Influence of Sandy Foreshores on Overtopping in Non-Tidal Low-Energy Shallow Lake Environments

A case study to optimize maintenance strategies and approximate the probability of failure

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Technische Universiteit Delft

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A case study to optimize maintenance strategies and approximate the probability of failure

by

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Abstract

Sea level rise will increase the risk of flooding in coastal areas. This poses a risk to the coastal protection as well as rivers and lakes close to the coast. Solutions are needed to cope with this threat. The past decade, nature based solutions have gained significant interest. One of these solutions could be sandy foreshores. Due to the use of natural materials, sandy foreshores are a ‘nature-based solution’ opposed to a more traditional approach of dike reinforcement. For sandy foreshores to be a viable alternative to regular dike reinforcements, the order of magnitude of construction and maintenance costs need to be known. For this reason, it is necessary to be able to calculate a failure probability of a dike with sandy foreshore, to predict the required maintenance and to optimize the design based on life-cycle costs.

To improve the calculation method, a step-wise calculation of the failure probability for wave overtopping of a hybrid structure was developed. The calculation included an iterative process. The calculation methods consists out of calculating the failure probability for wave overtopping in Riskeer and a dune erosion calculation in Xbeach. In this research, overtopping was considered as the dominant failure mechanism. For the assessment, the dike was seen as an impermeable hard layer and the foreshore as a beach. Therefore, dune erosion could not erode the main structure and is considered a sub-mechanism of overtopping.

Next, a calculation was performed to predict the longshore transport along the beach in Almere Duin. The occurrence of different wind directions, together with a Delft3D model of the Markermeer, was combined to find the longshore transport. The LST was calculated with transport formula calibrated for coastal areas. The life cycle costs (LCC), including design and maintenance costs, were calculated for different strategies. The calculated alongshore transport together with cost estimates were used to calculate the LCC. Subsequently, the net present value of the maintenance costs was calculated to determine the LCC for each maintenance strategy. At the end the uncertainty and sensitivity of each strategy were analysed.

The time-varying protection level can be optimized by calculating the failure probability due to wave overtopping, with Riskeer and calculating the erosion of the foreshore, with Xbeach. To use this method, Riskeer and Xbeach should have the same design point and if the design point shifts, an extra iteration is necessary. For a 1/10 profile, only a 0.25 m lowering of the foreshore height was found and one extra iterative step was required to carry out the calculation. A maintenance strategy with a 100 year maintenance interval period was found to be optimal in this thesis and a Monte Carlo simulation, which included uncertainties, led to similar results. However, the total LCC were 21% to 34% higher, when uncertainties were included in the calculation. These findings suggest that it is not necessary, to carry out a probabilistic calculation, to find the optimal maintenance strategy. However, the models used, as well as the local boundary conditions, such as the longshore transport and the cost of sand, were found to influence the life cycle costs significantly. For this reason, it is concluded that a location specific analysis is important to optimize the maintenance strategy.

Preface

This thesis has been written as the final part of my master degree in Hydraulic Engineering at the Delft University of Technology. For about 9 months I have been a guest at Antea Group B.V., where I have had a great time and met a lot of wonderful people.

There are a few people I would like to thank for their support during this thesis project. First of all, I would like to thank Jan-Bert for all the time spent together every Monday in Almere. You have always made time for me and were always available when I was stuck. I truly enjoyed learning from you and all the interesting discussions we had. I would like to thank Claus for all the great recommendations during our meetings and making me feel welcome at Antea. I would like to thank Vincent, for helping me when I was lost and always knowing a solution when I got stuck. I would like to thank Robert for all the kind words to motivate me and I would like to thank Matthijs for guiding the process during the meetings. I would like to thank Anne, for helping me get going, giving me advice and allowing me to use your data and scripts for my thesis. I would like to thank all other colleagues at Antea for the great time I have had and all the advice they have given me over the 9 months I have spent with them. Lastly, I would also like to thank my friends and family, for the support they gave me and helping me make the finishing touches.

Oscar le Grand
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Contents

[hideallsubsections]

Nomenclature

Acronyms

DaF	The Dams and Foreshore module
GEKB	Gras Erosie Kruin Binnentalud
HRD	Houtribdijk
LCC	Life Cycle Costs
LST	Longshore Transport
NAP	Nieuw Amsterdams Peil
PV	Present Value
WBI	Wettelijk Beoordelings Intrumentarium

Greek Symbols

ω	The failure probability share

Roman Symbols

N	The length effect

Subscripts

ρ_s	sediment density
H_S	Significant wave height
h_t	Water depth
$H_{2\%}$	Average wave height of highest 2% of waves
$H_{m0,o}$	Offshore spectral wave height
$H_{s,br}$	significant wave height at the breakerline
K_{swell}	swell correction factor
$L_{m-1,0}$	Deep Water Wave Length
P_f	Failure probability
$P_{crosssection:OT}$	The failure probability at cross section level for overtopping
$Q_{t,mass}$	Spectral Offshore Wave Period
$T_{m-1,0}$	Spectral Offshore Wave Period
V_{wave}	wave-induced longshore current velocity

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Introduction

1.1. Context

Sea level rise will increase the risk of flooding in coastal areas [1]. This poses a risk to the coastal protection as well as rivers and lakes close to the coast. Flooding causes great risk of economic damage and lost lives and must be prevented [2], for this reason new solutions are needed to cope with this threat. The past decade, nature based solutions[3] have gained significant interest. Previous examples of this so called “building with nature ” include the *Zandmotor*, *Markerwadden*, *Hondsbossche duinen* and *Houtribdijk zandige versterking* [4].

There may be alternatives to traditional dike reinforcements, which could be more pleasing to the eye, more sustainable and which could improve leisure facilities and natural habitat of local fauna. One of these solutions could be sandy foreshores [5]. Due to the use of natural materials, sandy foreshores are a ‘nature-based solution’, opposed to the more traditional approach of dike reinforcement. A sandy foreshore consists of a large quantity of sand that is deposited in front of a dike. This body of sand attenuates the waves and therefore eliminates or reduces the impact of the waves on the dike. If a sandy foreshore could be cheaper or as cheap to build and maintain as a traditional dike reinforcement, it could be an excellent alternative to traditional dike upgrades.

A sandy reinforcement has already proven a solution for coastal reinforcements. For example the Hondsbossche duinen, prins Hendrikzanddijk and the zandmotor. As previously mentioned, hybrid dykes may also be situated in lake environments. Lake environments where sandy foreshore were constructed are the two Dutch lakes the Markermeer and the IJsselmeer. In general however, the concept of sandy foreshores in lake environments is less well-studied than it’s coastal counterpart.

Lake environments differ from the coastal environment in various ways. Lakes are often more shallow than seas, typically smaller, have a shorter fetch and there are no tidal waves in lakes. Therefore, the hydrodynamic forces are generally less energetic in lake environments than in coastal areas. This project will look in further detail into sandy foreshores in shallow, low-energy, non-tidal lake environments. A thorough explanation of lake characteristics will follow in the literature study.

Hybrid is defined as ’*having two different types of components performing essentially the same function*’. In this thesis a hybrid levee is defined as a combination of a ‘soft’ sandy foreshore and a ‘hard’ dike core. Where the foreshore is a dynamic object of which the shape could be rearranged by a storm while the dike is a robust object of which the shape should remain unaltered. Both are part of the same flood defence but they each require a different design philosophy.

Three different foreshore categories are mentioned in the literature [6]. Figure 1.1 illustrates a schematic drawing of the three different configurations. The first category is a sandy foreshore as a fully independent flood defence, shown as ‘configuration A’ in figure 1.1. In this scenario the sandy foreshore functions as a dune and bares the full hydraulic load. ‘configuration B’ is a sandy foreshore which can be fully load bearing, but for ‘B’ the foreshore is integrated with the levee. The levee and the foreshore work together as one structure. In the third class ‘configuration C’ the foreshore is used to reduce the hydraulic load. Here the foreshore reduces the wave height of the incoming waves and thus makes the wave attack on the levee smaller. A thorough explanation of the three foreshore categories will follow in the literature study.

