower. Employees from the local water board also mentioned that in their experience, the hydraulic loads computed by Hydra-NL seem to be overestimated compared to what is actually seen in this area. In order to fully assess these effects, it is useful to perform a more detailed analysis of the occurring loads in the Ems-Dollard estuary. The local water board has already set up a multi-year measuring programme to further analyse these effects. Also, sea level rise has a large impact on the results. Unfortunately, uncertainty in future sea level rise will probably remain a discussion in the near future.

The results showed that when this particular erosion model is used, the Twin Dike does not meet the required safety levels according to the rules and methods stated by the WBI and OI. The law, however, states that this particular area may experience floods with a minimum allowable return period of 3,000 years. In this report, the failure of the Twin Dike system was assessed. But failure does not necessarily equal a flood. A flood means that casualties and considerable damage have to occur. If the inner dike fails and a breach occurs, the water from the basin will flow into the area behind the Twin Dike. From the results, it followed that this can happen after the peak of the storm. If conditions are such that the amount of water does not cause a flood, the Twin Dike is still safe enough according to the law. In order to fully assess if the Twin Dike provides the required safety, one has to analyse if the occurring failure mechanisms will indeed lead to a flood. This has to be done with more detailed models which can estimate the course of a flood and the consequent damage and casualties. This is deemed outside the scope of this research. Nevertheless, this has to be taken into account when interpreting the results.

This research concludes that the Twin Dike does not satisfy the required safety norms. This is interesting since previous assessments of this project concluded that it does provide the required safety. There are some vital differences between this report and previous assessments. The first one is a change in calculated overtopping volumes. In this report overtopping volumes are sometimes a factor five lower than for example in Hölscher (2018). This is due to the fact that in the older reports an older version of Hydra-NL was used: Hydra-K. New calculation methods in Hydra-NL lead to lower overtopping volumes in this area. Off course, this does not alter the results: the Twin Dike still satisfies the safety norms for this mechanism. For erosion of the outer slope, the results are also different. In older reports, the outer dike satisfied the required safety norms for this mechanism (Gautuer et al., 2015; Hölscher, 2018). These were, however, assessed with old assessment instruments where the required safety norm for this failure mechanism was lower. Because of this, in the older reports, the Twin Dike does meet the required safety norms for this failure mechanism while it this report is does not. This has to be kept in mind when comparing results from this research with previous research.

8.3. Recommendations

Based on the conclusions and discussion the following recommendations are given.

Investigate accuracy erosion model The erosion model used is a prototype model. It is useful to assess its applicability and validity in comparison with other models and actual cases. This can reduce the uncertainties presented in the sensitivity analysis. Also, including the effects of a hard revetment will further improve the results.

Assess flood conditions A failure of the Twin Dike system does not necessarily lead to a flood. A breach sometimes occurs after the peak of a storm. The amounts of water coming through the breach and over the inner dike might be low enough not to cause a flood. Further analysing if flood conditions actually occur in these situations can improve the accuracy of the conclusions presented in this research. For this, it is also useful to analyse erosion effects with a 3D model. This way the actual amount of overtopping and erosion can be better estimated.

Further investigate the soil composition and layout on sight Desired detailed information about the composition of the dike was not available. Old design drawings were used which did not contain all the required information. Performing multiple cone penetration tests will give a better insight into the composition of the dike. This way, the thickness of soil layers, their location and composition can be better incorporated in the assessment. A better insight into these parameters will give more reliable results.

Improve accuracy and knowledge on hydraulic loads in the region The results showed that the wave height development over the course of a storm as generated by the current instruments is unrealistic. Using data from other sources already led to lower loads. Eyewitness reports of the local water board also mention lower hydraulic loads than predicted by Hydra-NL. Further analysing the actually occurring loads can help to increase the accuracy of the results.

Investigate possible other cases The Twin Dike did not prove to be an optimal solution in this case. However, there might be other situations where this concept might be more favourable. It is recommended to investigate other cases to analyse the applicability of the concept of multiple lines of dikes behind each other.

Bibliography

Actueel Hoogtebestand Nederland. (2009). Retrieved 2019-01-08, from https://ahn.arcgisonline.nl/ahnviewer/

Almere Tours. (n.d.). *Zuiderzee Works - Almere Tours.* Retrieved 2018-10-02, from https://almeretours.com/almere-history/

Baart, F., Rongen, G., Hijma, M., Kooi, H., de Winter, R., & Nicolai, R. (2018). *Zeespiegelmonitor 2018* (Tech. Rep.). Deltares, HKV.

Bötger, E., & Te Linde, A. (2014). *Verschillende perspectieven op de nieuwe waterveiligheidsnormen* (Tech. Rep.). Amsersfoort: Twynstra Gudde.

Bouwer, L. M., Bubeck, P., Wagtendonk, A. J., & Aerts, J. C. J. H. (2009). Inundation scenarios for flood damage evaluation in polder areas. *Natural Hazards and Earth System Sciences*, *9*(6), 1995–2007. Retrieved from https://www.nat-hazards-earth-syst-sci.net/9/1995/2009/ doi: 10.5194/nhess-9-1995-2009

Breteler, M. K. (2015). *Residual strength of grass on clay in the wave impact zone* (Tech. Rep.). Deltares.

Brinkman, R., Nuttall, J., Noordam, A., & Teixeira, A. (2018). *Project software macro stability User manual 2018, Part 3: Scientific Background* (Tech. Rep.). Deltares.

Caires, S. (2009). *Extreme wind statistics for the Hydraulic Boundary Conditions for the Dutch primary water defences. SBW-Belastingen: Phase 2 of subproject "Wind Modelling"* (Tech. Rep.). Deltares.

CBS, PBL, RIVM, & WUR. (2016). *Sea level: Dutch coast and worldwide, 1890-2014 (indicator 0229, version 09 , 16 November 2016).* Retrieved 2018-10-04, from www.environmentaldata.nl

Chbab, H. (2017). *Basisstochasten WBI-2017* (Tech. Rep.). Deltares.

Chbab, H., & de Waal, H. (2017). *Achtergrondrapport Hydraulische Belastingen* (Tech. Rep.). Deltares.

ComCoast. (2006). *Pilot Hondsbossche en Pettemer Zeewering* (Tech. Rep.).

de Jong, K. (2019). *Richtlijnen benutting beschermingszone Dubbele Dijk* (Tech. Rep.). Waterschap Noorderzijlvest.

Deltares. (2017). Ontwerpuitgangspunten Hoogwateropgave Dijkverbetering Eemshaven-Delfzijl.

den Heijer, F., Vos, R., Diermanse, F., Groeneweg, J., & Tönis, R. (2006). *Achtergrondrapport HR 2006 voor de Zee en Estuaria* (Tech. Rep.). Rijkswaterstaat.

Duits, M. (2018). *Hydra-NL Gebruikershandleiding* (Tech. Rep.). HKV.

EurOtop. (2016). *Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application.* (J. van der Meer et al., Eds.). Retrieved from www.overtopping-manual.com

Gautuer, C., Schelfhout, H., & van Meurs, G. (2015). *Dijkversterking Eemshaven- Delfzijl waterveiligheid van een dubbele dijk* (Tech. Rep.). Deltares.

Geerse, C., Stijnen, J., & Kolen, B. (2007). *Richtlijn Normering compartimenteringskeringen* (Tech. Rep.). STOWA; HKV; Nelen & Schuurmans.

Groeneweg, J., & den Bieman, J. (2017). *Hydraulische Belastingen Kust* (Tech. Rep.).

Haasnoot, M., Bouwer, L., Diermanse, F., Kwadijk, J., van der Spek, A., Oude Essink, G., … Lenselink, G. (2018). *Mogelijke gevolgen van versnelde zeespiegelstijging voor het Deltaprogramma. Een verkenning.* (Tech. Rep.). Delft: Deltares.

Hofsté, S. (2017). *Waterveiligheid bij een dubbele dijk* (Unpublished doctoral dissertation). NHL Hogeschool.

Hölscher, R. (2018). *Proeftoetsing Dubbele Dijk Normtraject 6-7* (Tech. Rep.). Arcadis.

Hoogwaterbeschermingsprogramma. (n.d.). *Hondsbossche en Pettemer Zeewering.* Retrieved 2019-05-23, from https://www.hoogwaterbeschermingsprogramma.nl/actueel/PersPress/English+press/Exceptional+projects/Hondsbossche+en+Pettemer+Zeewering+Engels/default.aspx

Inspectie Leefomgeving en Transport. (2013). *Verlengde derde toets primaire waterkeringen* (Tech. Rep.). Ministerie van Infrastructuur en Milieu.

Inspectie Verkeer en Waterstaat. (2011). *Derde toets primaire waterkeringen* (Tech. Rep.). Ministerie van Infrastructuur en Milieu. doi: IVW/WB/2011/000002

Jongejan, R. (2016). *Factsheet Het lengte-effect.*

Jonkman, S., Schweckendiek, T., Jorissen, R., & van den Bos, J. (2017). *Flood Defences.* Delft University of Technology.

Jonkman, S. N., Vrouwenvelder, A. C. W. M., Steenbergen, R. D. J. M., Morales-Nápoles, O., & Vrijling, J. K. (2015). *Probabilistic Design : Risk and Reliability Analysis in Civil Engineering* (M. H. G. Baas, Ed.). Delft: Delft University of Technology.

Klein, M. D., & Kroon, J. (2011). *Productieberekeningen Waddenzee voor WTI-2011 : rapportage fase 1* (Tech. Rep.). HKV, Svasek Hydraulics.

Klein Breteler, M., Capel, A., Kruse, G., Mourik, G., & Kaste, D. (2012). *Erosie van een dijk na bezwijken van de steenzetting door golven* (Tech. Rep.). Deltares.

Klerk, W. J., & Jongejan, R. (2016). *Semi-probabilistic assessment of wave impact and runup on grass revetments* (Tech. Rep.). Deltares.

Klijn, F., Baan, P., de Bruijn, K., & Kwadijk, J. (2007). *Overstromingsrisico's in Nederland in een veranderend klimaat* (Tech. Rep.). WL | delft hydraulics.

Kok, M., Jongejan, R., Nieuwjaar, M., & Tanczos, I. (2017). *Grondslagen voor hoogwaterbescherming* (2nd ed.). Ministerie van Infrastructuur en Milieu.

Kwakernaak, C., & Lenselink, G. (2015). *Economische en ecologische perspectieven van een dubbele dijk langs de Eems-Dollard* (Tech. Rep.). Deltares.

Leusink, W. (2018). *Ontwerpnota Dubbele Dijk* (Tech. Rep.).

Luinenburg, A. (2016). *Afweging alternatieven Dijkverbetering Eemshaven - Delfzijl* (Tech. Rep.). Waterschap Noorderzijlvest; Provincie Groningen.

Milczarek, A. (2017). *What are the different methods of structural analysis? What are some examples?* Retrieved 2018-10-09, from `https://www.quora.com/What-are-the-different-methods-of-structural-analysis-What-are-some-examples`

Ministerie van Infrastructuur en Milieu. (n.d.). *Regeling veiligheid primaire waterkeringen 2017 - BWBR0039040.* Retrieved 2018-12-21, from `https://wetten.overheid.nl/BWBR0039040/2017-01-01`

Ministerie van Infrastructuur en Milieu. (2016a). *Achtergrondrapport Ontwerpinstrumentarium 2014* (Tech. Rep.). Rijkswaterstaat.

Ministerie van Infrastructuur en Milieu. (2016b). *Regeling veiligheid primaire waterkeringen 2017 Bijlage II Voorschriften bepaling hydraulische belasting primaire waterkeringen.*

Ministerie van Infrastructuur en Milieu. (2017a). *Factsheet (versie 2) verschillen tussen het faalmechanisme Gras Erosie Kruin en Binnentalud (GEKB) en het Hydraulische BelastingNiveau (HBN)* (Tech. Rep.). Rijkswaterstaat.

Ministerie van Infrastructuur en Milieu. (2017b). *Regeling veiligheid primaire waterkeringen 2017 Bijlage III Sterkte en veiligheid.*

Ministerie van Infrastructuur en Milieu. (2018). *Schematiseringshandleiding grasbekleding* (Tech. Rep.). Rijkswaterstaat.

Mourik, G. (2015). *Prediction of the erosion velocity of a slope of clay due to wave attack* (Tech. Rep.). Deltares.

NAM B.V. (2015). *Bodemdaling door Aardgaswinning* (Tech. Rep.).

Nederlands Centrum voor Geodesie en Geo-Informatica (NCG). (2018). *Bodemdaling in Nederland.* Retrieved 2019-02-26, from `https://atlasleefomgeving.nl/bodemdaling-in-nederland`

Provincie Groningen, & Waterschap Noorderzijlvest. (2016). *Uitvoeringsovereenkomst betreffende de dijkverbetering Eemshaven-Delfzijl en het koppelproject 'de Dubbele dijk'.*

Rijksdienst voor het Cultureel Erfgoed. (2018). *Waterveiligheid door de eeuwen heen | Landschap in Nederland.* Retrieved 2018-09-12, from `https://landschapinnederland.nl/waterveiligheid-door-de-eeuwen-heen`

Rijkswaterstaat. (n.d.). *Waterinfo.* Retrieved 2019-06-29, from `https://waterinfo.rws.nl/{#}!/nav/publiek/`

Rijkswaterstaat. (2016). *SSK uitvoersheet bij KOSWAT v2.3.*

Rijkswaterstaat. (2017). *Handreiking ontwerpen met overstromingskansen* (Tech. Rep.).

Rijkswaterstaat. (2018a). *Primaire keringen - Helpdesk water.* Retrieved 2018-09-13, from https://www.helpdeskwater.nl/onderwerpen/waterveiligheid/primaire/

Rijkswaterstaat. (2018b). *Waterveiligheid - Kenniscentrum InfoMil.* Retrieved 2018-10-03, from https://www.infomil.nl/onderwerpen/lucht-water/handboek-water/thema{%}27s/waterveiligheid-0/

Rongen, G., Stenfert, J., Depuits, G., & Barbosa, C. M. (2018). *Time dependent loads on revetments* (Tech. Rep.). HKV; Deltares.

Ruimte voor de Rivier. (n.d.). *Ruimte voor innovatie stapsgewijs of met grote sprongen* (Tech. Rep.).

Slomp, R. (2016). *Basis hydraulische belastingen.*

Slootjes, N., & van der Most, H. (2016). *Achtergronden bij de normering van de primaire waterkeringen in Nederland* (Tech. Rep.). Ministerie van Infrastructuur en Milieu.

Stichting Deltawerken Online. (2018). *Deltawerken - Deltaworks.* Retrieved 2018-10-02, from http://www.deltawerken.com/deltaworks/23.html

Sweco. (2018). *Presentatie tussentijdse resultaten 2D berekeningen.* Sweci.

’t Hart, R. (2018). *Fenomenologische beschrijving Faalmechanismen WBI* (Tech. Rep.). Deltares.

TAW. (2002). *Technisch Rapport Golfoploop en Golfoverslag bij Dijken* (Tech. Rep.). Technische Adviescommissie voor de Waterkeringen.

ter Horst, W. (2005). *Safety Analysis of the ComCoast concept* (Tech. Rep.). Delft University of Technology.

van Balen, W. (2016). *Hydra-Ring 2.0 Technical Reference Manual* (Tech. Rep.). Deltares.

van der Velden, B. (1995, jan). *Op onderhoud werd bezuinigd, in afwachting van de dijkversterking.* Retrieved from https://www.nrc.nl/nieuws/1995/01/31/op-onderhoud-werd-bezuinigd-in-afwachting-van-de-dijkversterking-7254904-a1165145

van der Wijk, R. (2018). *Dwarsprofielen Dubbele Dijk Opleverhoogte* (Tech. Rep.). Ommelanderdiek.

Vranken, L., & Sinoo, F. (2016). *Inpassingsplan Dijkverbetering Eemhaven - Delfzijl* (Tech. Rep.). Sweco, Provincie Groningen.

Vrijling, J. K., Schweckendiek, T., & Kanning, W. (2011). Safety standards of flood defences. *Proceedings of the 3rd International Symposium on Geotechnical Safety and Risk, ISGSR 2011, Munich, Germany*(June), 67–84.

Wageningen University & Research. (n.d.). *Dubbele dijk Eemshaven-Delfzijl biedt kansen voor ecologie en economie.* Retrieved 2018-11-01, from https://www.wur.nl/nl/project/Dubbele-dijk-EemshavenDelfzijl-biedt-kansen-voor-ecologie-en-economie.htm

Waterschap Noorderzijlvest. (n.d.-a). *Dijkverbetering Eemshaven - Delfzijl.* Retrieved 2019-05-30, from https://www.noorderzijlvest.nl/dijkeemshavendelfzijl/

Waterschap Noorderzijlvest. (n.d.-b). *Project Meerjarige Veldmetingen.* Retrieved 2019-06-28, from https://www.noorderzijlvest.nl/ons-werk/projecten/projecten-(lopend)/project-meerjarige/

Waterschap Noorderzijlvest. (n.d.-c). *Verschil zomerpeil en winterpeil.* Retrieved 2019-06-03, from https://www.noorderzijlvest.nl/actueel/waterstanden/zomerpeil-winterpeil/

Waterschap Noorderzijlvest. (2010). *Legger van de Waterkering van waterschap Noorderzijlvest deel I: Zeedijk* (Tech. Rep.).

Waterschap Noorderzijlvest. (2016a). *Bijlage 1: Nadere Beschrijving Demonstratieproject Dubbele Dijk: Opgave , Beschouwde Alternatieven , Business Case en Onderzoeksaanpak* (Tech. Rep.).

Waterschap Noorderzijlvest. (2016b). *Dijkverbetering Eemshaven-Delfzijl Plan van aanpak* (Tech. Rep.). Waterschap Noorderzijlvest; Hoogwaterbeschermingsprogramma.

Waterschap Noorderzijlvest. (2016c). *Project Dubbele Dijk: opgave, beschouwde alternatieven, business case en onderzoeksaanpak. Plan van aanpak* (Tech. Rep.).

Waterschap Noorderzijlvest. (2017a). *Dijkverbetering Eemshaven-Delfzijl mogelijk zo’n 35 miljoen euro goedkoper.* Retrieved 2019-06-07, from https://www.noorderzijlvest.nl/@14159/dijkverbetering/

Waterschap Noorderzijlvest. (2017b). *Herijking kredietaanvraag Dijkverbetering Eemshaven-Delfzijl (DED)* (Tech. Rep.).

Waterschap Noorderzijlvest, & Grontmij Nederland B.V. (2016). *MER Dijkverbetering Eemshaven-Delfzijl* (Tech. Rep.).

Watersnoodmuseum. (2018). *Eerdere vloeden - Watersnoodmuseum.* Retrieved 2018-10-02, from https://watersnoodmuseum.nl/kennisbank/rampen/

Waterveiligheidsportaal. (2018). *Waterveiligheidsportaal Landelijk Veiligheidsbeeld.* Retrieved 2018-10-03, from https://www.waterveiligheidsportaal.nl/{#}/nss/nss/norm

Waterwet. (2009). Retrieved from https://wetten.overheid.nl/BWBR0025458/2018-07-01

Wemelsfelder, P. (1939). Wetmatigheden in het optreden van stormvloeden. *De Ingenieur*, *54*(9), 31–35.

Wijermars, E. (2017). *Dijkverbetering Eemshaven - Delfzijl ontwerpbasis dijkontwerp* (Tech. Rep.).

A

Glossary of Terms

Dutch	**English**
Achterliggende kering	Rear barrier
Actueel Hoogtebestand Nederland (AHN)	Digital surface model of the Netherlands
Bekleding	Revetment
Bres	Breach
Coupure	Cut
Dijk	Dike, Dyke, Embankment or Levee
Dubbele Dijk	Twin Dike
Hoogwaterbeschermingsprogramma (HWBP)	Flood Protection Programme (FPP)
Hydraulisch Belastingniveau (HBN)	Hydraulic load level, required height of a flood defence given a maximum allowable overtopping discharge
Koninklijk Nederlands Meteorologisch Instituut (KNMI)	Royal Dutch Meteorological Institute
Krib	Groyne
Kruin	Crest
Kwel	Seepage
Kwelweglengte	Seepage length
Normaal Amsterdams Peil (NAP)	Amsterdam Ordnance Datum
Onderloopsheid	Piping
Ontwerpinstrumentarium (OI)	Design Instruments
Opdrukken, Opbarsten	Uplift
Restprofiel	Residual profile
Reststerkte	Residual strength
Rijkswaterstaat (RWS)	The Dutch Directorate-General for Public Works and Water Management
Standtijdlijn model	Resistance-duration curve
Stijghoogteverschil	(Hydraulic) Head difference
Strandhoofd	Groyne
Strijklengte	Fetch
Terp	Mound
Verhang	(Hydraulic) Gradient
Verval	(Hydraulic) Head difference
Voorliggende kering	Front barrier
Watersysteem	Hydraulic region
Welvorming	Heave
Wettelijk Beoordelingsinstrumentarium (WBI)	Statutory Assessment Instruments

B

Background Assessment Software

Different computer programs are used throughout this report. This appendix discusses the applicability and background of these programs.

B.1. Hydra-NL

B.1.1. General

Hydra-NL is a probabilistic assessment program which can be used to assess or design flood defences in the Netherlands. It can calculate hydraulic loads for specific return periods along all the primary Dutch flood defences. The hydraulic loads that can be calculated are water levels, significant wave heights, spectral wave periods, peak wave periods as well as hydraulic load levels, overtopping volumes and wave conditions for revetments. This can be done for the current situation (2023) but also for future situations (2050 and 2100). For future situations, sea level rise is taken into account. Also, different climate scenarios can be chosen. Table B.1 shows the sea level rise for each future situation and climate scenario. The program assumes that sea level rise is uniform all along the Dutch coast.

Scenario	Sea level rise compared to 2017 (m)
2023 – KNMI2006 G	0.00
2023 – KNMI2006 W+	0.00
2050 – KNMI2006 G	0.05
2050 – KNMI2006 W+	0.25
2100 – KNMI2006 G	0.25
2100 – KNMI2006 W+	0.75

Figure B.1: Sea level rise per future situation and climate scenario (Duits, 2018)

Hydra-NL was created by merging Hydra-Zout (salt) and Hydra-Zoet (sweet/fresh) which were respectively used for calculations in the saltwater and freshwater parts of the Netherlands. Hydra-Zoet was again created by merging 8 different Hydra models. For example Hydra-B (B stands for benedenrivieren - lower rivers) and Hydra-VIJ (Vecht and IJssel Delta) (Duits, 2018).

This history of merging different software can still be seen in Hydra-NL today, as it works differently for 8 different hydraulic region types. For each region type, different processes are important for the calculation of the hydraulic loads. The region types are listed below.

- **River to sea with storm surge barrier** The lower rivers belong to this region. Here the discharge of the Meuse and Rhine are important as well as the sea water levels. The Europoort barrier has an influence on the hydraulic loads in this region.

- **River to lake with storm surge barrier** This is the Vecht and IJssel delta. Here the discharge of these two rivers is important as well as the lake water level of the IJsselmeer. The Ramspol barrier has an influence on the hydraulic loads in this region.
- **Lake** This region consists of the main lakes in the Netherlands. In these lakes, the wind and water level are the most important factors for the calculation of the hydraulic loads.
- **River** This region consists of the upper rivers. In these areas, the river discharge and wind are the dominant factors to the contribution of the hydraulic loads.
- **River sea barrier** This is the Hollandse IJssel river. This is comparable to the *River to sea with storm surge barrier* type, but an additional barrier (The Hollandse IJssel barrier) is present and has an influence on the hydraulic loads in this region.
- **River sea lake barrier** This is the Volkerrak-Zoommeer area. It is comparable to *River to sea with storm surge barrier* except that also the lake water level is important for the determination of hydraulic loads.
- **Estuary with barrier** This is the Eastern Scheldt. Here, the hydraulic loads are dependent on the sea water level and wind, but also the storm surge length, the phase difference between the maximum tide and the maximum wind speed. Also, the Eastern Scheldt barrier has an influence on the hydraulic loads in this estuary.
- **Sea** This consists of the Wadden Sea, Western Scheldt and the Holland coast. Here the sea water level and the wind are important.

The processes which are important for the determination of the hydraulic loads differ per hydraulic region. An overview of the relevant processes per region is given in Table 2.3. The hydraulic regions are depicted in Figure B.2.

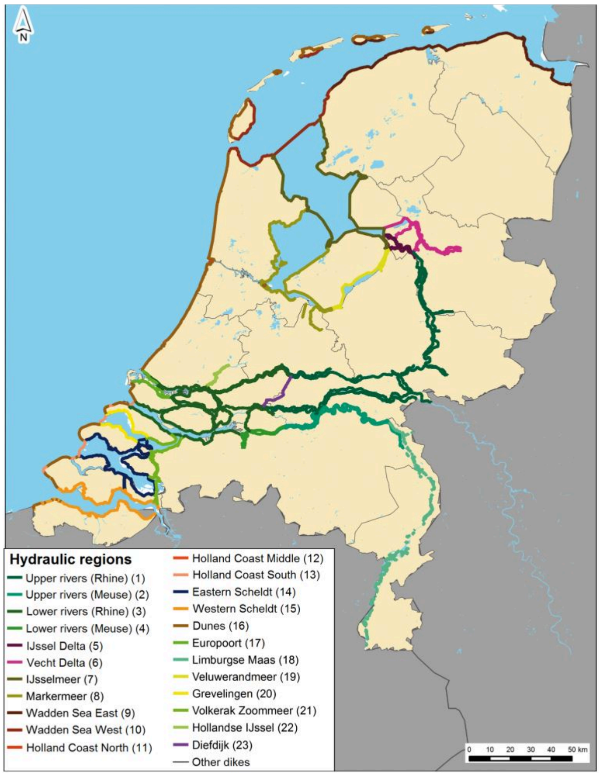


Figure B.2: Hydraulic regions in the Netherlands (Ministerie van Infrastructuur en Milieu, 2016b)

For most hydraulic regions different data is used to determine the hydraulic loads. Since this project focusses on the Twin Dike area (located in area 9: Wadden Sea East), the explanations below are focussed on this area.

B.1.2. Calculation location

When starting Hydra-NL, a dike trajectory has to be chosen where the user wants to perform calculations. The dike trajectories correspond with the trajectories depicted in Figure 2.6b. For each dike trajectory, a certain amount of output locations are available. A calculation can be performed on each of these output locations. Output locations are located approximately every 250 m alongshore and are situated approximately 100 m from the crest of the primary flood defence. Figure B.3 shows an example of the output locations. This is dike trajectory 6-7 in the Ems-Dollard estuary. Every green point is an output location.

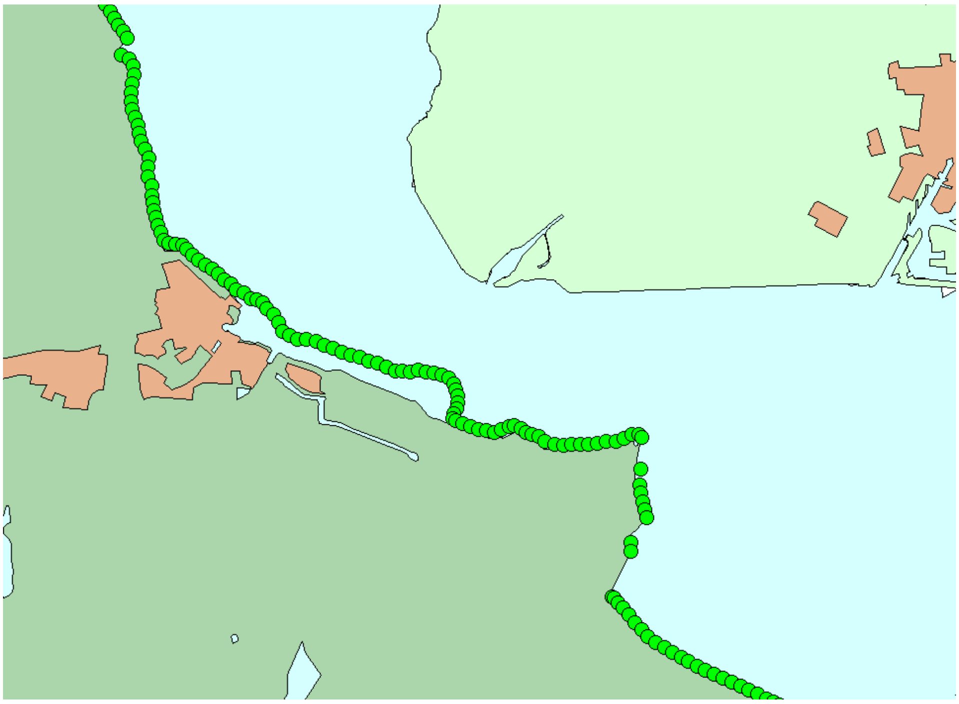


Figure B.3: Hydra-NL output locations for dike trajectory 6-7. Image taken from Hydra-NL interface.

B.1.3. Calculation methods

As mentioned, Hydra-NL can calculate various hydraulic loads. The calculation of these variables consists of multiple steps and is different for most variables. The calculation methods per variable will be described below.

Water levels Water levels are calculated using spatial triangular interpolation of known water level statistics at measuring locations. Using the known water level at three locations the water level at any location in between (or just outside) these locations can be calculated. Figure B.4 shows a schematisation of how triangular interpolation works. With the water levels at location S1, S2 and S3 known, the water level at location L can be determined using interpolation.