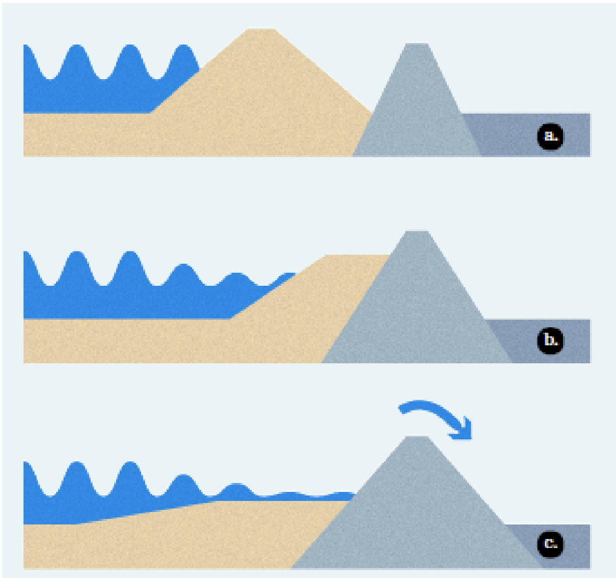


Figure 1.1: Sandy foreshore categories, a: sandy foreshore as a fully independent flood defence, b: sandy foreshore integrated with the levee, c: sandy foreshore as load reduction.

This research focused on foreshore category C, where the function of the foreshore was to dissipate the incoming wave energy. In this scenario, wave overtopping is the main failure mechanism. Contrary to normal dikes, morphological changes can occur at a sandy foreshore during an extreme event. Because of this, it is important to be able to predict the behaviour of a sandy foreshore during a storm event.

The flood defences of most lake environments in the Netherlands are primary flood defences. The Dutch national ’Waterwet’ law states the safety standard which primary flood defences should meet. In the ’Waterwet’ the safety standard is denoted in annual probability of flooding. To know if a sandy foreshore complies with the safety standard, a method is necessary to examine the probability of failure of the hybrid structure.

A number of methods to determine the probability of failure of a hybrid levee have been developed[7][8][9], which will further be elaborated upon in paragraph 2.3. To improve the calculation, a new method to determine the probability of failure is carried out in this thesis. With this method, the safety profile, which is the minimum volume of sand required to meet the safety standard, can be optimized. When the volume of the safety profile is known, the initial construction costs of a sandy foreshore can be calculated and could thereafter be compared to a normal dike reinforcement.

To be able to fully consider a hybrid levee as a possible dike reinforcement, the life cycle costs (LCC) should be known. These are the total construction and maintenance costs of a structure during its lifespan. During its lifetime, a foreshore could also slowly erode during daily conditions and to counter this loss of sand over the years, a maintenance strategy is required. When the maintenance strategy is known, the life cycle costs can be calculated.

1.2. Case Study

To study the application of sandy foreshores in lakes in this thesis, the case study of Almere Duin Noord was used. A case study is a real-life problem which involves qualitative methods to understand, compare and describe different aspects of a research problem. Almere Duin is an excellent case study because of the ambition of the municipality to construct a sandy foreshore in front of the current dike.

This allowed for this research to test and challenge the current methods to determine a failure probability and optimize the maintenance strategy. The results of which could be compared to other cases. Furthermore it offered the opportunity to examine if a sandy foreshore is a viable alternative to a dike reinforcement. Almere Duin was also a good case study because there is no straight forward solution yet, hence new solutions should be considered. Figure 1.2 presents the location of the Almere Duin Noord beach.

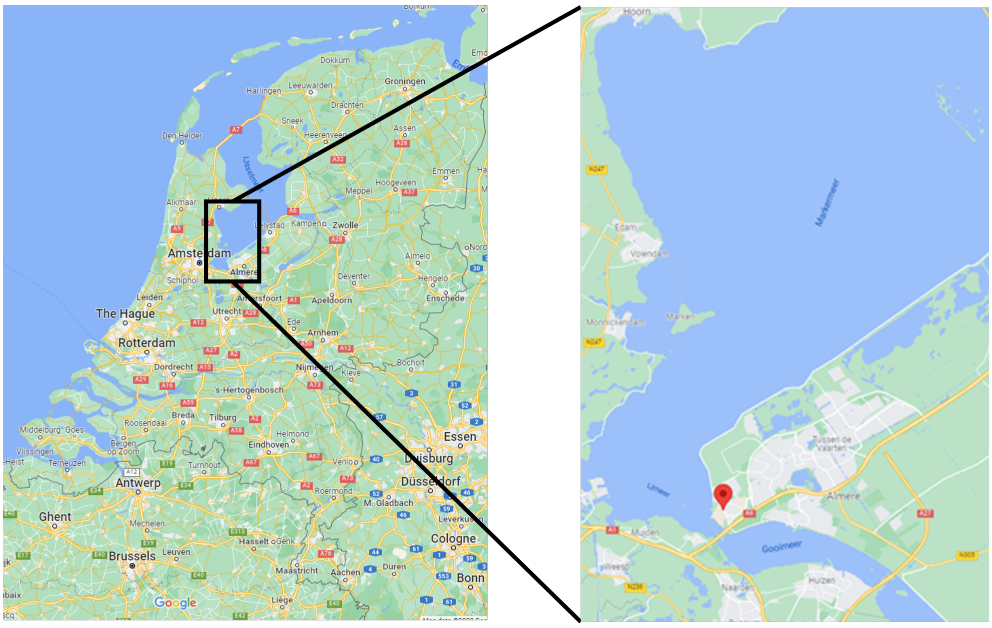


Figure 1.2: Location of Almere Duin

1.3. Problem Description

The order of magnitude of construction costs, and maintenance costs needs to be known, for sandy foreshores to be a viable alternative to regular dike reinforcements. It is therefore necessary to research the influence of different maintenance strategies on the life cycle costs.

It is also not known if a probabilistic calculation, which includes uncertainty, would shift the optimal maintenance strategy.

To calculate different maintenance strategies, the longshore transport needs to be known. At this moment it is unclear how applicable the current longshore formulae are for the prediction of erosion at the Markermeer and IJsselmeer. This is the same for a more general view: A low-energy non-tidal shallow lake environment. In the literature study the applicability of the transport formulas will be quantified to be used in the following parts of this research. For the case study Almere Duin, only wind data is available to predict erosion. There is currently no method that can inherently provide predictions of the longshore transport based on wind data.

Furthermore, there could be a more optimal calculation of the failure probability. Previously proposed methods both have long calculation times and do not implement the already existing calculation software for wave overtopping. Improving the calculation of such a levee could reduce the initial costs or add a maintenance buffer to existing dikes.

A more concise understanding of the influence of different layouts of sandy foreshore with or without hard structures is necessary. Also a deeper understanding of influence of different foreshore parameters on wave overtopping and dune erosion is required to optimally calculate the failure probability.

1.4. Research Question

The main research question to be answered in this thesis is:

- How can the time varying effect and maintenance strategy of flood defences with sandy foreshores in low-energy non-tidal shallow lake environments be optimized?

The main research question consist of two questions. One question focuses on a method to examine the failure probability and the other question focuses on finding a method to calculate the life cycle costs. In order to answer the main research question, four sub-questions need to be answered. The first two sub-questions are directed at the failure probability and the third and fourth sub-question are directed at the life cycle costs. To answer these sub-questions, this research is split up into four different tracks, each of which is devoted to one sub question.

The sub-questions are:

- What is the influence of different foreshore heights and slopes on wave overtopping and how do different foreshores and parameters influence dune erosion?
- How to determine the failure probability of a hybrid levee due to wave overtopping?
- What order of magnitude of transport can be expected at Almere Duin?
- How can a sandy foreshore be designed to minimize the life cycle cost

1.5. Methods

In this section the methods used in each track will be explained. In section 1.6: the outline, a flowchart is shown to further help clarify the methodology.

In the first track, an analysis was performed for both overtopping in Riskeer and dune erosion of the foreshore for the case study Almere Duin. In the first analysis of the wave overtopping, different foreshore heights and foreshore slopes were analysed to examine their influence on reducing the failure probability. In the sensitivity analysis of dune erosion the influence of different input parameters was examined and the influence of different layouts of the hard structures was examined.

In the second track, the failure probability of different profiles was calculated. This was done in four steps. The first step was to calculate the failure probability of a hybrid levee in Riskeer for overtopping. The next step was to model the dune erosion of the profile in Xbeach for the design point calculated in the first step. Then a schematization of the eroded profile was made and calculated again for overtopping in Riskeer.