The measuring locations that are used for interpolation differ per sub-region within the Wadden Sea. Figure B.5 shows the different sub-regions and their corresponding interpolation locations. The Twin Dike project is located in subregion 1, here measuring locations 4, 5 and 6 are used for triangular interpolation.

Wave characteristics The wave characteristics for the primary flood defences in the Wadden Sea are calculated with a physical model and not using statistics of wave characteristics (which was the case with water levels). A physical model uses wind speed and wind direction to calculate the resulting wave characteristics near the shore. This relation is very complex since it is not only dependent on the wind speed and direction but also the bathymetry, water level and phase difference between the peak

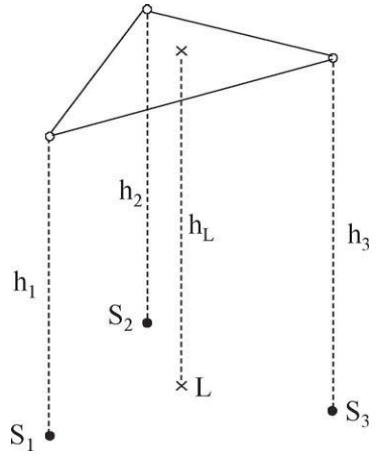


Figure B.4: Triangular interpolation of water levels (den Heijer et al., 2006)

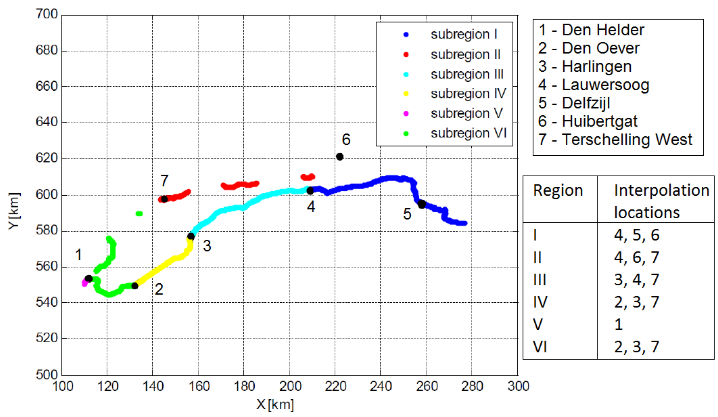


Figure B.5: Wadden Sea triangular interpolation locations. Modified from Chbab and de Waal (2017).

of the tide and peak of the wind speed. It will take a long time for Hydra-NL to calculate this relation every time that wave characteristics are requested at a specific location for a specific return period. That is why these relations are calculated only once, up front, for many different combinations of the above-mentioned variables. See Appendix B.2 for more information. The results of these calculations are stored in a database.

Hydra-NL uses statistics of wind speed, wind direction, water level and their correlation to find the corresponding wave characteristics in the database for a requested location and return period. Water level statistics are used as mentioned in the previous paragraph. Wind speed statistics of the West-Terschelling measuring station on the island of Terschelling are used and wind direction statistics of the Hook of Holland measuring station are used. Figure B.6 gives an overview of how Hydra-NL calculates the wave characteristics in the Wadden Sea region.

Wave characteristics for revetments Hydra-NL can also determine hydraulic loads on revetments. A hydraulic load combination is calculated by specifying the return period and water level, Hydra-NL can then calculate the corresponding wave characteristics. Loads are calculated based on their combined probability of occurrence. The combination of wave characteristics and water level are representative for a certain specified return period. The loads are known 50-100 m from the dike. In order to assess revetments they, however, need to be known at the toe of the dike. The loads are corrected by a dam- and foreshore module. See the section below.

Overtopping aspects To calculate overtopping volumes the wave characteristics and water levels at the toe of the flood defence need to be known. Since the output locations with known data are located

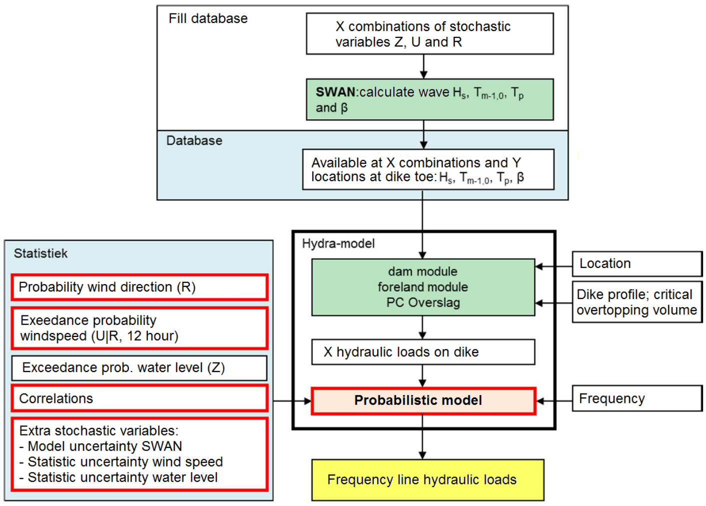


Figure B.6: Determination process of hydraulic loads in the coastal areas. Modified from Groeneweg and den Bieman (2017).

approximately 50-100 m from the dike, it needs to be corrected for the influence of a possible foreshore and breakwater, see Figure B.7. The effect of a foreshore is that water levels will be higher at the toe of the dike due to wind effects, and waves are further dampened. Also, a breakwater dampens wave heights. The exact details of these models will not be discussed here.

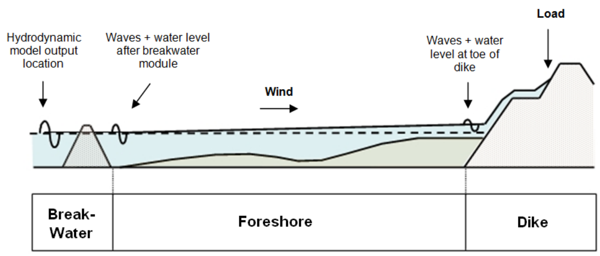


Figure B.7: Breakwater and foreshore influence on water level and waves between the Hydra output location and the toe of the flood defence (van Balen, 2016)

When the wave characteristics and water levels at the toe are known, overtopping volumes and the hydraulic load level are calculated with a separate overtopping module. The overtopping module uses the following equations to calculate overtopping.

$$\frac{q}{\sqrt{g \cdot {H_{m0}}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1.0} \cdot \exp\left(-4.3 \cdot \frac{R_c}{\gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v \cdot \xi_{m-1.0} \cdot H_{m0}}\right) \tag{B.1}$$

In which:

q	=	average overtopping volume	[l/s/m]
g	=	gravitational acceleration	[m^2/s]
α	=	angle of sea side slope	[°]
$\xi_{m-1.0}$	=	breaker parameter (ratio of steepness of waves compared to the sea side slope)	[-]
R_c	=	free-board	[m]
H_{m0}	=	significant wave height	[m]
γ_b	=	reduction factor berm	[-]
γ_f	=	reduction factor slope roughness	[-]
γ_β	=	reduction factor oblique wave approach	[-]
γ_v	=	reduction factor vertical wall	[-]

The overtopping volume will increase with increasing breaker parameter until a certain maximum is reached. This maximum is defined as:

$$\frac{q}{\sqrt{g \cdot H_{m0}{}^3}} = 0.2 \cdot \exp\left(-2.3 \cdot \frac{R_c}{\gamma_f \cdot \gamma_\beta \cdot \gamma^* \cdot H_{m0}}\right) \tag{B.2}$$

Equation B.1 and B.2, which are used in Hydra-NL today, result in 'recommended' values of average overtopping discharge. The values 4.3 and 2.3 in respectively Equation B.1 and B.2, are characteristic values based measured data during small and large scale tests in wave- flumes and basins. If one would want to perform a probabilistic analysis the following mean and standard deviation have to be used (truncated normal distribution): σ(4.75) = 0.50 for value 4.3 in Equation B.1 and σ(2.6) = 0.35 for value 2.3 in Equation B.2.

The equations above are originally from TAW (2002). More recently, EurOtop (2016) updated some coefficients in the equations above due to new insights. Currently in the Dutch design- and assessment instruments (WBI & OI) the old TAW (2002) equations, as depicted above, are still used (Ministerie van Infrastructuur en Milieu, 2017a).

B.2. SWAN wave characteristics relations

This section is largely based and taken over from Klein and Kroon (2011)

Overview Appendix B.1.3 described that Hydra-NL uses a pre-defined database to calculate wave characteristics at specific locations for specific return periods. This section will describe in more detail how this database is created.

A physical model is used to determine the relation of water levels and wind characteristics on one side and wave characteristics on the other side. The physical model which is used is a SWAN model. SWAN (Simulating WAves Nearshore) is a computer program which can simulate wave- propagation, growth, dissipation etc. in a given area with a certain bathymetry and wind field.

Calculation set-up Calculations are performed for many combinations of imposed conditions. Namely 5 different wind velocities, 8 wind directions, 3 phase differences between the peak of the tide and peak of the highest wind velocity, 3 storm surge levels at the edge of the model and 5 time moments around the peak of the storm. The different imposed conditions are summarised in Table B.1.

Potential peak wind velocity [m/s]	Wind direction [°N]	Phase difference [h]	Storm surge [m]	Time moment around peak [h]
20	90	0	0	-2
25	180	4	2	-1
30	210	8	4	0
35	240			1
40	270			2
	300			
	330			
	360			

Table B.1: Overview of the used imposed conditions for the SWAN model of the Wadden Sea region (Klein & Kroon, 2011)

This results in a total of 1800 possible combinations which are all calculated. The above mentioned potential peak wind velocities are translated into an open water wind velocity. The wind fields are spatially uniform, but not temporally uniform. A built-up of wind speed before the peak of the storm and decrease after a storm is taken into account, as well as rotation of wind over time. Bathymetry was received from Deltares and Svasek Hydraulics and has a resolution between 10 m and 200 m.

Next, calculations are performed for 4 different areas. These calculation grids can be seen in Figure B.8. Grid 1 is used to create boundary conditions for the other grids. Grid 2 is used to calculate the wave conditions near the flood defences. Grid 3 is used to better incorporate the effects of long waves entering this area. Grid 4 is used to better incorporate the effects of waves entering this area in between the Wadden islands.

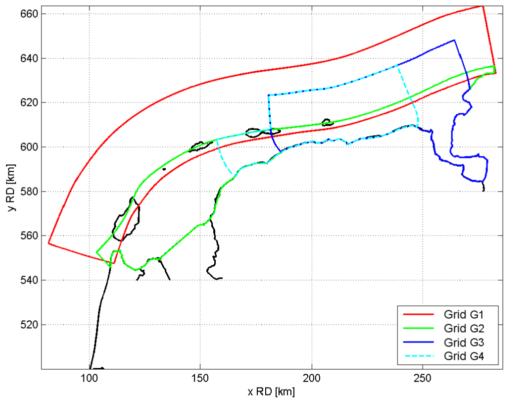


Figure B.8: Calculation grids used for the SWAN calculations (Klein & Kroon, 2011)

Grid 1 provides boundary conditions for the other grids. However, also boundary conditions for grid 1 are needed. This has been done by deriving correlations for wind speed and direction on one side and wave characteristics on the other. These correlations have been determined for measuring platforms in the North Sea (inside grid 1) and are used as boundary conditions for the western, eastern and northern edge of grid 1.

For the calculations, the wave spectra have been discretised. The directional space has been discretised in 36 sectors of each 10°. The frequency domain has a fixed minimum of 0.015 Hz and a maximum of 1 Hz. SWAN then chooses the required frequency bins itself. This resulted in 45 bins. This discretisation has been performed for all 4 grids.

Water levels, flow velocities and flow directions need to be imposed within the grid. The storm surge level at the edge of the model is imposed but will differ per location in the Wadden Sea. The water levels and flow properties have been calculated by Deltares using the WAQUA program.

Next, the SWAN calculations are performed until a steady state is reached. This is defined as following: the wave height in at least 99% the cells cannot change more than 1% between two iterations. For these calculations, SWAN takes into account wave growth and propagation due to wind, wave breaking due to depth and wave steepness, bottom friction and triad- and quadruplet wave interactions. The model does not simulate diffraction, reflection and wave set-up in the breaker zone.

Results For all of the above-mentioned combinations (a total of 1800), the resulting wave characteristics (wave- height, period and direction) near the shore (approximately 100 m from the crest, and 50 m from the toe) are computed. This is done for a total of 1395 points in the Wadden Sea. The points are no more than 250 m apart from each other. This means at least every 250 m data is available for 1800 different combinations of imposed conditions. The locations where data is available are the Hydra-NL output locations as depicted in Figure B.3.

Hydra-NL uses this dataset to find wave characteristics at a specific location for a specific return period. Using statistics of wind speed and direction with the corresponding return period, the wave characteristics at a location can be determined. Since wind speeds are only available every 5 m/s in the SWAN database and wind direction only every 30 degrees, interpolation is used to find wave characteristics for wind values that fall in between the values that are available in the dataset.

Wind speed statistics are used at the West-Terschelling measuring station on the island of Terschelling. Wind direction statistics are used at the Hook of Holland measuring station. It was discussed that calculations have been performed for different phase differences between the peak of the tide and peak of the wind. In the Statutory Assessment Instruments (WBI) it has been decided to use a fixed phase difference of -4.5 hours, meaning the peak of the tide is 4.5 hours before the peak of the surge. This has been done because this phase difference occurs the most during storms (Chbab, 2017).

B.3. BM Gras Buitentalud

BM Gras Buitentalud (Basic Module Grass Outer Slope) is a computer program which is used to evaluate the strength of grass outer slopes with respect to erosion due to waves. After waves have passed a potential foreshore or breakwater they will start impacting on the dike. After the impact, the waves can run-up onto the slope. A dike can both fail due to the impact or due to the run-up of waves. The program can assess one of these mechanisms at a time, but only one is governing. This depends on the location of the grass on the slope.