In the third track, the alongshore transport was calculated. The first step was to calculate the occurrence of different wind directions. The next step was to calculate, for each wind direction, the flow velocity along the beach, using a Delft3D model of the Markermeer. Next, this data was combined to find the occurrence of different flow velocities and wave heights along the beach. Finally, after the longshore transport was calculated, with the van CROSMOR2013 transport formula.

In the final track, the life cycle costs were calculated for four different maintenance strategies, each representing a maintenance interval period of respectively 1, 4, 10, and 25 years. The yearly erosion per cross section was calculated. First, the size of the maintenance buffer for each profile was calculated. Then, the interval time between maintenance and costs per maintenance were calculated. Next the present value of the maintenance was calculated and added to the initial costs of construction to find the total costs of each maintenance strategy. Finally, a probabilistic calculation was performed, to examine if the optimal maintenance strategy would shift, if the uncertainty is included in the calculation.

1.6. Outline

The content of the chapters and methodology is explained in the flowchart below. The blue parts are the main parts in the previous chapters. Chapter 3 contains track one and subsequent chapters include the subsequent tracks as shown in the figure.

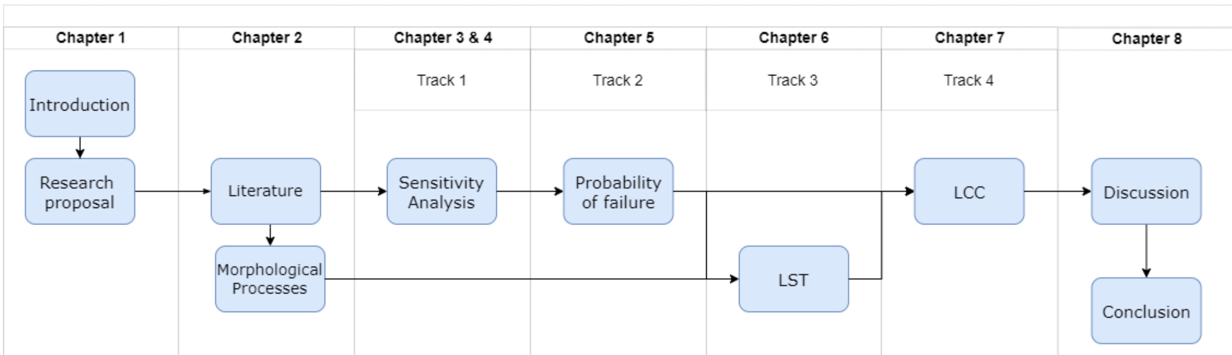


Figure 1.3: Outline of this thesis

1.7. Demarcations

Due to time constraints or complexity it is often not possible to include everything in a thesis. Therefore, some demarcations were made which are specified in this section.

Eolic Transport

Although Eolic transport is of importance in the Markermeer and IJsselmeer, it is a research topic on its own. Therefore it will not be further discussed in this thesis.

Consolidation

Although consolidation is of importance for the location of the case study, it is left out of this thesis.

Failure Mechanisms

Wave overtopping is a failure mechanisms where the probability of failure can be significantly influenced by the foreshore, therefore in this thesis only the failure mechanism wave overtopping is considered. Other failure mechanisms such as piping, dune erosion and instability of the slope are left out of the scope of this thesis.

1.8. Background

Almere Duin

Around the year 2000 the state ordered Almere to build 60.000 houses between 2010 and 2030. The city Almere decided to use the surrounding areas to built new neighbourhoods. One of these new neighbourhoods is Almere Duin, which is located on the west side of Almere on the IJmeer shore. A tender was started and won by Amvest, who designed a dune like landscape to spread the beach atmosphere through the entire neighbourhood. Almost 2 million cubic meters of sand was needed to realise the new neighbourhood. The design includes the construction of a dune over the existing levee along the IJmeer (dijkring 8) and the construction of a sandy foreshore (beach) in front of the levee. It is foreseen that future reinforcement of the levee will be expensive because the levee needs to be dug out from under the dune to do so. Therefore, the decision was made to guarantee the strength of the levee until 2125. This brings the interesting new question: in what capacity can the beach be seen as a reinforcement of the levee? Ideally the sandy foreshore strengthens the existing levee enough, so no additional reinforcements need to be done to the levee. As sandy reinforcement for levees are still new and calculation methods are not yet completely developed, new calculation methods are needed because unnecessary levee heightening and reinforcing are unwanted by everyone.

Bathymetry

In this subsection the bathymetry of the IJmeer is presented. As can be observed in figure 1.4 the depth of the IJmeer in front of dike section 8 is between 1.5 and 3 meters.

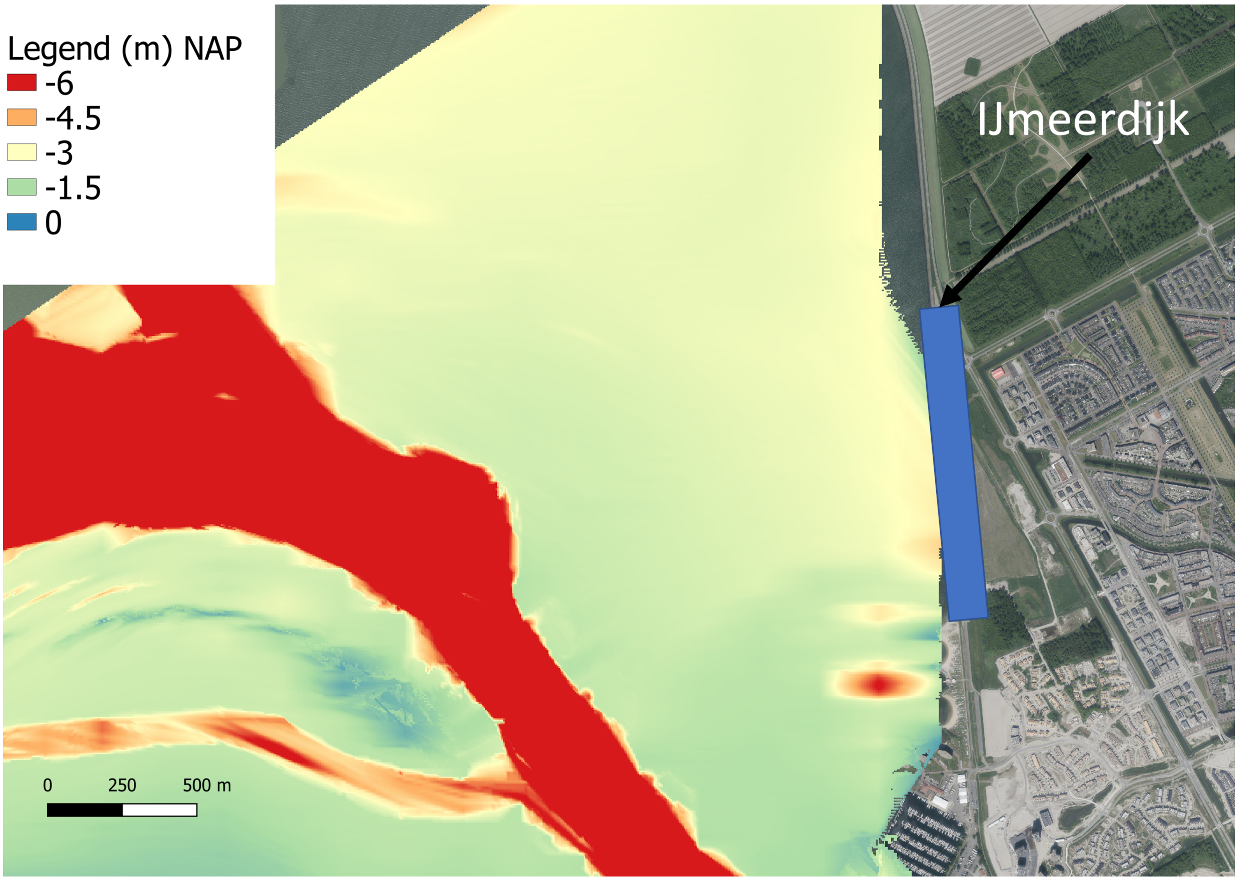


Figure 1.4: Bathymetry of the IJmeer

IJmeerdijk

A case study was carried out for the IJmeerdijk which is part of dike ring 8, shown in figure 1.6 . To be more specific IJmeerdijk is dike section 8 – 2, from kilometer 3.1 to 4.4 km. Profile at dike kilometer 3.8 is shown in figure 1.5 below. The top of the dike is at level + 2.70 NAP. The dike has 1:3 slope until 1.6 meters height. Followed by a biking path. Then there is block revetment from +1.5 to 0 NAP in a 1:3 slope. Thereafter there is quarry stone for the stone revetment from 0 to -2.05 NAP. The mean water level in the lake ranges from -0.10 to -0.30 NAP. The dike section is straight and the normal of the dike has an orientation of 265 degrees.