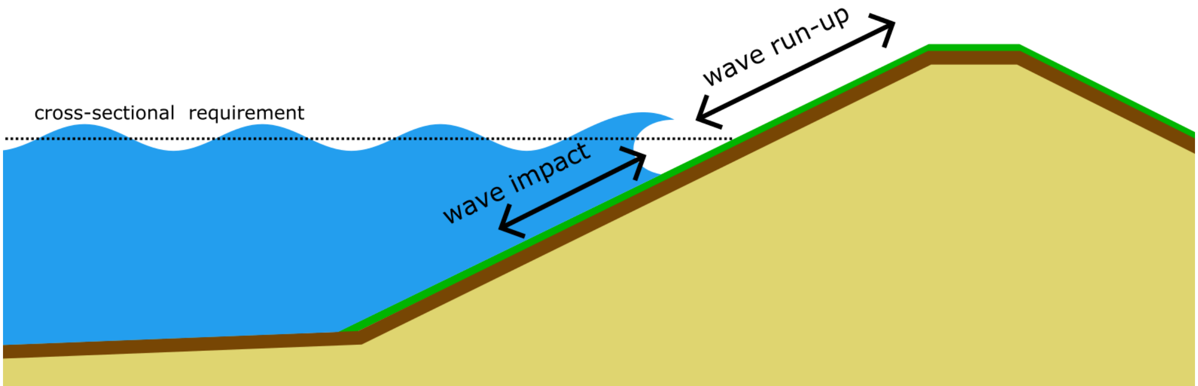


Figure B.9: Area of interest for wave impact and wave run-up on a dike

If a slope has a grass cover which is located below the water level at the cross-sectional requirement (waterstand bij doorsnede-eis), wave impact is the governing mechanism. If there is no grass cover below the water level at the cross-sectional requirement but there is a grass cover above this level, wave run-up is the governing mechanism. See Figure B.9. The water level at the cross-sectional requirement can be calculated with Equation 2.4. As discussed in Section 4.1.3 the maximum allowable failure probability (cross-sectional requirement) is 1/180,000 per year. For all the cases in this report, a grass cover is present in the wave impact zone. Only these calculations are used, and only the wave impact part of *BM Gras Buitentalud* will be explained.

Wave impact Wave impact is being assessed using a lifetime approach. It is assessed how long a certain wave height impacts on a certain part of the slope. Depending on the wave height, a slope is able to withstand the wave attack for a certain amount of time.

In order to calculate the lifetime of a slope, the water levels and wave heights during the storm need to be known. The water levels during a storm are equal to the regular tide plus the storm surge. In the Wadden Sea, this storm surge has a trapezoidal shape as depicted in Figure 4.6. The maximum tide plus the governing surge have to coincide and are equal to the governing water level for a certain return period as determined by Hydra-NL. As discussed in Appendix B.1.3, Hydra-NL can calculate the wave loads given a certain water level for a specific return period and location at the toe of a dike. This means that for a particular situation the course of the water level and wave loads can be calculated.

Next, the slope is cut up in vertical sections (default: 10 cm) and for every time step (default: 15 minutes) it is checked if a wave is impacting on each section. A section is being impacted if it is located between the current water level and half a wave height below that water level (Ministerie van Infrastructuur en Milieu, 2017b).

A section of the slope is able to withstand impacting waves for a particular time. This amount of time is its lifetime. The lifetime is dependent on wave height and grass quality. Equations B.3 and B.4 can be used to calculate the lifetime of a section of slope .

$$H_{m0} > c : t_{damage} = max\left(\frac{1}{b} \cdot ln\left(\frac{H_{m0} - c}{a}\right); 0\right) \tag{B.3}$$

$$H_{m0} < c : t_{damage} = 1000\, hours \tag{B.4}$$

In which:

H_{m0}	=	Significant wave height	[m]
a, b, c	=	Coefficients dependent on grass quality, see Table 4.4	[-]
t_{damage}	=	Time that a section of a slope can withstand the specified wave height	[h]

For example, a closed sod slope (a = 1, b= -0.035, c = 0.25) is able to withstand impacting waves with a wave height of 0.5 m for 40 hours. The failure fraction is the fraction of time a slope has been exposed to a load with respect to the lifetime (exposure time divided by lifetime). A wave height of 0.5 m acting on a closed sod section for 30 hours, results in a failure fraction of 0.75. When the failure fraction is 1 or higher, a failure has occurred. When a calculation is performed, the program checks how long each section is exposed to particular wave heights. Failure fractions of all the different wave heights acting on the slope over time are calculated and summed per section. If the total failure fraction is 1 or higher for any of the sections, the grass cover has been damaged. Additionally, the program is also able to incorporate the residual strength of the first 0.5 m of clay layer underneath the grass cover.

Input and results The user defines a few general parameters like clay layer thickness under the grass cover, time step, section (evaluation) length on the slope and grass quality parameters. Next, the water level development over time during the storm is needed as input. For different hydraulic regions in the Netherlands different water level developments over time are used. Ministerie van Infrastructuur en Milieu (2018) gives an overview. Hydra-NL is then used to find corresponding wave characteristics per water level. Over the total course of the storm, the water level development and wave characteristics are now known. The result is the failure fraction of the most governing section or its safety factor (the inverse of the failure fraction). Note that per calculation only one storm can be analysed, which is representative for a single return period. The program only gives a safety factor as output, not a probability of failure. To find a more exact failure probability the user has to perform a few calculations and iterate towards a safety factor of 1 (the return period where the slope will just not fail).

B.4. Prototype erosion model

This section is largely based and taken over from Rongen et al. (2018)

A prototype computer program developed by HKV is used to determine the erosion of dike bodies when exposed to certain hydraulic loads. The model combines the assessments for wave impact and run-up on the outer slope and erosion of the crest and inner slope due to overtopping. The initial assessment for the failure of the grass cover is done the same way as prescribed in the Statutory Assessment Instruments (WBI) and Design Instruments (OI) (see Appendix B.3 for the outer slope, B.5 for the crest and inner slope). When a grass cover has failed according to these assessment methods, a flood defence does not satisfy the required safety norms. This model, however, also takes into account the residual strength of a dike after a failure of the grass cover on the outer slope (so not the inner slope due to overtopping). Depending on the wave loads, water level, dike geometry and soil type of the dike, the dike will start eroding at a certain location with a certain speed.

Three different materials are distinguished within a dike body in this model. For each material, the erosion speeds are calculated in a different way.

- **Clay with grass roots** This is the top layer of a dike (usually around 0.5 m thick). This layer has more strength than a regular clay layer since grass roots are present which provide additional strength. The equations below give the erosion rate of this clay layer.

$$H_{m0} < 0.5 : t_{RS,grass} = 1000\, hours \tag{B.5}$$

$$H_{m0} > 0.5 : t_{RS,grass} = \frac{min(D_c; 0.5) - 0.2}{c_d(tan(\alpha))^{1.5} max(H_{m0} - 0.5; 0.001)} \tag{B.6}$$

$$c_d = 1.1 + max(0; 8(F_{sand} - 0.7)) \tag{B.7}$$

In which:

$t_{RS,grass}$	=	residual strength of grass	[h]
D_c	=	thickness of the clay layer	[m]
α	=	slope angle	[°]
c_d	=	correction factor for the sand content in clay	[-]
F_{sand}	=	sand content in clay	[-]

It is noted that the equations above provide an erosion resistance in hours and not an eroded volume which the model needs. This means the equations need to be translated to erosion volume for the model to be able to work with it. The total resistance time is set equal to an erosion of the entire clay layer (0.5 m). It can then be calculated what the erosion volume per time step is.

- **Clay** This is the clay in the rest of the dike where grass roots do not provide additional strength. The erosion rate volumes are given in the equations below.

$$\frac{\delta V_e}{\delta t} = c_e[1.32 - 0.079\frac{V_{e0}}{H_s^2}] \cdot [16.4(tan(\alpha))^2] \cdot [min\left(3.6; \frac{0.0061}{s_{op}^{1.5}}\right)] \cdot [1.7 \cdot (H_s - 0.4)^2] \tag{B.8}$$

In which:

c_e	=	erosion coefficient for the clay type	[m³/s]
V_e	=	erosion volume	[m³]
V_{e0}	=	already present and corrected erosion volume in the loaded zone	[m³]
H_s	=	significant wave height	[m]
α	=	dike slope	[°]
T_p	=	wave period of spectrum peak	[s]
s_{op}	=	wave steepness	$[H_s/(1.56 \cdot T_p^2)]$

- **Sand** This is usually the sand core of the dike. The erosion rate can be calculated with the following equation.

$$\frac{\delta V_e}{\delta t} = \frac{H_s^2}{T_p}\left(\frac{0.15}{s_{op}^{1.3}} tan(\alpha)^{0.8}(135 - 1500 \cdot s_{op}) \cdot exp\left(-0.0091 \cdot \left(\frac{B_t}{H_s}\right)^2\right)\right) \tag{B.9}$$

In which:

B_t = width of the erosion terrace (shallow part of the erosion profile) [m]

The erosion volume equation used depends on which layer is exposed. The erosion equation of the deepest layer that is exposed within a section where a wave is impacting is used. This means the sand erosion equation is used even if only a centimetre of sand is exposed. This erosion volume is then removed from the dike profile, also from the part that is still covered by clay. This means this is a conservative approach.

The soil erodes from a dike in a particular shape. A triangle gets eroded from the dike. It has a bottom slope of 1:8 and a landward slope of 1:1. See Figure B.10 for a schematisation.

With a constant water level the eroded profile of a dike over time will look like in Figure B.11

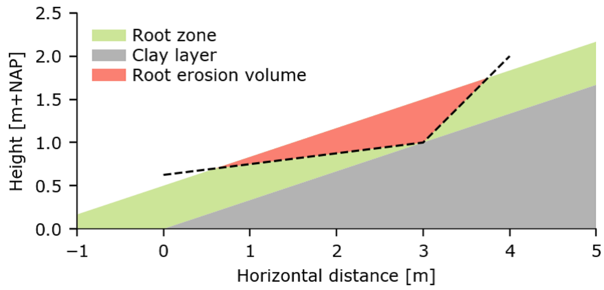


Figure B.10: Shape of the eroded volume (Rongen et al., 2018)

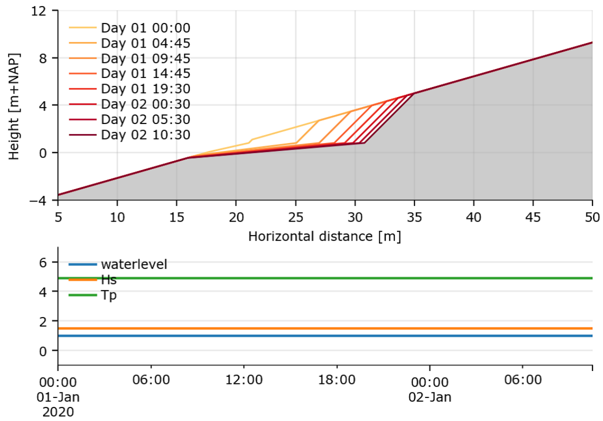


Figure B.11: Erosion of a dike profile over time with a constant water level, wave height and wave period (Rongen et al., 2018)

B.5. RisKeer (Ringtoets)

RisKeer is a probabilistic assessment program which can be used to assess flood defences. Similar to Hydra-NL, RisKeer can calculate hydraulic loads for specific return periods along all the primary Dutch flood defences. The program is however only able to perform calculations for the year 2023. Additionally, RisKeer is able to assess flood defences for a few failure mechanisms. In this report, RisKeer is used to assess the failure mechanism erosion of the inner slope and/or crest due to overtopping.

Hydraulic loads The hydraulic loads are determined in the same way as with Hydra-NL, so no further explanation will be given. The difference between Hydra-NL and RisKeer is that both programs use other probabilistic methods to determine the hydraulic loads for a given return period from the statistics. RisKeer uses Hydra-Ring as a computational core. For the Wadden Sea region FORM with directional sampling is used as calculation method (Chbab & de Waal, 2017). Hydra-NL, however, uses numerical integration (Slomp, 2016).

Erosion of the crest and inner slope When the combination of water levels and waves is high enough overtopping can occur. When these overtopping volumes get too large, the grass cover on the crest and inner slope can start eroding due to the large flow velocity of the overtopping water. Depending on the wave height and quality of the grass, the crest and inner slope are able to withstand a certain critical overtopping volume before it fails. This critical overtopping volume is log-normally distributed with means and standard deviations as given in Table B.2.

In the assessment, the probability of occurrence of the overtopping volume (dependent on the statistics of wind and water levels) is compared to the probability of occurrence of the critical overtopping volume (Ministerie van Infrastructuur en Milieu, 2017a). If the overtopping volume is higher than the critical overtopping volume, a failure occurs. The probability that the overtopping volume is larger than the critical overtopping cannot be larger than the allowable probability of failure due to this mechanism.

Wave height class	Closed sod		Open sod	
	μ (l/s/m)	σ (l/s/m)	μ (l/s/m)	σ (l/s/m)
0-1 m	225	250	100	120
1-2 m	100	120	70	80
2-3 m	70	80	40	50

Table B.2: Mean (μ) and standard deviation (σ) for a log-normal distribution of critical overtopping volume for different sod qualities and wave height classes (Ministerie van Infrastructuur en Milieu, 2018)

B.6. D-Stability

The computer program D-Stability is used to assess the safety of dike bodies with respect to macro-stability.

The user needs to provide information about the geometry of the dike, the soil composition and the soil characteristics. In this case, all this data was gathered from design reports. Next, the user needs to specify the hydraulic heads. This can be generated automatically by the program if a few details are known. These are for example the regular water levels (both on the polder and water side), the water level in the specific scenario which is assessed, the intrusion length and the leakage length.

Before the program can find the governing slip circle a calculation method has to be chosen. The four options are Bishop, Spencer, Fellenius and UpliftVan. Each method uses a different manner to find the governing slip circle. In this report, UpliftVan is used. This model has more options to find the governing slip circle which can lead to slip circles occurring more easily.

Calculations can be performed for a single scenario (with only one specific water level) and result in a factor of safety. The factor of safety can be translated to a probability of failure per year. It can then be determined if the probability of failure is less than the maximum allowable probability of failure. For more information about this program see Brinkman, Nuttall, Noordam, and Teixeira (2018).

C
Detailed Results

C.1. Overtopping results without erosion

See next page.

2023

Return period [1/year]		10	100	300	1000	3000	10000	12500	37500	100000
Profile	Output loc									
76	5	0	0	0	0	0	0	0.113	0.991	4.833
77	6	0	0	0	0	0	0.153	0.225	1.35	5.165
77	7	0	0	0	0	0	0.171	0.25	1.38	5.126
78	8	0	0	0	0	0	0.158	0.246	1.85	8.521
78	9	0	0	0	0	0	0	0	0.44	2.724
79	9	0	0	0	0	0	0	0	0.309	1.887
80	12	0	0	0	0	0	0	0	0	0
81	12	0	0	0	0	0	0	0	0	0
82	12	0	0	0	0	0	0	0	0	0.765
82	13	0	0	0	0	0	0.192	0.295	1.92	7.459
83	13	0	0	0	0	0	0	0	0.569	3.118
83	14	0	0	0	0	0	0	0	0.146	1.19
84	15	0	0	0	0	0	0	0	0	0
84	16	0	0	0	0	0	0	0	0	0.134
84	17	0	0	0	0	0	0	0	0	0.177

2050

Return period [1/year]		10	100	300	1000	3000	10000	12500	37500	100000
Profile	Output loc									
76	5	0	0	0	0	0.107	1.215	1.805	9.809	36.903
77	6	0	0	0	0	0.201	1.527	2.099	8.502	24.752
77	7	0	0	0	0	0.217	1.525	2.11	8.265	23.755
78	8	0	0	0	0	0.181	1.856	2.745	15.131	56.524
78	9	0	0	0	0	0	0.515	0.801	5.83	27.406
79	9	0	0	0	0	0	0.355	0.549	3.857	17.314
80	12	0	0	0	0	0	0	0	0	0
81	12	0	0	0	0	0	0	0	0	0.828
82	12	0	0	0	0	0	0.14	0.249	1.8	7.913
82	13	0	0	0	0	0.25	2.112	2.962	12.383	38.413
83	13	0	0	0	0	0	0.634	0.972	5.87	24.449
83	14	0	0	0	0	0	0.192	0.328	2.655	13.127
84	15	0	0	0	0	0	0	0	0	0
84	16	0	0	0	0	0	0	0	0.335	2.179
84	17	0	0	0	0	0	0	0	0.437	2.709

Figure C.1: Overtopping volumes [l/s/m] for different return periods per profile and Hydra output location.

C.2. Safety factors erosion outer slope

2023 Open

Return period [1/year]		10	100	300	1000	3000	10000	60000	180000	600000
Profile	Output loc									
76	5	INVALID	INVALID	INVALID	INVALID	1.063	0.325	0.203	0.18	0.153
77	6	10	0.743	0.56	0.4	0.34	0.296	0.163	0.111	0.079
77	7	10	0.644	0.524	0.376	0.325	0.261	0.153	0.109	0.078
78	8	10	0.618	0.508	0.365	0.31	0.251	0.147	0.097	0.07
78	9	10	0.712	0.555	0.421	0.34	0.3	0.187	0.114	0.078
79	9	10	0.911	0.574	0.424	0.349	0.302	0.187	0.115	0.079
80	12	10	10	10	10	10	10	2.946	1.31	1.309
81	12	10	4.757	3.179	1.935	0.866	0.628	0.408	0.218	0.149
82	12	10	8.503	6.283	3.08	1.311	1.081	0.397	0.215	0.146
82	13	10	0.932	0.633	0.525	0.45	0.334	0.223	0.13	0.092
83	13	10	0.938	0.623	0.529	0.448	0.33	0.197	0.121	0.086
83	14	10	0.943	0.633	0.517	0.485	0.334	0.214	0.128	0.091
84	15	10	2.638	1.31	0.664	0.48	0.416	0.278	0.144	0.107
84	16	10	1.72	0.944	0.617	0.447	0.389	0.282	0.158	0.109
84	17	10	1.565	0.957	0.615	0.451	0.394	0.305	0.152	0.105

2023 Closed

Return period [1/year]		10	100	300	1000	3000	10000	60000	180000	600000
Profile	Output loc									
76	5	INVALID	INVALID	INVALID	INVALID	1.063	0.325	0.203	0.18	0.153
77	6	10	1.76	0.56	0.4	0.34	0.296	0.163	0.111	0.079
77	7	10	0.745	0.524	0.376	0.325	0.261	0.153	0.109	0.078
78	8	10	0.618	0.508	0.365	0.31	0.251	0.147	0.097	0.07
78	9	10	2.087	0.555	0.421	0.34	0.3	0.187	0.114	0.078
79	9	10	2.918	0.574	0.424	0.349	0.302	0.187	0.115	0.079
80	12	10	10	10	10	10	10	6.254	3.477	3.993
81	12	10	9.454	6.837	4.982	3.127	1.85	0.408	0.218	0.149
82	12	10	10	10	8.018	4.446	1.714	0.397	0.215	0.146
82	13	10	3.699	0.795	0.525	0.45	0.334	0.223	0.13	0.092
83	13	10	3.472	0.623	0.529	0.448	0.33	0.197	0.121	0.086
83	14	10	3.761	0.897	0.517	0.485	0.334	0.214	0.128	0.091
84	15	10	6.079	4.253	2.185	0.572	0.416	0.278	0.144	0.107
84	16	10	4.655	3.384	1.107	0.459	0.389	0.282	0.158	0.109
84	17	10	4.502	3.453	1.009	0.464	0.394	0.305	0.152	0.105

Figure C.2: Factors of safety[1] due to wave impact for 2023 for different combinations of sod qualities, return period, profiles and Hydra output locations as generated by *BM Gras Buitentalud*.

[1] INVALID appears at the cases where wave impact is not governing since the transition of stone revetment and grass cover slope is above the still water level

2045 Open

Return period [1/year		10	100	300	1000	3000	10000	60000	180000	600000
Profile	Output loc									
76	5	INVALID	INVALID	INVALID	0.7	0.361	0.272	0.194	0.154	0.14
77	6	3.594	0.8	0.538	0.395	0.265	0.139	0.092	0.075	0.055
77	7	3.09	0.662	0.454	0.388	0.257	0.134	0.086	0.069	0.054
78	8	2.703	0.648	0.449	0.377	0.22	0.124	0.075	0.062	0.05
78	9	2.951	0.647	0.547	0.455	0.273	0.144	0.084	0.07	0.055
79	9	2.951	0.708	0.55	0.458	0.274	0.144	0.085	0.071	0.056
80	12	10	10	10	10	5.841	3.076	2.588	2.309	1.443
81	12	10	3.597	2.89	1.181	0.633	0.275	0.152	0.12	0.108
82	12	10	7.267	4.182	1.92	0.625	0.271	0.149	0.119	0.103
82	13	3.941	0.744	0.651	0.452	0.329	0.153	0.099	0.079	0.066
83	13	4.542	0.789	0.608	0.459	0.295	0.146	0.09	0.074	0.061
83	14	4.653	0.815	0.67	0.462	0.326	0.16	0.099	0.08	0.064
84	15	5.66	1.022	0.793	0.557	0.409	0.184	0.105	0.083	0.067
84	16	4.599	1.055	0.672	0.509	0.398	0.198	0.111	0.09	0.073
84	17	4.987	1.022	0.669	0.541	0.404	0.196	0.108	0.087	0.071

2045 Closed

Return period [1/year		10	100	300	1000	3000	10000	60000	180000	600000
Profile	Output loc									
76	5	INVALID	INVALID	INVALID	0.7	0.361	0.272	0.194	0.154	0.14
77	6	7.649	0.82	0.538	0.395	0.265	0.139	0.092	0.075	0.055
77	7	6.831	0.724	0.454	0.388	0.257	0.134	0.086	0.069	0.054
78	8	6.061	0.648	0.449	0.377	0.22	0.124	0.075	0.062	0.05
78	9	6.469	0.969	0.547	0.455	0.273	0.144	0.084	0.07	0.055
79	9	6.469	0.984	0.55	0.458	0.274	0.144	0.085	0.071	0.056
80	12	10	10	10	10	10	5.642	4.993	5.078	4.436
81	12	10	7.505	6.499	3.922	1.275	0.283	0.152	0.12	0.108
82	12	10	10	10	7.238	1.132	0.278	0.149	0.119	0.103
82	13	8.201	2.463	0.651	0.452	0.329	0.153	0.099	0.079	0.066
83	13	9.325	1.206	0.608	0.459	0.295	0.146	0.09	0.074	0.061
83	14	9.497	2.498	0.754	0.462	0.326	0.16	0.099	0.08	0.064
84	15	10	3.605	2.755	0.76	0.409	0.184	0.105	0.083	0.067
84	16	9.413	3.768	2.256	0.614	0.398	0.198	0.111	0.09	0.073
84	17	10	3.627	2.019	0.655	0.404	0.196	0.108	0.087	0.071

Figure C.3: Factors of safety due to wave impact for reference year 2045 for different combinations of sod qualities, return period, profiles and Hydra output locations as generated by *BM Gras Buitentalud*.

C.3. Results erosion model

See next page.

Overtopping m³/m

Return period [1/year]		1000	2000	3000	5000	6000	7000	8000	9000	10000	15000	20000	25000	30000	35000	40000	45000	50000	55000	60000	180000	600000
Profile	Output loc																					
76	5	0.00017	0.00292	0.02381	0.07386	0.15106	0.17993	0.14396	23092.2	26609.5	48969.9	70778.6	93249.3	96852.9	101891	104298	107566	113878	115361	123620	158975	199626
77	6	0.04005	0.28639	0.61816	1.83389	2.05456	3.4993	3.28476	3.98199	4.94122	7.01888	14.0244	11996.8	13186.3	17565.6	20831.9	24687.1	27650.6	36762.5	43034.9	72385.6	118662
77	7	0.04503	0.31517	0.65846	1.29667	2.29075	2.4734	3.17138	3.92674	4.05693	5.98024	10.6568	6957.39	7798.43	9318.33	11220.4	16012.2	14161.7	16744.8	21966.9	67291.4	109270
78	8	0.03326	0.28948	0.53436	1.18876	2.34907	2.59031	2.37173	5.99149	4.88995	7.77743	15.6246	24.0564	32.3868	38.0505	41.4257	62597.3	64273.6	63856.3	66495	102699	146543
78	9	0.00256	0.03011	0.11272	0.11253	0.46282	0.36002	0.43384	2.34148	0.85263	2.15103	3.83346	11.9235	8.59675	18.923	14.041	24.6716	33.6609	25.1349	23.9707	64276.8	97465.3
79	9	0.00065	0.00539	0.0468	0.2114	0.6806	0.53571	1.20558	2.506	1.08926	18968.8	28316.3	35395	47587.7	53350.1	52723	55334.6	59165.9	58628.2	58980.5	107125	149702
80	12	9E-29	5.4E-25	6E-23	9.8E-22	2E-20	3.6E-20	3.6E-19	2.5E-18	1.4E-17	8.3E-15	5.7E-13	1E-11	1.1E-10	6.7E-10	4E-09	1.5E-08	4.6E-08	1.3E-07	3.1E-07	0.00088	0.04498
81	12	1.3E-09	2.7E-07	4.4E-06	0.00014	0.00029	0.00086	0.0002	0.00044	0.00095	8.7E-05	0.00019	0.00042	0.00114	0.01664	0.0051	0.00708	0.01818	0.01263	0.016	0.29295	7.44075
82	12	4.5E-05	0.00033	0.00094	0.00823	0.00777	0.09241	0.13249	0.06908	0.22885	0.67092	1.23539	1.27304	3.53435	3.19705	5.802	7.49842	6.76487	10.8977	11.7147	88.1011	28581.1
82	13	0.02996	0.15585	0.54644	0.83398	1.38944	1.53357	3.01319	2.66123	4.6559	9.01362	15.1575	15.5296	27.0963	23.9959	27.6123	45.0287	38.5534	56.3088	64.9616	55159.8	80609
83	13	0.00013	0.00786	0.0936	0.04041	0.52377	0.18073	0.63999	0.63501	1.08748	2.44744	4.6476	7.22377	10.8719	15.0918	12.4434	23.8678	25.2681	52724.2	53723.9	79386.2	118413
83	14	5.5E-05	0.00232	0.01902	0.00567	0.06815	0.01713	0.16773	0.0384	0.13857	0.79848	1.71926	1.60002	4.07936	4.65654	5.32876	13.3755	9.0591	15.1441	17.3767	58881.5	100671
84	15	0.00023	0.00589	0.02725	0.15112	0.21605	0.30354	0.55295	0.69155	0.84341	1.99772	3.92362	5.96253	9.32306	11.318	13.6729	16.1263	22.386	25.1932	27.751	49820	78969.8
84	16	8.7E-05	0.00303	0.01926	0.07832	0.14992	0.20046	0.36525	0.48345	0.58641	1.43648	2.78793	4.84798	6.79301	10.0926	11.9235	14.9232	20.2312	22.9764	25.634	56312.1	95019.9
84	17	0.0003	0.00837	0.04171	0.20139	0.38246	0.51151	0.66947	1.09329	1.29054	2.6034	4.77116	7.69529	9.96561	14.5121	17.1433	23.0906	27.2501	30.4546	33.4914	59791.8	95522

Erosion m³/m

Return period [1/year]		1000	2000	3000	5000	6000	7000	8000	9000	10000	15000	20000	25000	30000	35000	40000	45000	50000	55000	60000	180000	600000
Profile	Output loc																					
76	5	14.4118	17.295	18.9303	22.5825	23.1002	24.0804	24.6409	173.195	173.791	177.808	180.219	183.337	187.12	188.989	189.384	192.824	196.282	201.499	207.956	242.568	312.52
77	6	9.93814	12.9709	14.3218	18.536	19.425	19.6726	20.3076	20.6484	21.0134	22.7562	23.907	163.93	163.253	165.37	164.736	164.899	165.701	167.312	167.992	172.217	208.823
77	7	10.3741	13.7039	14.7413	18.8446	19.2585	20.0642	20.3682	21.1054	21.2262	22.6298	23.0438	156.048	156.077	154.622	156.155	158.424	155.907	157.321	159.161	162.905	188.478
78	8	15.1243	18.4427	20.2496	22.3624	23.682	24.1062	24.5786	25.3286	26.2714	28.2859	29.8655	31.4823	31.8451	33.1693	33.5454	184.14	182.574	183.929	189.233	201.746	234.809
78	9	11.514	14.3571	15.5298	17.5883	18.2959	18.8613	19.1181	19.8449	20.0283	21.6185	23.2436	24.0141	25.0134	26.1455	26.011	27.2776	27.9907	27.9542	28.2257	176.564	184.867
79	9	13.0927	18.4219	20.4174	23.5955	24.9917	25.364	26.4737	26.9469	27.6552	152.661	157.668	156.563	162.115	165.287	163.072	162.95	168.591	168.212	169.46	180.919	227.778
80	12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
81	12	0	0.64939	1.07255	1.4651	2.33535	2.55298	5.24537	5.58084	6.6331	6.49084	6.83521	7.27436	8.73068	10.3872	9.98567	9.69985	10.9382	10.9704	10.7277	13.1636	18.244
82	12	0	0.55923	0.97723	1.81654	2.63954	5.10641	6.32677	6.5446	6.88238	7.8999	8.60979	9.59608	9.85013	10.1526	10.6091	10.729	10.9642	11.1195	11.2445	16.9817	137.132
82	13	8.54029	10.5473	11.2292	13.8873	14.4444	14.8149	15.3643	15.7554	16.1357	17.5493	18.6235	19.0143	20.2427	20.6075	20.7036	21.839	22.0315	22.5502	22.8693	160.819	169.666
83	13	9.98806	13.5304	14.7628	16.3817	17.0382	17.8908	18.8462	19.2031	19.7003	21.4185	22.9098	23.5767	24.8701	25.2237	25.5946	26.9627	27.0661	154.737	155.208	169.545	186.959
83	14	9.22854	11.0459	12.6441	13.9429	14.7051	15.1464	15.6391	16.1532	16.6026	18.5838	19.7782	20.3235	21.3086	21.5831	21.9988	22.7127	22.9475	23.8379	24.0041	154.377	171.191
84	15	7.43479	9.69277	10.6875	12.9241	13.2901	13.8882	14.2557	14.4996	14.8242	16.1754	17.4147	18.1488	18.8398	19.3604	19.9548	20.4106	20.743	20.9873	21.6279	159.712	172.448
84	16	7.42667	9.14568	10.3627	11.3923	11.7463	11.9438	14.1354	14.5128	14.8662	16.101	17.1427	18.1748	18.8491	19.6391	20.1562	20.5889	21.1174	21.5224	21.972	150.618	167.212
84	17	8.40293	10.2745	11.1236	13.5988	14.2807	14.6686	14.9711	15.496	15.8001	17.1005	18.4771	19.4173	20.0094	20.898	21.0916	21.8079	22.1514	22.7089	22.9806	155.174	168.474

Figure C.4: Total overtopping- and erosion volume after a storm for different return periods per combination of profile and Hydra-NL output location. Including effects of erosion. Current situation.