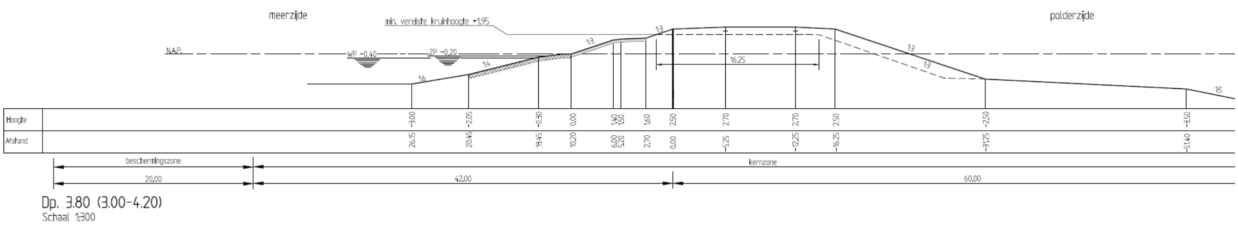


Figure 1.5: Cross section of the IJmeerdijk

Requirements

The IJmeerdijk is a primary flood defence with a safety standard of 1/10000 per year. The Municipality has another requirement that the maximum tolerable wave overtopping allowed is 0.1 l/s/m. So people can safely walk the dunes. Which makes an interesting case because

Figure 1.6: Location of the Dijkring 8

normally dunes do not have wave overtopping limits. The waterboard wants their dike not to be changed by the construction of the beach and wants to make sure there is no way they will make any costs.

1.9. Definitions

WBI

The WBI 2017 (*wettelijk instrumentarium voor de beoordeling*) is the Dutch toolbox for the assessment of primary flood defences. It contains both the rules for determining the hydraulic loads and strengths of primary flood defences as the procedural rules for the assessment of the safety of primary flood defences.

Schematization manuals have been written to prescribe the use of data for making schematizations for different assessments and how to use the data as input for the software. The manuals are written to ensure the proper use of data and software.

1.9.1. Alert Value and Lower Threshold

Two values are always specified for levee segments with a flood or failure probability, each of which has its own function:

Firstly the alert value. If the periodic statutory assessment finds that the probability of flooding for a levee segment exceeds this value, the Minister of Infrastructure and the Environment must be informed. Once the alert value has been reached, one of the conditions for subsidised measures has been met.

Secondly the lower threshold. This is the minimum probability of flooding or failure which the flood defence structure must be designed to prevent. The lower threshold is the maximum permissible value for the probability of flooding or failure. Compliance with this value guarantees the basic level of protection.

1.9.2. Design Point

The design point is the point in the failure space with the greatest joint probability density of both strength and load (i.e. the combination of load and strength variables that are most likely to lead to failure). In case of a failure it is likely that the values of strength and load are close to the values of the design point.

In case of this thesis it is the combination of water level, wave height, wave period and wave direction, which exceed the critical overtopping discharge, with the the highest chance of occurrence. The probability of occurrence of these design parameters is referred to as the failure probability.

Riskeer uses a cross section and outputs a design point for overtopping. In Xbeach the design point is used to output a bed level. If both outputs match it was appropriate to use this method because the design point led to the bed erosion profile, which led to the design point.

If both outputs do not match it was not possible to use this method because the failure probability was calculated for a bed profile which did not occur during the extreme event.

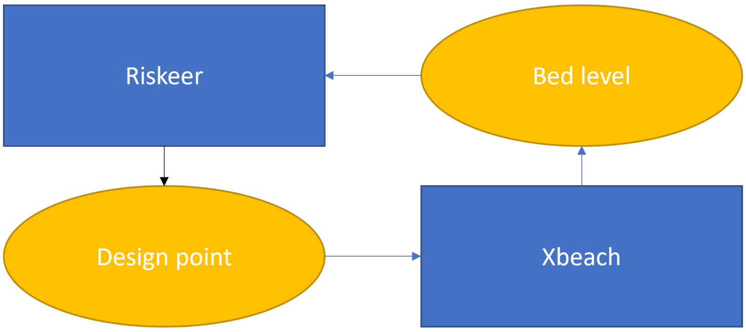


Figure 1.7: Relation between Riskeer, Xbeach, Design point and bed level

NAP
The NAP (Nieuw Amsterdams Peil) is the Dutch standard reference level for bottom height measurements. In the Netherlands all measured heights are compared to the NAP reference level.

1.10. Software

Riskeer
Riskeer is the software used for most WBI assessments. Riskeer can be used to calculate the hydraulic loads for flood defences. A schematization of a flood defence can be put in Riskeer whereupon the failure mechanism model of Riskeer will calculate the failure probability. Riskeer uses a probabilistic core to calculate the probability of flooding of a dike section. Riskeer assesses the strength of a flood defence by calculating the strength for each failure mechanism for different hydraulic loads and weighing the results by the probability of occurrence. Subsequently the results of all failure mechanisms are combined to a detailed assessment of the whole dike section.

Xbeach
In this thesis Xbeach is used to calculate the morphological changes during an extreme event. Xbeach calculates the transport of sand between cells and when the slope between two cells gets too steep, the sand is exchanged between the two cells. Xbeach is used to assess whether different foreshores are sensitive to dune erosion. More information on Xbeach can be found in the theory section 2.3.

Probabilistic Toolkit

The Probabilistic Toolkit analyzes the effects of uncertainty to any model. These models range from python scripts to geotechnical en hydrodynamical Deltares applications and non-Deltares applications. The Probabilistic Toolkit offers a number of analysis types of which the sensitivity and output uncertainty were used [10].

In sensitivity analysis the effects of changes to input variables are investigated. This will give insight in which input parameters are important and help the user decide which input parameters must be measured more precisely.

Output uncertainty is an extension of sensitivity analysis. Now all input parameters are varied according to their uncertainty definition. This leads to uncertainty of the output parameters. This is useful when the user is interested in possible values in the future of a physical property.

2

Theoretical background

This section provides background in the fields this research touches upon. Information about sandy foreshores as well as morphological processes is given.

2.1. Foreshore

Foreshore Definition

According to the Eurotop manual on wave overtopping [11] the foreshore is: "the part of the seabed bathymetry seaward of the toe of a coastal structure, breakwater or sea wall that is characterized by depth-induced wave processes such as wave breaking in front of the breakwater, coastal structure or sea wall. Foreshores are able to dissipate wind-generated waves so that little wave energy remains at their landward limit, the toe of the structure. " According to the Eurotop manual[11], the maximum slope of the foreshore is $1:10$. They argue that a steeper slope could better be considered part of the structure. In the manual a foreshore is defined as "having a minimum length of one wavelength $L_{m-1,0}$".

Shallow Foreshore

According to the Eurotop manual [11] the foreshore can be deep, shallow, very shallow and even extremely shallow. They state that shoaling and depth limiting effects will need to be considered if the water is shallow or very shallow. To make sure the wave height at the toe or end of the foreshore is also considered. A more detailed explanation of the foreshore categories will be given in section 2.3

Foreshore Categories

As previously mentioned in the introduction, there are three main categories for sandy foreshores, shown in figure 2.1. The first category is a sandy foreshore as a fully independent flood defence, shown as 'configuration A' in the figure below. It acts as a dune and takes the load all by itself. The main failure mechanism for such a structure is dune erosion. According the Dutch guidelines standards (WBI2017), to calculate the failure probability the program MorphAN should be used [12].