Overtopping m³/m

Return period [1/year]		1000	2000	3000	5000	6000	7000	8000	9000	10000	15000	20000	25000	30000	35000	40000	45000	50000	55000	60000	180000	600000
Profile	Output loc																					
76	5	0.0136	0.24878	57024.4	78916.4	92178.4	96344.5	107224	108649	113095	126697	147004	154694	162267	170958	164809	170226	180622	184075	187134	238041	282010
77	6	0.85125	2.70228	6.26715	12.0772	25046.1	17.8229	44921.8	45360.3	47165.6	55986.8	61794.7	65586	74971.2	74605	76913.3	86429.4	89972.9	90065.1	101555	149149	177735
77	7	0.66374	3.33418	7.17209	12.0518	17.4629	18.0742	47668.3	47602.6	48894.1	57234.4	59922.2	62893.5	75016.4	71927.5	73365.1	83099	85320.3	85705.6	97137.1	140708	175803
78	8	0.77902	5.72712	5.51789	9.8911	16.2386	16.806	47.3222	31.1182	36.1559	69475.2	83665.9	90411.8	95258.7	97651.2	100072	111043	108485	107720	114849	179025	232282
78	9	0.08886	1.02843	1.19248	3.41544	4.76208	6.48343	11.3783	10.5625	13.3903	34.7807	47.255	60.1143	64529.8	65250.1	66493.7	73299	74843.6	73275.1	82635.3	110739	152468
79	9	0.09355	1.22526	1.7211	4.43141	50913	51223.2	56849.3	57904.6	58612.8	66418	72476.8	83030.5	96379.5	94216.9	93991.1	114751	112903	110781	127355	178800	236249
80	12	2.4E-24	4.8E-19	4.1E-16	8.8E-13	9.7E-12	7.1E-11	3.7E-10	1.6E-09	5.6E-09	6.1E-07	1E-05	5.3E-05	0.00015	0.0003	0.00051	0.00082	0.00125	0.00185	0.00264	0.0314	3.45022
81	12	3.3E-05	0.00182	2.7E-05	0.00026	3.4E-05	0.0005	0.00373	0.00335	0.00801	0.00386	0.02155	0.06412	0.03759	0.04297	0.13568	0.09269	0.11645	0.15045	0.37256	9.02926	225.738
82	12	0.00073	0.04683	0.14757	1.07066	0.97481	2.61462	2.22307	2.8171	5.3604	10.3609	15.0926	20.1791	29.5297	38.9869	61.2618	79.3689	85.6076	129.991	136.588	369.584	72153.7
82	13	0.52226	1.67663	4.32816	13.1867	16.4656	23.8354	24.2321	28.4508	39.202	51.396	91.893	102.727	141.087	166.875	59069.4	59438.5	60835.1	67524.9	69060.8	100482	128771
83	13	0.01731	0.25375	0.83211	4.54438	4.63496	7.06777	8.14465	10.0163	24.1562	58073.1	63150.2	66445	72699.5	75450.2	85094.5	85981.7	88772.2	95092.4	94711.5	126366	179789
83	14	0.00204	0.02192	0.14002	2.90839	1.64089	3.94735	3.32064	4.53014	11.7564	14.5579	35.5301	32.8737	59038.7	58696.4	63175.4	64508.3	66537.7	73031.4	73752.5	105867	146883
84	15	0.07922	0.93752	2.07395	6.44911	8.22466	10.035	14.859	17.2122	19.6766	38.3153	66.5359	86.64	54064.8	55166.7	57232.5	60307.2	64861.7	66868	68448.3	93102.4	127503
84	16	0.03922	0.32562	1.41067	3.75828	7.0071	7.88759	10.373	14.3506	16.5899	31.6115	44667.6	51216.3	55042.2	57625	60297.9	68889.7	71613.3	72909.8	75870.2	99393.4	134715
84	17	0.08844	0.82814	2.48509	6.23873	9.42802	11.7441	14.0325	18.7688	21.3664	38.8591	48319.4	55252.3	57608.7	60359.3	63715	72684.6	74708.1	77884.5	79758.7	102067	139054

Erosion m³/m

Return period [1/year]		1000	2000	3000	5000	6000	7000	8000	9000	10000	15000	20000	25000	30000	35000	40000	45000	50000	55000	60000	180000	600000
Profile	Output loc																					
76	5	23.3486	26.8573	178.601	181.487	183.399	185.254	187.514	188.412	194.873	201.305	217.043	223.139	233.069	232.485	235.257	241.294	243.557	246.688	253.794	283.432	342.512
77	6	14.1505	20.5873	22.2907	25.5634	165.267	27.2347	170.92	171.03	171.659	178.08	182.074	184.374	188.498	193.966	187.449	194.152	190.436	192.339	194.796	222.726	277.736
77	7	17.7968	21.148	23.9445	25.6756	26.6272	27.4473	171.217	171.135	171.699	176.947	179.095	181.528	187.494	190.28	194.244	192.861	189.539	192.346	196.081	221.212	261.177
78	8	20.7695	24.7543	26.8899	29.2708	30.9075	31.4336	32.5787	33.5687	33.8456	187.726	193.135	196.137	196.324	195.31	199.866	204.53	199.144	203.577	208.578	245.807	314.274
78	9	15.7785	19.0562	20.5545	22.4991	23.8108	24.1624	24.6383	25.7319	25.9667	28.0275	29.1838	30.9453	168.961	170.296	171.363	174.057	176.454	175.782	184.074	188.073	217.722
79	9	20.8266	25.6104	28.2741	30.715	161.117	161.344	166.131	165.568	167.437	171.044	176.881	177.811	177.456	185.029	178.624	184.376	187.997	181.566	196.543	246.946	303.2
80	12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
81	12	1.31133	1.81667	5.43458	6.5791	6.80955	7.12752	9.61803	9.52474	10.8033	11.3304	11.1916	11.8419	12.1321	12.4367	12.8795	13.1026	13.2853	13.615	14.0081	18.8415	28.6514
82	12	1.30228	4.97987	6.85216	8.22513	8.74165	9.30803	9.91886	10.1473	10.3678	11.474	11.9315	12.501	15.4269	15.7895	16.4465	16.6391	16.8318	17.3362	17.7512	22.7797	150.426
82	13	11.6054	15.2687	16.4405	18.7927	19.6508	20.4702	20.8752	21.2796	22.0383	23.4705	24.8977	25.8465	26.8141	27.0577	165.66	165.421	165.731	170.248	172.654	182.315	206.148
83	13	15.271	18.7115	20.2679	22.3408	23.1952	24.0436	24.8174	25.0775	26.0228	155.603	161.886	166.02	165.678	166.305	169.518	175.175	169.746	175.41	170.79	188.131	251.087
83	14	13.0964	15.25	16.9843	19.1629	19.9343	20.2067	21.3345	21.4143	22.3677	24.0833	25.6558	25.7866	154.19	153.081	158.485	157.512	159.901	158.048	160.044	171.853	190.042
84	15	11.1182	14.7069	16.1137	18.1811	18.7857	19.1854	19.9458	20.2905	20.5868	22.3033	23.7943	24.4235	158.242	158.725	158.107	159.444	160.838	161.801	161.668	174.622	200.949
84	16	11.1652	12.5347	15.8627	17.4565	18.2724	18.7884	19.8094	20.4148	20.9415	22.4711	145.179	147.656	147.729	148.862	150.309	153.264	152.674	153.644	152.869	165.657	209.252
84	17	11.982	15.2813	16.8046	19.1653	19.9997	20.596	20.9358	21.4969	22.067	23.6873	147.927	150.831	151.82	153.872	157.178	156.936	157.068	159.825	158.618	174.931	220.507

Figure C.5: Total overtopping- and erosion volume after a storm for different return periods per combination of profile and Hydra-NL output location. Including effects of erosion. Reference year 2045.

C.4. Results sensitivity analysis

See next page.

Return period	Average	a+2sigma	a+sigma	a-2sigma	a-sigma	Clay+2sigma	Clay+sigma	Clay-2sigma	Clay-sigma	Fsand07	Fsand08	Fsand09	Fsand10	Grass+2sigma	Grass+sigma	Grass-sigma	Crest5.5m	Crest6m
1000	0.463576	0	0	9.8642933	9.33007	0.4635755	0.463575506	0.463575506	0.463575506	0.46357551	1.087853558	1.712131611	2.336409663	0.962997948	0.71328673	0.2138643	0.463576	0.46358
3000	1.711736	0	0	12.42799	12.2137	1.7117361	1.711736059	1.711736059	1.711736059	1.71173606	4.592529685	5.368625622	5.835626527	4.251154135	2.63851687	0.7849553	1.711736	1.71174
5000	3.235316	0	0	14.894586	14.6878	3.2353156	3.235315568	3.235315568	3.235315568	3.23531557	8.374111607	10.21028319	10.82016676	7.872567871	6.60291797	1.4821866	3.235316	3.23532
10000	7.398413	0	0	17.142579	16.9706	9.3426963	8.385667825	5.332901772	6.381071773	7.39841346	12.54538488	15.09016205	15.21408837	11.61306571	9.77490059	2.0431263	7.398413	7.39841
20000	13.11176	0	0	141.40267	19.5694	15.934995	14.5531222	10.03863312	11.6072148	13.111763	141.3058299	141.802404	141.9819537	141.1434503	22.8598209	3.8967994	13.11176	13.1118
30000	17.06046	0	1.3277	148.10616	148.735	21.491155	19.33866567	12.07178584	14.64049904	17.0604627	149.229122	149.6216993	150.4061339	148.9875249	24.5503523	4.3500901	17.06046	17.0605
40000	23.86166	0	1.6812	150.29492	150.1	31.040105	27.62187516	15.03705394	19.66288585	23.8616607	151.4038406	150.8960016	152.1576607	149.5184661	149.720405	4.6973842	23.86166	23.8617
50000	25.14587	0	1.9283	153.11306	152.876	144.14472	144.0883383	15.98363907	20.79759079	25.14587	155.5576444	157.7691884	155.4831642	159.3335747	153.858829	8.3329106	25.14587	25.1459
60000	21.99647	0	2.2544	155.2185	154.965	146.52431	147.7978405	15.05792125	18.63864629	21.9964716	160.5784824	164.138036	158.9710677	160.1577347	157.849147	8.9643259	21.99647	21.9965
70000	26.73576	0	2.2935	158.14132	157.827	149.42039	148.1396751	16.99589132	22.1267117	26.7357579	160.2505567	162.4230949	159.1639557	159.4822308	158.851217	9.1526061	26.73576	26.7358
80000	27.07866	0	2.5472	161.5171	161.094	149.63827	149.5419549	17.26827803	22.43658736	27.0786599	164.4981082	162.2552324	163.4967012	163.3334741	160.56017	9.8685372	27.07866	27.0787
90000	151.9876	0	2.9527	161.40393	161.061	152.01424	152.9180277	17.51681192	22.76781455	151.987614	160.9223928	162.5103539	163.4208533	168.6138278	164.911671	9.8911814	151.9876	151.988
100000	149.8181	0	3.0471	160.69335	160.263	155.43043	150.2256241	17.90370834	23.26593861	149.818122	163.4154704	165.5530929	166.491288	162.6122892	161.737385	9.9948562	149.8181	149.818
110000	161.6089	0	6.6964	162.47316	161.741	172.86216	162.0419833	19.94057135	154.6159659	161.60895	164.8259794	167.5631037	168.3184519	163.8242293	161.442889	10.120417	161.6089	161.609
120000	166.3674	0	6.3494	160.25571	160.072	175.46826	163.0442094	20.64079214	165.0798947	166.367422	163.734177	164.8996352	165.3070056	162.9097665	167.629341	10.354279	166.3674	166.367
130000	159.6204	0	6.8687	162.48247	162.243	177.35123	165.7370635	20.92060718	163.238375	159.620405	166.7469365	167.7506225	168.2758783	165.6385725	172.549138	10.435475	159.6204	159.62
140000	160.4604	0	7.0162	165.26852	165.288	178.51683	165.8165664	20.86911842	164.5973124	160.460395	168.4594858	170.2066579	170.6591793	167.105119	165.36059	10.891184	160.4604	160.46
150000	160.9927	0	9.8217	166.17377	166.032	178.4699	166.6539241	20.69842434	158.4309715	160.992737	169.5858613	172.719356	173.5284606	168.1864142	166.050175	10.988365	160.9927	160.993
170000	169.9678	0	11.94	164.49453	165.131	193.00577	185.6996786	161.0924733	166.314581	169.967803	167.1262693	166.8652189	167.1207829	166.3932067	165.070136	11.201528	169.9678	169.968
180000	172.5944	0	12.134	166.96553	166.836	187.14082	176.3334689	167.2050863	168.4834651	172.594439	168.4075496	170.7915166	171.1105548	168.1384611	167.157354	11.38649	172.5944	172.594
600000	217.4441	0	168.41	220.20376	220.023	216.92562	210.1911768	171.7961042	185.0237832	217.444077	210.7715146	228.1205903	225.8462527	210.5738836	210.004813	14.698419	217.4441	217.444

Figure C.6: Erosion volume after a storm (m^3/m) per return period for different parameter modifications (ID 6)