'configuration B' is a sandy foreshore which can be fully load bearing, but for 'B' the foreshore is integrated with the levee. The levee and the foreshore work together as one structure. The

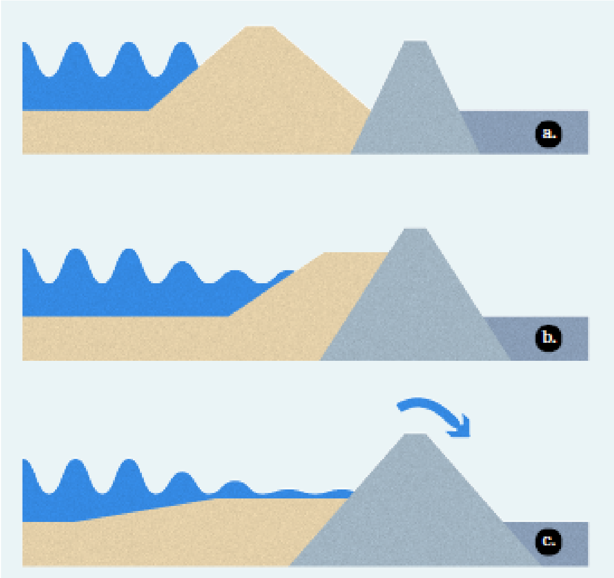


Figure 2.1: Sandy foreshore categories, a: sandy foreshore as a fully independent flood defence, b: sandy foreshore integrated with the levee, c: sandy foreshore as load reduction.

main failure mechanism for this category is also dune erosion. To assess the failure probability of this category the dune should not erode further than the border. To calculate the failure probability for category B this should also be modelled with MorphAN. This underestimates the failure probability but no better techniques are yet to be found in literature.

In the third class 'configuration C' the foreshore is used to reduce the hydraulic load on a flood defence. Here the foreshore reduces the wave height of the incoming waves and thus reducing the hydraulic load on the dike. For this category the main failure mechanism is wave overtopping. Eventhough the sandy foreshore could still erode the foreshore, the hybrid levees is considered to fail when the wave overtopping is too much. The sandy foreshore reduces the overtopping and is thus part of the overtopping failure mechanism. Calculating the failure probability of a hybrid levee has been done before but has not yet been attempted in lake environments. Also there are no guidelines yet to calculate the failure probability of this category.

For wave dissipating foreshores, it is critically that the setup level is above the height of the foreshore. As a result of this waves travel over the foreshore and reach the dike, which causes the foreshore to reduce the wave height. If the waves do not reach the dike the foreshore loses its wave reducing function and becomes a load bearer. In the latter case categories A or B should be considered. This research will further focus on sandy foreshores which reduce the overtopping risk such as category C.

2.2. Failure Mechanisms

To assess the safety of a dike, the assessment is divided into different mechanisms which could lead to failure of the dike. These causes are called 'failure mechanisms'. Different failure mechanisms can impact each other, hence interaction between these mechanisms should be taken into account. There are multiple failure mechanisms of importance for hybrid levees. This thesis focuses on the failure mechanisms wave overtopping and dune erosion.

Overtopping

Wave overtopping occurs when waves reach a structure. During wave overtopping two coastal processes take place: wave transmission and volumes of water travelling over the structure. In the overtopping manual two different types of wave overtopping are mentioned. “Wave overtopping which runs up the face of the seawall and over the crest in (relatively) coherent water mass is often termed ‘green water’. In contrast, ‘white water’ or spray overtopping tends to occur when waves break seaward of the defence structure or break onto its seaward face, producing non-continuous overtopping, and/or significant volumes of spray.” [11]

A hydraulic load can be produced by waves overtopping the dike. Every wave causes a short wave volume over the crest of the dike for a short period which causes a flow velocity and a waterlayer on the slope.

The wave overtopping volumes for each wave are statistically distributed which is dependent on the volume of wave overtopping per wave and the corresponding wave conditions. A wave overtopping discharge 10 l/s/m combined with a high water level and smaller waves corresponds with a very different distribution of the overtopping discharge than overtopping discharges of 10 l/s/m combined with a low water level and larger waves. In the latter case, the amount of waves overtopping the dike is smaller, but the discharge of each wave is much larger. It turns out erosion is mostly driven by larger overtopping discharges. The hydraulic load is dependent on the overtopping discharge as well the as the wave height. [13]

Dune Erosion

Dune erosion is a comprehensive studied cross-shore transport phenomena, which is well understood. Dune erosion is the process where waves hit a dune, causing the dune to suffer large losses. Dune erosion is mostly studied in coastal regions. But the process is similar for a lake shore.

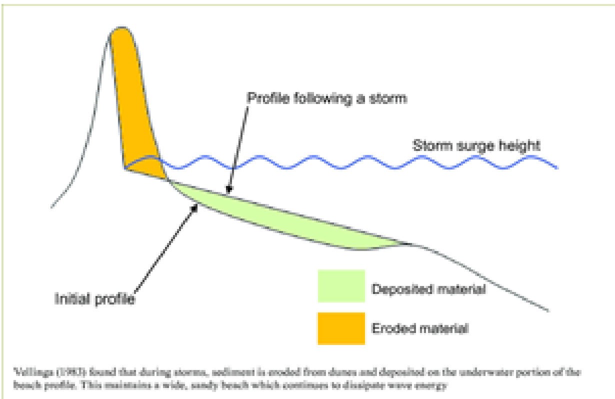


Figure 2.2: Dune erosion adapted from Veilinga (1983)

Normally the dunes are free from wave attack. However during a storm multiple factors can cause the water level to rise and bring waves to the dune. As a consequence of the wave breaking, a high turbulence level occurs at the shore, bringing a lot of sand into suspension. There is a onshore directed mass flux during the storm which is compensated with a strong undertow current. The suspended sand is then transported offshore by the undertow current. The undertow decreases in offshore direction due to the increasing depth and hence loses its potential to transport the sediment. This causes sediments to settle offshore and leads to an increased bed level offshore and decreased bed level near the shore face, resulting in a new profile, as shown in 2.2. The volume of the deposited material is equal the volume of the eroded material. The newly formed profile is more efficient at dissipating the incoming wave energy, reducing the erosion rate. This process continues until a certain moment, where the waves become too little and are no longer able erode the dune, then a new equilibrium profile is formed. The newly formed profile is in better equilibrium with storm conditions. The equilibrium profile is independent of the initial profile. Therefore if the storm lasts long enough the initial profile does not have to be considered.

In coastal regions the storm equilibrium profile will be brought back to the daily equilibrium profile in the time period following the storm. However in lakes the daily wave conditions do not effect the beach state and thus the storm equilibrium profile will remain intact.

2.3. Sandy Foreshores

Foreshore Processes (wave transformation)

A foreshore can play an important role in multiple types of wave transformation. The reduced water depth of a foreshore can influence shoaling, bottom friction, wave breaking, wave runup, refraction and diffraction.

When the water depth becomes less than half the wave length the waves start to deform, this is known as shoaling. The front of the wave starts to be slowed down by the bottom friction causing the waves to steepen and shorten.

Bottom friction occurs when the water depth is less than half the wave length. Then the wave motion causes a horizontal flow in the thin wave boundary layer near the bed. This causes friction between the water flow and the bottom which will lead to dissipation of the incoming waves.

The water level at the wave through is smaller than the water level at the wave crest. In shallow water the wave propagation speed is a function of the water depth. Hence the through slows down more than the crest, causing the wave steepness to increase. Wave breaking occurs when the wave becomes too steep.

When the water depth is not uniform under the wave crest the shoaling effect will cause different wave propagation speeds along the crest. This is called refraction and will cause incoming waves to change their direction to become more parallel to the shoreline.