Return period	ClayLayer+20cm	ClayLayer-20cm	Sand+2sigma	Sand+sigma	Sand-2sigma	Sand-sigma	Terrace+2sigma	Terrace+sigma	Terrace-2sigma	Terrace-sigma	Open_a+2sigma	Open_a+sigma	Open_a-2sigma	Open_a-sigma	Open_Average	SLR+70cm	SLR+15cm
1000	0.463575506	0.463575506	0.46357551	0.4635755	0.4635755	0.4635755	0.463575506	0.463575506	0.463575506	0.46357551	0	2.415347264	9.911836629	9.864293263	8.24234093	0	0
3000	1.711736059	1.711736059	1.71173606	1.7117361	1.7117361	1.7117361	1.711736059	1.711736059	1.711736059	1.71173606	0.717035134	7.671521739	12.44987707	12.4279898	10.9061082	0.739611	0
5000	3.235315568	3.235315568	3.23531557	3.2353156	3.2353156	3.2353156	3.235315568	3.235315568	4.850602806	3.23531557	1.310225091	9.830141327	14.91668821	14.89458584	12.9287557	0.934954	0
10000	7.39841346	7.39841346	7.39841346	7.3984135	7.3984135	7.3984135	6.870676804	7.148210777	7.632632353	7.40597805	1.689655531	11.50393127	17.20423128	17.17394655	16.7641321	1.738347	0
20000	13.11176298	13.11176298	13.111763	13.111763	13.111763	13.111763	13.03262766	13.07272386	16.39480957	13.144436	1.988080412	13.5089421	141.4541353	141.4541353	19.3589976	7.661964	3.537671
30000	17.06046274	17.06046274	17.0604627	17.060463	17.060463	17.060463	14.35602578	14.12082638	22.89407568	17.2792731	2.608307266	17.28473484	148.200703	148.173588	148.41893	17.48744	3.759993
40000	23.8616607	23.8616607	23.8616607	23.861661	23.861661	23.861661	18.27698024	18.36583797	24.30908785	24.1331107	2.85865095	23.8616607	150.3141265	150.2949199	149.421013	132.0585	5.439655
50000	25.14586997	145.877644	25.14587	25.14587	25.14587	25.14587	19.01165505	25.00170606	25.49244978	21.1935736	3.11878823	24.81606809	153.1416205	153.1130645	152.208089	12.45614	5.739229
60000	21.99647157	147.2229697	21.9964716	21.996472	21.996472	21.996472	25.98468748	26.12577258	158.1839838	26.4362057	5.472296513	19.85992376	155.2490742	155.2185047	154.316387	20.43089	7.083378
70000	26.73575794	149.9493178	26.7357579	26.735758	26.735758	26.735758	20.26697944	22.13364191	170.1997357	26.9146634	6.223838388	26.53340464	158.1716495	158.1413242	156.863875	137.1376	7.162068
80000	27.07865991	152.675666	27.0786599	27.07866	27.07866	27.07866	22.40901929	26.92486907	160.7040243	168.459943	7.407920278	26.80134804	161.553956	161.5171047	159.430381	137.9634	15.22171
90000	27.50614827	157.9298465	182.09509	164.61446	117.44186	131.36574	22.64848829	146.3590328	163.1469603	162.875227	8.035760254	151.7287147	161.4379552	161.4039334	160.231563	138.4382	15.3929
100000	28.07940696	154.3526025	188.297017	168.05674	117.9149	131.80963	23.33230281	146.7854079	164.7518314	161.631345	7.645379546	149.65442	160.7490746	160.6933468	159.337509	21.5673	15.51028
110000	157.8357986	160.7201964	197.195736	172.44808	121.66815	141.29811	23.61813426	146.6231347	165.3667467	162.69887	8.900796182	147.5801252	162.5077566	162.4731554	160.929714	134.1342	16.14675
120000	162.5022694	176.9305555	190.223195	183.6007	123.56172	146.72606	24.07015409	148.1830477	174.9399868	170.840487	9.119725149	145.5058305	160.2799369	160.255707	159.11801	21.76293	16.24243
130000	167.3520614	168.1124632	190.819009	173.13515	123.91683	143.43511	24.31740213	149.7429608	182.6239259	174.771096	8.614180415	145.1683971	162.5090001	162.4824688	161.37458	22.03967	16.39342
140000	158.6411194	166.7648662	191.884598	175.25154	123.30868	143.34916	24.29370236	158.7561306	186.3965406	179.666268	151.3560637	152.9985532	165.3040065	165.2685236	162.364083	21.92096	16.49351
150000	158.1779709	164.8330546	194.022494	176.6277	122.80102	143.57904	24.31598961	160.6738399	190.2432836	165.69003	152.3449124	160.8287092	166.2143661	166.1737664	164.082791	22.26468	16.63187
170000	167.7653745	183.5705251	193.251442	184.27509	125.18435	149.62916	25.07246462	162.5915492	171.3193993	170.696543	157.1215711	168.6588653	164.5195876	164.4945315	165.439226	22.58556	17.44969
180000	170.6484435	189.6670212	193.024201	187.96164	125.61952	145.40141	25.39095977	164.5092585	172.3934254	172.529979	159.112192	170.6455746	167.0076352	166.9655297	166.002765	144.0914	17.49822
600000	193.7292096	223.7937604	277.884016	227.31787	151.30751	177.65304	170.9318852	183.5937449	243.727298	228.62723	168.0785972	204.0878006	220.2962414	220.2644558	219.289532	69.90845	134.207

Figure C.7: Continued: Erosion volume after a storm (m^3/m) per return period for different parameter modifications. (ID 6)

Return period	Average	a+2sigma	a+sigma	a-2sigma	a-sigma	Clay+2sigma	Clay+sigma	Clay-2sigma	Clay-sigma	Fsand07	Fsand08	Fsand09	Fsand10	Grass+2sigma	Grass+sigma	Grass-sigma	Crest 5.5m	Crest6m
1000	0.03497	0.03497	0.03497	0.0722513	0.072229	0.03496975	0.03496975	0.0349697	0.0349697	0.03496975	0.03497	0.03497	0.0349697	0.034969749	0.034969749	0.034969749	0.0349697	0.03497
3000	0.19203	0.1917496	0.19175	0.2296086	0.2296	0.19202683	0.19202683	0.1920268	0.1920268	0.19202683	0.192152	0.192423	0.1925608	0.192026832	0.192026832	0.192026832	0.1920268	0.19203
5000	0.6882	0.6870161	0.687016	0.7375961	0.737554	0.68819761	0.68819761	0.6881976	0.6881976	0.68819761	0.68977	0.68856	0.6886367	0.689562224	0.689063585	0.688197608	0.6881976	0.6882
10000	2.69693	2.6585113	2.658511	3.6212481	3.621193	2.69794612	2.6973338	2.6945042	2.6958888	2.6969328	2.700916	2.703648	2.6990775	2.700140268	2.699315737	2.692822602	2.6969328	2.69693
20000	15.5467	8.988757	8.988757	42969.19	15.62724	15.545344	15.5454501	15.543429	15.544706	15.5467389	41545.52	42624.89	42944.448	40861.53497	13.32498788	14.72237502	15.546739	15.5467
30000	49.2758	31.777495	31.78552	56337.823	56359.48	49.2704679	49.2654967	49.272624	49.277499	49.2757961	56414.16	57227.02	57546.48	56345.23409	52.4848277	47.79711955	49.275796	49.2758
40000	51.8228	29.345086	29.37038	58604.814	58656.34	51.7541801	51.7819006	51.752989	51.750587	51.8228432	59167.52	59749.74	59976.023	59185.8021	58484.8277	50.47568921	51.822843	51.8228
50000	73.8452	41.639084	41.67524	63459.436	63548.05	54060.7168	53890.425	73.809752	73.883012	73.8451842	66580.67	67968.08	67533.363	65646.14222	63914.99349	53.81171258	73.845184	73.8452
60000	139.698	89.853489	89.92735	72283.002	72362.33	56469.7919	56131.0772	139.70831	139.70761	139.698144	76663.04	77635.65	78900.888	76547.46631	75970.49112	143.369044	139.69814	139.698
70000	125.532	71.4523	71.53405	74546.451	74610.93	57926.7471	57713.1052	125.58427	125.63936	125.532044	78396.98	79080.69	78670.721	78067.47504	75025.40996	105.5321079	125.53204	125.532
80000	156.544	91.692288	91.78937	74390.181	74416.71	59383.7023	59295.1331	156.58189	156.64995	156.543687	78874.36	79168.48	80282.237	78539.57224	77217.6975	130.6983237	156.54369	156.544
90000	60008.2	138.29763	138.4053	83935.777	84046.85	61704.3353	60281.7306	222.25106	222.24258	60008.1675	89423.59	90181.84	90510.649	87782.49501	86834.69716	210.9580891	60008.167	60008.2
100000	61741.4	125.53177	125.6677	83555.611	84054.34	63947.8385	61879.499	214.19733	214.15498	61741.4015	89480.68	90322.17	90822.449	89233.41767	87922.69253	158.7193867	61741.401	61741.4
110000	81163.2	143.20156	143.3377	84482.821	84688.49	112924.288	95829.2071	289.37914	71357.88	81163.1537	90237.63	91589.42	92010.522	89791.5047	89098.26484	168.1972151	81163.154	81163.2
120000	90796.7	236.24102	236.3879	93175.453	93316.32	122867.718	111496.373	422.10384	81463.442	90796.6864	94523.86	95585.7	95613.81	94384.2993	95252.21771	263.756497	90796.686	90796.7
130000	94101.4	220.13405	220.2795	94579.236	94735.32	122526.504	113584.239	416.86772	81628.339	94101.4405	95985.66	96498.91	96990.141	95651.00521	97582.98659	239.7406685	94101.44	94101.4
140000	90254.1	211.33094	211.4723	94040.045	94160.89	120922.33	110569.433	413.51309	81991.561	90254.0867	94931.88	95890.48	96654.298	94844.41138	94339.31033	231.4863764	90254.087	90254.1
150000	90121.1	230.4263	230.6615	90967.574	91039.11	120100.636	111062.306	445.29756	81483.119	90121.109	91838.25	93123.8	93596.81	91622.71885	91139.65388	252.6126021	90121.109	90121.1
170000	100087	386.71224	387.1117	103172.56	103023.2	130099.452	123461.573	72436.653	91421.751	100087.393	106419.6	107642.8	107902.24	105907.9451	103926.1341	426.4807483	100087.39	100087
180000	98100.9	374.80814	375.1901	104669.15	104718.7	137006.086	124440.875	73641.371	91677.552	98100.8972	108053.4	108245.5	108635.82	107679.4384	106211.088	510.6477111	98100.897	98100.9
600000	154806	1638.7046	102889.3	155411.52	155514.4	197200.163	186495.755	105861.27	118989.29	154805.551	160714.1	161165.5	161586.21	160860.1945	161047.5922	1889.644842	154805.55	154806

Figure C.8: Overtopping volume after a storm (m³/m) per return period for different parameter modifications (ID 6)

Return period	ClayLayer+20cm	ClayLayer-20cm	Sand+2sigma	Sand+sigma	Sand-2sigma	Sand-sigma	Terrace+2sigma	Terrace+sigma	Terrace-2sigma	Terrace-sigma	Open_a+2sigma	Open_a+sigma	Open_a-2sigma	Open_a-sigma	Open_Average	SLR+70cm	SLR+15cm
1000	0.034969749	0.034969749	0.0349697	0.0349697	0.0349697	0.03497	0.034969749	0.034969749	0.034969749	0.03496975	0.034970045	0.035005954	0.072251448	0.072251321	0.072147192	0.145813	0.0092
3000	0.192026832	0.192026832	0.1920268	0.1920268	0.1920268	0.192027	0.192026832	0.192026832	0.192026832	0.19202683	0.191798598	0.228998118	0.229608633	0.229608573	0.22937323	1.596728	0.01437
5000	0.688197608	0.688197608	0.6881976	0.6881976	0.6881976	0.688198	0.688197608	0.688197608	0.68840943	0.68819761	0.68750343	0.734617001	0.737596295	0.737596107	0.736954883	5.329244	0.07392
10000	2.696932802	2.696932802	2.6969328	2.6969328	2.6969328	2.696933	2.694625008	2.695909796	2.695071271	2.69687228	2.662648447	3.603785162	3.621251397	3.621249879	3.620296654	8.914173	0.7408
20000	15.54673888	15.54673888	15.546739	15.546739	15.546739	15.54674	16.20184056	16.68414748	10.8587574	14.8280157	9.017714283	15.54889235	42985.88812	42985.88812	15.62468827	49.56921	2.69702
30000	49.27579609	49.27579609	49.275796	49.275796	49.275796	49.2758	52.25614256	50.7740444	36.17773495	37.7265143	31.815211	49.29300672	56310.67095	56313.20139	56283.88971	57.69406	4.6224
40000	51.82284322	51.82284322	51.822843	51.822843	51.822843	51.82284	53.73025899	51.86145069	47.66728852	51.7283741	29.4152548	51.82284322	58599.5458	58604.81428	58433.04588	35337.01	8.43708
50000	73.84518419	55538.7239	73.845184	73.845184	73.845184	73.84518	76.89331225	75.88182433	73.59587457	71.2910444	41.71071714	73.75257362	63451.02599	63459.43621	63417.19911	361.2624	7.99423
60000	139.6981445	57574.15396	139.69814	139.69814	139.69814	139.6981	142.352591	141.9995035	66390.10129	139.305818	89.95802163	137.6920608	72274.44438	72283.00153	72194.20348	316.9206	22.2972
70000	125.5320445	60158.11468	125.53204	125.53204	125.53204	125.532	129.21378	127.935265	66561.04793	125.545026	71.57461342	125.4591251	74538.41023	74546.45131	73855.05996	53096.42	14.3417
80000	156.5436871	60681.40949	156.54369	156.54369	156.54369	156.5437	160.9725148	156.7332163	70225.19928	71643.4935	91.80465464	156.3962835	74385.25962	74390.18055	73796.68934	55208.41	26.8567
90000	222.2363731	64295.70317	77090.525	69497.224	18703.813	48738.9	236.4356854	64735.57705	78490.44389	84161.4163	138.4036292	59919.62406	83924.92835	83935.77679	83948.88564	56212.53	30.5628
100000	214.1208906	63111.81986	79529.644	70809.683	21618.815	50979.93	221.7151732	66450.85699	78283.48985	83543.4154	125.6662391	61684.77684	83545.80843	83555.61113	83553.54901	614.3049	26.2441
110000	76994.08144	96994.40452	102148.45	95433.778	47430.308	67417.78	250.3231803	68421.12312	78691.93025	83772.8305	143.4081358	63449.92962	84469.9454	84482.82149	84038.36658	51470.93	46.6378
120000	83453.30718	113600.2902	115594.03	100776.41	57246.45	76734.17	387.1151573	69907.31993	82823.55294	89070.4002	236.4675875	65215.0824	93161.05988	93175.45273	93201.45021	741.6702	63.4001
130000	84890.91721	115796.9782	119499.05	104770.01	58706.727	78402.29	374.4513002	71393.51675	84062.13356	90208.528	220.3425315	66980.23518	94564.20338	94579.23598	94676.11865	825.5001	54.8433
140000	85995.78417	112998.7281	116314.83	101035.5	56644.093	74629.61	362.5558845	72879.71356	81674.04499	86858.4582	64639.70878	68745.38796	94030.81881	94040.04514	90736.01912	945.4411	52.7296
150000	84763.238	109564.42	115885.2	101129.47	56665.378	74583.92	394.0760582	74365.91037	81346.82842	89467.203	62173.1094	90071.96262	90958.37392	90967.57409	95117.55547	1127.425	57.4083
170000	92116.74202	121561.2107	119149.87	109966.89	67853.732	85544.56	561.2335523	102247.2172	91898.02402	96207.4395	67334.95925	99130.12697	103199.2677	103172.5567	101498.6059	1172.368	157.062
180000	92774.52576	126057.1349	118715.3	109016.84	65791.317	84235.41	513.4862412	99903.49613	92367.46294	97344.7076	70935.99691	97498.05724	104666.0886	104669.1507	104452.9636	68392.75	142.131
600000	133695.3692	175561.5006	177638.83	168347.67	124013.91	145632.3	149226.0401	143362.2716	142016.8093	147960.029	102869.8018	137805.0114	155394.0488	155395.8102	155601.1026	6398.827	49700.6