Foreshore Depth Categories

In the literature, the foreshore is categorized into different classes to how much wave breaking can be expected. These classes are: Offshore, shallow, very shallow and extremely shallow as mentioned before. Offshore can be defined as $\frac{h_t}{H_{m0,o}} > 4$. At this water depth no depth-induced wave breaking occurs. A shallow foreshore is defined as $1 < \frac{h_t}{H_{m0,o}} < 4$. At this depth the foreshore starts to influence the wave breaking. The wave spectrum is still predominantly similar to the offshore spectrum. A very shallow foreshore is defined as $0.3 < \frac{h_t}{H_{m0,o}} < 1$. This is the water depth where the wave height is reduced to 50% to 60% of the offshore wave height by depth induced wave breaking. The majority of offshore spectrum is dissipated at this depth, and a large amount of low frequency has emerged. An extremely shallow foreshore can be defined as $\frac{h_t}{H_{m0,o}} < 0.3$. At this depth most of the high frequency part of the wave spectrum has been dissipated, also the low frequency energy is dominant.[14]

Foreshore Influence on Wave Conditions

The influence of the foreshore on the wave conditions is well-known. Water depth is the most important influential dampening factor on wave height on foreshores. This dampening effect can be attributed to the increase in wave breaking. [15] The wave breaking generates bores in the surf zone, the horizontal momentum of the bores are slowed down by the foreshore.[16] The ability of the foreshore to reduce wave run-up is dependent on the width of the foreshore and the width of the breaker zone. This effect increases with the width of the foreshore up until the width of the foreshore is larger than the breaker width. Afterwards the additional reduction of wave run-up will be minimal. [17]

With a shallow foreshore, waves will break at the foreshore, especially the higher waves. Causing the $\frac{H_{2\%}}{H_S}$/ to be lower than deep foreshores and thus no longer satisfy the Rayleigh distribution. [18] A shallow foreshore considerably changes the wave height distributions and wave energy spectra between deep water and the toe of the structure. It is advised to make use of the wave period parameter $T_{m-1,0}$. [19] In multiple other occasions the foreshore has been found to have a load reducing effect and decrease the incoming wave height, both in experimental flume studies and in field measurement studies. [20] [21] [22]

Markermeer Foreshore

In the Markermeer, the application of foreshores, wave breakers and islands have been studied as possible alternatives to dike heightening [23]. To assess these alternatives, SWAN 1D and 2D wave transformation and probabilistic calculations with HYDRA were performed. From these calculations it was concluded that wave dampening constructions can be a good alternative to dike heightening. Depending on the location, distance to the dike and height of the dampening construction the critical load can be reduced 0.5 to 2 meters.

It was also concluded that adjacent beaches and berms can be very effective dependent on the height of the berm and beach. The effectiveness of a beach appears to be very large in comparison to a more gently sloped and lower foreshore. This is likely due to the difference in height. An adjacent berm and a beach in front of the primary dike have a large effect on wave dampening. The effect of a berm is around 0.5 meters and the effect of a beach is in the order of 0.5 to 1.5 meters.

Conclusion

In summary, Foreshores can have a big influence on overtopping, especially shallow foreshores or extremely shallow foreshore. Foreshores are more influential from $1 < \frac{h_t}{H_{m0,o}} < 4$, but especially form $\frac{h_t}{H_{m0,o}} < 1$. For 1/125000 storm wave height in Almere Duin is 1.70 m. Depth is round NAP -3 m and setup is 1,97 m thus $h_t = 4.97$ and thus $\frac{h_t}{H_{m0,o}} \approx 3$. Bottom is shallow everywhere. For foreshoreS to be very shallow $\frac{h_t}{H_{m0,o}} < 1$ -> $h_t < H_{m0}$ -> $ht = +0.20m$ NAP. Daily water level is 0,0 to + 0,2 NAP m. Thus the foreshores is above daily water level.

According to Verheij [23] foreshore for Markermeer should be even higher: the height of the foreshores for Markermeer should be 0.5 to 1.5 meters. Foreshores should have a width of at least 1 wave length to effectively decrease the wave height, which translates to a wavelength of 18 meter for waves on the IJmeer. If foreshores are wider than one wave length, the additional width only slightly decreases the wave height.

Dams and Foreshore Module

The Dams and Foreshore (DaF) module is an extra toolkit in Riskeer to calculate some of the physical processes which occur at a foreshore. The foreshore model within the DaF module solves the energy balance equations by taking the following physical processes into account:

- Shoaling (2.1)
- Depth-Induced Wave Breaking, following the formulation of Baldock et al. [24] (2.2)
- Bottom Friction, following the formulation of Jonsson [25] (2.3)

Apart from the physical processes listed above, the change in wave angle due to refraction has been added to the foreshore model. (Effects on wave height due to refraction are neglected). Snell's Law is used to calculate the change in wave angle due to refraction at the toe of the dike (2.4). The table below shows which physical processes are and which are not included in the DaF module.

$$\frac{\delta c_g E}{\delta x} + D_{bot} + D_{br} = 0 \tag{2.1}$$

$$D_{br} = \frac{3\sqrt{\pi}}{16}\alpha\rho_w g f_{rep}\left[1 + \frac{4}{3\sqrt{\pi}}\left(H_r^3 + \frac{3}{2}H_r\right)\exp\left(-H_r^2\right) - erf(H_r)\right] \tag{2.2}$$

$$D_{bot} = \frac{2}{3\pi}\rho_w f_w\left(\frac{\pi H_{rms}}{T_{rep}\sinh kh}\right)^3 \tag{2.3}$$

$$\frac{\sin\Phi_T}{\Phi_o} = \frac{c_T}{c_o} \tag{2.4}$$

Xbeach

Xbeach is a commonly used open source numerical model to predict morphological changes. Xbeach was developed to simulate coastal zones during time varying extreme storm and hurricane conditions. Xbeach solves the wave propagation, flow and sediment transport equations to calculate the development of the bathymetry in 2DH. In case of 1D refraction is ignored. Xbeach can reproduce dune erosion through an avalanching algorithm. Xbeach calculates the critical slope between the cells and exchanges sediment between the cells if the slope is exceeded. Xbeach is also capable of simulating dune erosion near hard structures, such as sea walls or revetments, quite well, in both longshore and cross-shore direction [26]. More background information about Xbeach can be found in Roelvink et al. [27].

Methods to Calculate the Failure Probability due to Wave Overtopping of Hybrids Dikes

In multiple studies the application of foreshores to reduce wave overtopping has been examined and the failure probability of the combined foreshore and dike system has been calculated.

Oosterlo (2015) developed a method to calculate the probability of dike failure due to wave overtopping, which included infragravity waves and morphological changes of a sandy foreshore. For this method the eurotop formulae were used to calculate overtopping. Xbeach was used to model hydrodynamic and morphological changes on the foreshore and the probabilistic method ADIS was used to determine a combined probability of failure.

Vuik et al. (2018) presented a method to determine the failure probability of a hybrid flood defence, integrating models and stochastic parameters that describe dike failure and wave propagation over a vegetated foreshore. This method used a one dimensional wave energy balance to model the foreshore effects, the Eurotop formulae to calculated the wave overtopping and a the probabilistic method FORM was used to calculated the probability of failure.

Lashley (2021) developed a suite of tools to quantify the influence of infragravity waves on the safety of coastal defences with shallow foreshores against wave overtopping. A method was used that relies on a new set of overtopping formulae that rely on deep-water wave parameters and a FORM [28] probabilistic method was used to estimate the safety.

An assessment of failure probability of a hybrid levee with sandy foreshore including a maintenance strategy cost optimization has not yet been done. Also the use of Riskeer to probabilistically calculate the failure probability including the DaF module to simulate the wave transformation processes on the foreshore has not been studied.

2.4. Lake Environments

As mentioned before, this thesis focuses on hybrid dikes situated in lake environments. These lake environments differ from the more studied coastal environment in various ways. For coasts, many years of research and data have been gathered. For instance, with the JarKus transects, decades of data has been gathered about the behaviour of the Dutch coastline. The data has led to the methods and formulas in coastal engineering used nowadays. Alterations are probably needed, to use the same formulas in lake environment. For example, in a lake there will be no influence of the tide. Some of the main definitons and how they apply in the Markermeer and IJmeer lakes will be explained here.

Low-Energy Beaches

One definition which could be used for a lake environment is low-energy. Low-energy was first described by Jackson et al. [29] and the term is often used for estuaries, lakes and reservoirs. The requirements are firstly non-storm significant wave heights are minimal ($< 0.25m$), where storm conditions are < 8 m/s onshore winds; secondly significant wave heights due to strong onshore winds are low ($< 0.50m$); thirdly beach face widths are narrow ($< 20m$ in microtidal environments); and lastly morphologic features include those inherited from higher energy events. [30]

As the beach at Almere Duin has not yet been constructed, it is not possible to examine if the beach face width is narrow enough or if the morphological features are inherited from higher energy events. Furthermore, there were no wave conditions measurements found for the IJmeer, therefore it is hard to categorise the beach as low-energy. However several similar but less sheltered beaches in the Markermeer have been depicted as low-energy before [31]. hence the beach is expected to be classified as low-energy when constructed.