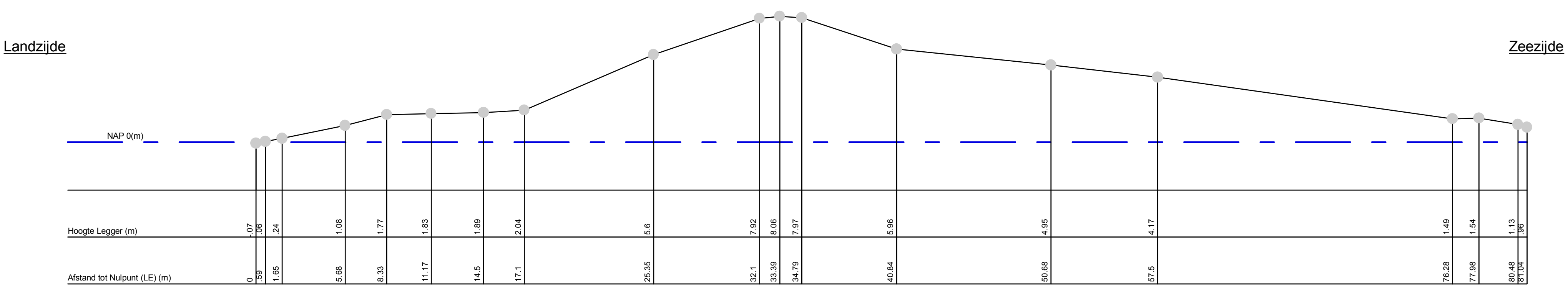
Figure C.9: Continued: Overtopping volume after a storm (m³/m) per return period for different parameter modifications (ID 6)

D

Dike Profiles

The following pages show the dike profiles which were used in this report. They are taken from Waterschap Noorderzijlvest (2010).

Profiel: 76



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

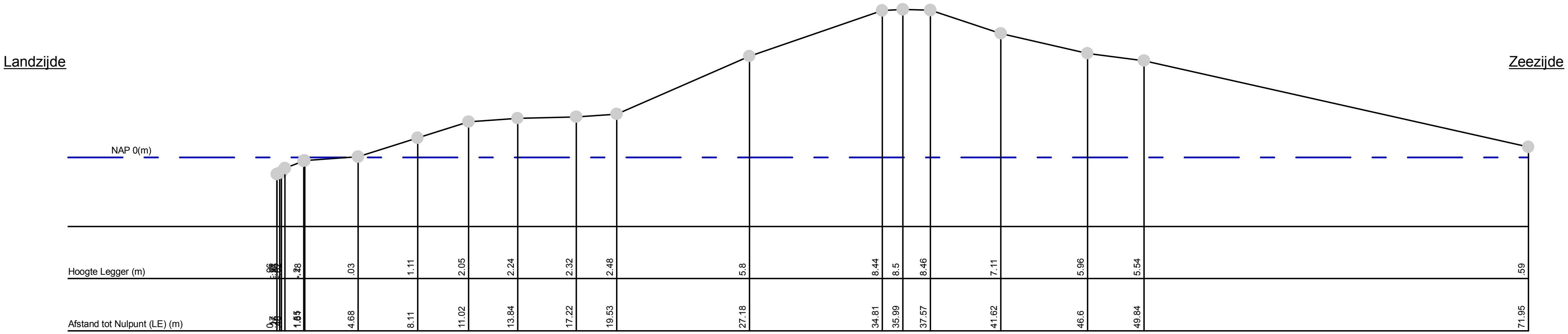
Legger Zeedijk

Profiel: 76

N

Datum : Nov. 24, 2010 | Formaat : A3 | Get. : H. Tjaden | Blad : 218 van 242

Profiel: 77



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

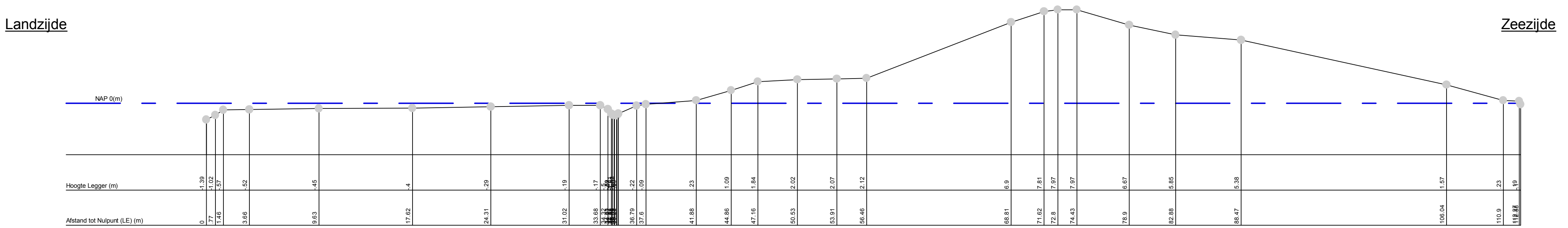
Legger Zeedijk

Profiel: 77

N

Datum : Nov. 24, 2010	Formaat : A3	Get. : H. Tjaden	Blad : 219 van 242

Profiel: 78



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

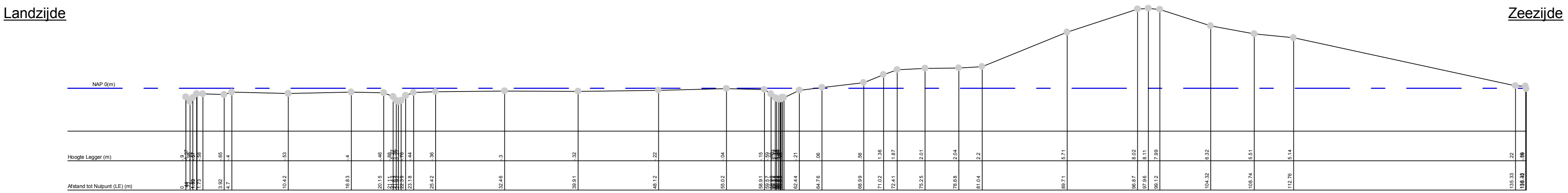
Legger Zeedijk

Profiel: 78

N

Datum : Nov. 24, 2010 | Formaat : A3 | Get. : H. Tjaden | Blad : 220 van 242

Profiel: 79



Maatgevende locaties
Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

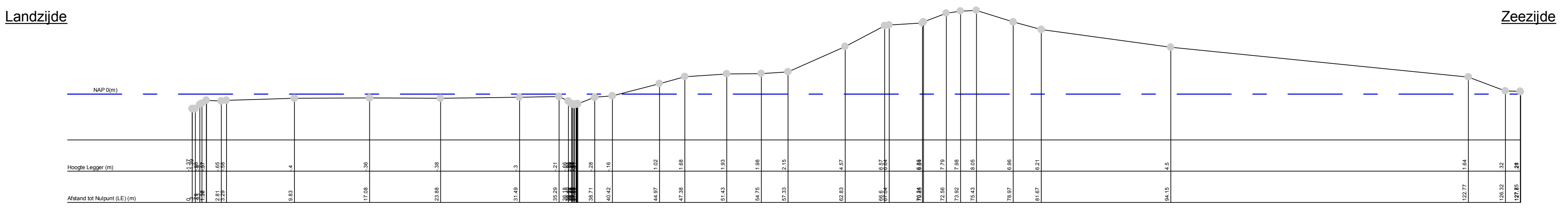
Legger Zeedijk

Profiel: 79

N

Datum : Nov. 24, 2010	Formaat : A3	Get. : H. Tjaden	Blad : 221 van 242

Profiel: 80



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

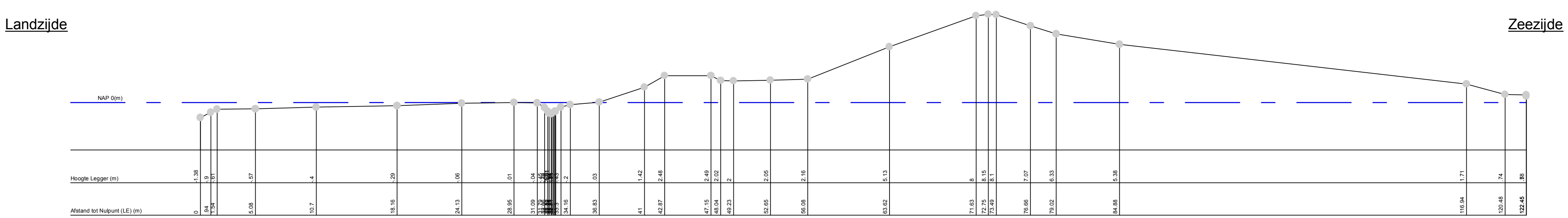
Legger Zeedijk

Profiel: 80

N

Datum : Nov. 24, 2010 | Formaat : A3 | Get. : H. Tjaden | Blad : 222 van 242

Profiel: 81



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

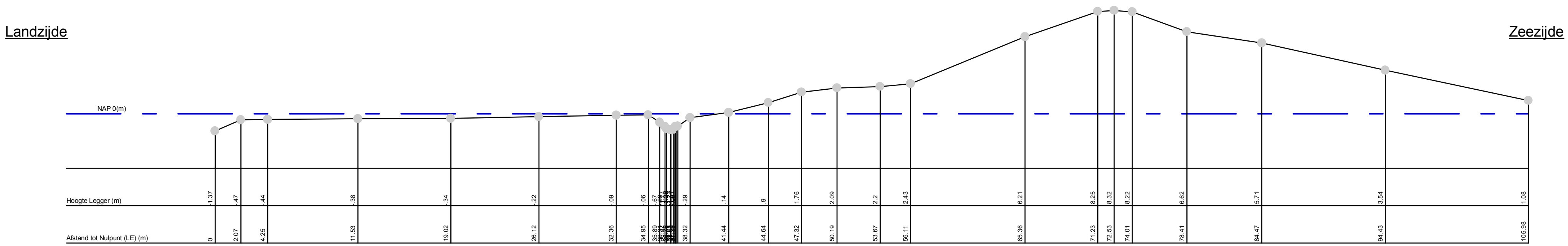
Legger Zeedijk

Profiel: 81

N

Datum : Nov. 24, 2010	Formaat : A3	Get. : H. Tjaden	Blad : 223 van 242

Profiel: 82



Maatgevende locaties
Vakgrenzen

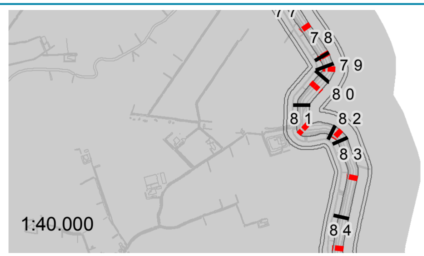
Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl



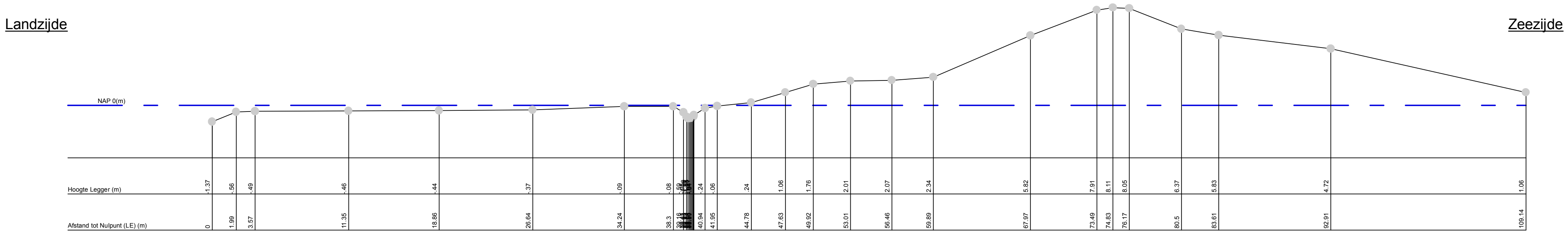
Legger Zeedijk

Profiel: 82

N

Datum : Nov. 24, 2010 | Formaat : A3 | Get. : H. Tjaden | Blad : 224 van 242

Profiel: 83



Maatgevende locaties
Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

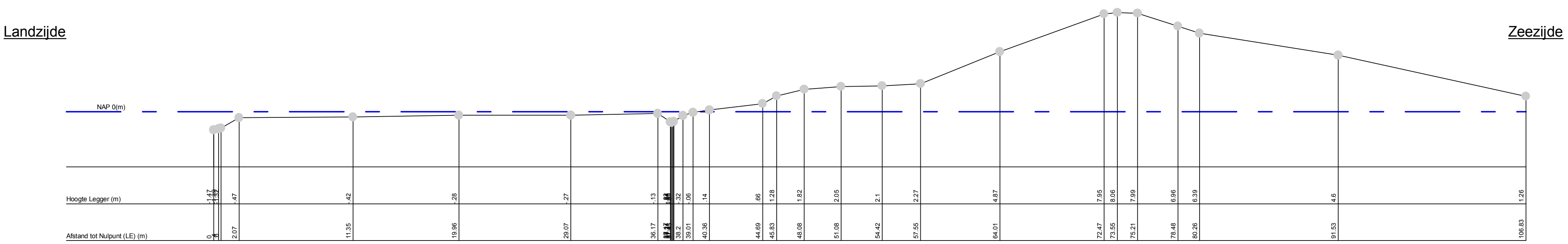
Legger Zeedijk

Profiel: 83

N

Datum : Nov. 24, 2010	Formaat : A3	Get. : H. Tjaden	Blad : 225 van 242

Profiel: 84



Maatgevende locaties

Vakgrenzen

Hoogteschaal = Lengteschaal

Waterschap Noorderzijlvest

Stedumermaar 1

Postbus 18
9700 AA Groningen
Tel : (050) 304 89 11
Fax : (050) 304 82 26

info @noorderzijlvest.nl

Legger Zeedijk

Profiel: 84

N

Datum : Nov. 24, 2010	Formaat : A3	Get. : H. Tjaden	Blad : 226 van 242