Fetch-Limited Beach

Another definition for a lake environment is 'fetch-limited'. For a 'fetch-limited' environment the waves responsible for geomorphic change are locally generated and not from external basins. [32] Another way of distinguishing fetch-limited beaches from other low-energy beaches is that the term fetch-limited could apply to basins that are small enough that distance rather than wind duration is always a limitation to generation of the waves that dominate beach change. An arbitrary threshold of 50 km for basin width is suggested as typical for low-energy environments. The maximum fetch for Almere Duin 10 km (Google Earth), hence the Almere Duin beach could be categorized as fetch-limited.

Sheltered Beach

A 'sheltered' beach is a beach that inherits storm events from nearby high energy systems, unlike fetch-limited environments where the energy is locally generated. At a sheltered beach the wave energy has been dissipated by wave transformation, such as bottom friction and refraction. There are no nearby high energy systems, thus the Almere Duin beach cannot be classified as sheltered.

Shallow Water Depth

Finally, the effect of the shallow water depth should not be neglected. Due to the shallow average water depth of the IJmeer and Markermeer 4 meters, the effect of bottom friction on the generation of wind waves could not be neglected. With a wave steepness of 0.05 and a 4 meter water depth using the wave steepness relation ($\frac{H}{L} = s$) to find the wave length and then the limit of deep water waves ($\frac{h}{l} < 0.5$) of, this indicates the waves start to 'feel' the bottom when wave height reaches 0.4 meters. Since waves of 1.5 meters or higher could be expected for critical conditions, the shallow water depth of the lakes should definitely be considered.

2.5. Hydrodynamic Processes

To understand the morphological processes, first the hydrodynamic processes should be considered. Hydrodynamic processes are all the processes that will initiate the motion of fluids. The main driver of hydrodynamic processes in fetch-limited environments is wind energy [32]. Hydrodynamic forces due to tide and boat waves are not influential in the Markermeer. The discharge in the Markermeer will only be of influence in combination with a prolonged South or South West winds. During such a wind event the change in water level will cause the drainage sluices at the Houtribdijk to be not able to sluice water to the IJsselmeer, slowly increasing the daily water level of the whole lake. When the wind holds for a longer period, this could lead to significant higher water level in the lake. Wind energy induces two main hydrodynamic processes: waves and fluctuations in water level. Both of these processes combined with the local bathymetry interact with each other to create the local governing hydrodynamic forces of the system.

Waves

The main source of energy is the Markermeer lake are locally generated wind waves. Given the limitation of basin size, it is typically the fetch rather than the wind duration that determines wave characteristics [32]. The direction of locally generated wind waves is aligned with the direction of prevailing wind. Fetch-limited waves generally have low wave heights and short wave periods [32]. The significant wave height in non-storm conditions typically does not exceed 1 meter. [29].

Setup

The top layer of the lake is pushed by the wind to flow in the same direction. To balance the wind force a difference in water level originates. This water level difference due to the wind is called setup. In combination with the local bathymetry a 3D or 2D circulation current can arise. In the Markermeer first a 3D circulation current was found during storms by Vijverberg et al. [33], however recent data shows this is a 2D current. The figure 2.3 below (adopted from F. Wellen 2021) is an example of a 2D circulation current which occurs in the Markermeer with a South Western wind direction.[34]

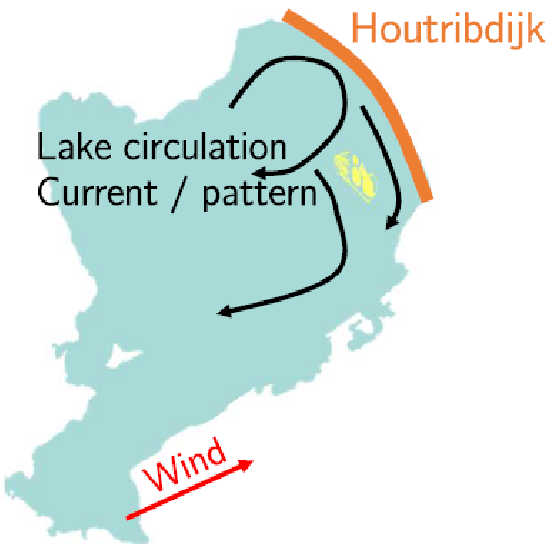


Figure 2.3: Circulation flow Markermeer adopted from Wellen (2021)

2.6. Morphological Processes

Compared to open-ocean beaches, there has been little research on the morphology of beaches in low-energy and low-fetch environments. The morphodynamics of beaches in fetch-limited environments are generally a function of wave conditions and periodic or aperiodic water level fluctuations that rework a profile shape that is partly a product of geological inheritance in terms of shoreline configuration, sediment provenance, fronting terrace and horizontal and vertical position of the foreshore relative to the terrace.[30] The erosion of the beaches in the Markermeer specifically, is primarily by storm-driven cross-shore transport, after which the sediment is most likely diffused both cross-shore and longshore over the platform and offshore sections. [35] Hence, to know the longshore transport the flow should be predicted.

The two main sediment transport mechanisms are longshore and cross-shore transport. Cross shore is sediment transport perpendicular to the shoreline. The main cross-shore transport mechanism is dune erosion, which is explained in the previous section 2.2. Longshore transport is sediment transport parallel to the shoreline. Although considered separately, erosion is often a combination of cross-shore and longshore transport.

Morphological and Hydrodynamic Processes in the Markermeer

"The effects of different hydrodynamic processes should be included in system design and its link to the bigger system (e.g. the Markermeer) has to be understood to prevent unexpected development of the beach. Moreover, we can conclude the different processes are very system- and site-specific, man-made ingenuities might have negative secondary effects on flow and beach development. This combined leads to the main recommendation: we need to change our way of thinking and designing in this kind of systems. Instead of designing a beach solemnly based on the wave climate, we need to consider the entire system. We need to consider all hydrodynamic processes such as water level set-up (or -down), lake circulation currents and its nearshore components. To conclude, we have observed that the term low-energy does not mean nothing exciting happens in our system especially when taking into consideration that we are only beginning to understand the different aspects of this and other low-energy systems and there is much more to learn." [34]

Longshore Transport

Longshore sediment transport is initiated by waves, which stir up the sediment. Thereafter sediment is transported by a flow, which could be caused by either the tide, setup or by oblique waves. Because there is no tidal flow in lakes, the longshore transport of sediment is driven by oblique waves or 2D flow caused by setup. Both of the above mentioned flows are wind driven, hence longshore transport is wind driven.

Longshore Transport Formulas

Most longshore sediment transport formula are primarily dependent on a wave height input parameter (e.g. H_{m0} or H_s) to predict sediment transport. Other common parameters to help predict LST rate are angle of incidence, the wave breaker parameter or currents. LST formulas can be distinguished in two groups. The first group are 'bulk' transport formulas; these LST formulas explicit equations are based on simplified representations of physical processes which generally use empirical coefficients for calibration. The second group of formulae are process based, where they attempt to simulate LST in detail using physical processes.

Ton et al. [3] showed that when some commonly used LST formulas are used for low-energy lake environments, the predicted LST rate could differ form observed LST rates in the Markermeer, by up to 1 or 2 orders of magnitude.

One of the more commonly used process based formula is the CROSMOR2013 formula (2.5). This formula, which is calibrated for coastal areas, is a general expression for longshore transport of sand and gravel, including the effects of profile slope and tidal velocity. Where Q_t is the alongshore sand transport (in kg/s, dry mass). 0.006 is a constant and K_{swell} is a swell correction factor for swell wave, because of the absence of swell waves on the Markermeer $K_{swell} = 1$. ρ_s is the sediment density, $\tan_{beta}$ is the slope of the beach or surf zone, $H_{s,br}$ is the significant wave height at the breakerline, which was calculated with the Bretschneider formula. Finally, V_{wave} is the wave-induced longshore current velocity (m/s) averaged over the cross-section of the surf zone. Additional flow velocities in the surf zone due to tide and wind could be included by adding them to the V_{wave}.[36]

$$Q_{t,mass} = 0.006 K_{swell} \rho_s (tan_{\beta})^{0.4} (d_{50})^{0.6} H_{s,br}^{2.6} V_{wave} \tag{2.5}$$

The van Rijn formula was tested for 22 good-quality field datastes of sand and shingle beaches by van Rijn [36]. Some of the datasets were lakes or not-tidal or microtidal environments, which are similar to the environments in this research.

Jackson et al. [37] gathered field data in a low-fetch (12-15km) microtidal (0.21m) environment and calculated LST rates with three LST formulae: Bagnold, CERC and Kamphuis. For this data Bagnold compared most favourably.

2.7. Life Cycle Costing

Life Cycle Costing (LCC) is a method which is used to examine the costs of a structure during its entire lifespan. The LCC method includes both the up front investment costs as well as the maintenance and repair costs and could even include the demolition costs. To calculate the life cycle costs, a few principles from the field of business economics were used.

The first principles is the Present Value (PV). The PV is the current value of future investments during a given time period and interest rate [38]. The PV can be calculated with:

$$\frac{C}{(1+t)^r} \tag{2.6}$$

Where C is the future cost, t is the time in years and r is the interest per year. When a stream of cash payment continues indefinite, it is considered a perpetuity [38]. When the the annual cash flow C and the annual interest rate r are known, the present value of a perpetuity can be calculated with the following formula:

$$\frac{C}{r} \tag{2.7}$$

3

Sensitivity Analyses

3.1. Wave Overtopping

3.1.1. Introduction

The aim of this chapter is to assess the influence of a foreshore on the failure probability of a dike in a low-energy non-tidal shallow lake environment. This chapter also aims to assess the resistance to erosion of different foreshores and how the erosion process of the foreshore influences the failure probability of the hybrid flood defence. To find out the influence of foreshore, different probabilities of failure are calculated for different foreshores. These calculations were done with Riskeer. Riskeer is an application for the assessment and design of primary flood defences in the Netherlands and uses the WBI2017 standards. The chosen failure mechanism for the different foreshores overtopping, called GEKB (grass erosion crest innerslope).

3.1.2. Method

Design criteria: the normative safety standard (P_f) for the IJmeerdijk is 1/10,000 per year. 24% of the failure probability share (ω) is for GEKB (overtopping) and according to the WBI the length effect factor N = 3 for this dike section. Thus calculated with formula (3.1), the required maximum failure probability for the cross section for overtopping is 1/125,000 per year.

$$P_{crosssection:OT} = \frac{\omega_{OT} * P_f}{N} \tag{3.1}$$

To design a foreshore, first the failure probability of a foreshore profile needs to be computed in Riskeer and meet the required safety standard. Then the profile needs to be tested for dune erosion in Xbeach. Whereafter the profile needs to be reschematised for Riskeer to calculate if the eroded profile still meets the required standards.

For Riskeer a schematization of the existing profile needs to be given. The profile consists out of two parts, the foreshore and the dike profile. The foreshore is where Riskeer calculates wave dissipation using the DaF module (dams and foreshores). Thus calculating the wave breaking effect of the foreshore. For the dike profile, the overtopping is calculated. Riskeer automatically assesses profiles. Slopes steeper than 1:8 will be seen as dike slopes, slopes gentler than 1:10 are seen as a berm by Riskeer. Slopes in-between 1:8 and 1:10 are not allowed. Riskeer will assess whether a berm is present and if the berm height is close enough to the water level, to an overtopping reducing factor will be applied for the berm. For the dike slope, Riskeer will use the average slope of the profile to calculate the overtopping according to the Eurotop manual[11].

For a cross section profile, there are four important parameters. First the height of the foreshore, second the slope of the foreshore, third the slope of the dike and fourth the height of the crest. The crest level will be kept constant to assess the influence of the foreshore. The dike slope will be assessed, even though it is not part of the foreshore, because the dike slope is influenced by the foreshore. The foreshore will influence the dune erosion which will influence the slope of the dike.

Hydraulic Loads

To calculate the failure probability, first the hydraulic loads need to be known. Hydraulic loads are provided from a database by Rijkswaterstaat for all around the Netherlands. They contain wave parameters such as height, period and direction and also the lake levels, wind and setup. Also, they contain the probability of occurrence of these parameters. Based on all these probabilities and wave parameters, the hydraulic load is calculated. For a probabilistic assessment for a certain failure mechanism, the probabilistic distribution of the hydraulic load is combined with the probabilistic distribution of the strength of the dike.

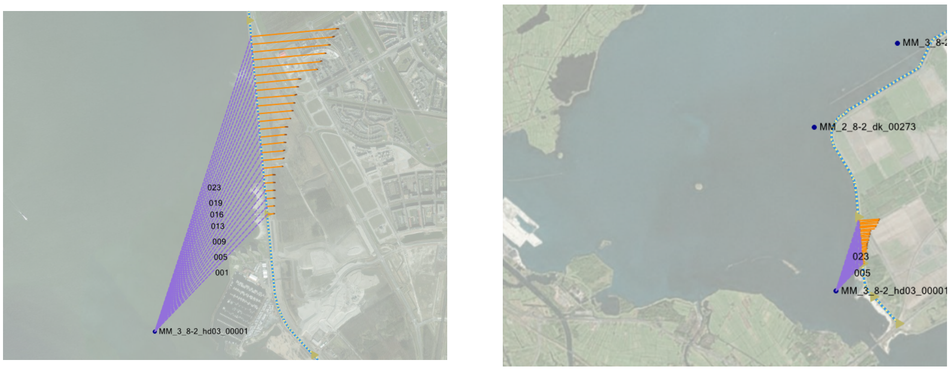


Figure 3.1: Locations of Hydraulic loads

In Riskeer, the hydraulic loads are given for nearshore locations. The closest hydraulic load location is chosen which is located a bit south east of the dike (3.1). The hydraulic loads are including the 0.3 meters of maximum lake level rise allowed by Rijkswaterstaat. They are approximately 1 kilometer from the dike. The used database is the WBI 2017 Markermeer database.

Schematic Profile

To assess the influence of the foreshore, 24 simplified cross section of a foreshore were calculated for overtopping. They were referred to as “schematic profiles”. The influence of two parameters of the foreshore was analysed: the height of the foreshore and the slope of the foreshore. The foreshore height is the highest point of the foreshore where the foreshore ends and the dike begins. This point is usually referred to as the dike toe.

The slopes of 1:10, 1:30, 1:50 or 1:100 were tested and the level of the dike toe, where the foreshore ends and the dike begins of 0, 0.3, 0.6, 0.9, 1.2 or 1.5 meter were tested. The crest of the dike will be constant at 2.70 m +NAP and the slope of the dike will be constant at 1:3.

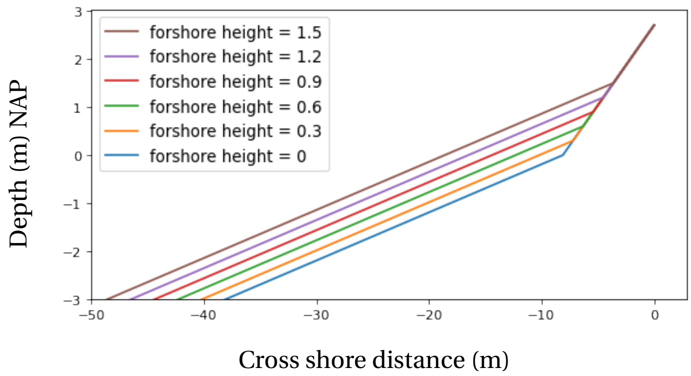


Figure 3.2: Cross section of the schematic profiles

Because the minimal slope for a foreshore is 1:10 in Riskeer, this is chosen as the minimal slope. The expected slope is around 1:30 because this is the slope used for the HRD. Two gentler slopes are added to research the additional strength of the foreshore slope. The range of 0 to 1.5 meter toe height is chosen because this is the expected range of the foreshore.

3.1.3. Results

The figures show the calculated failure probability for 0.1 l/s/m of overtopping. The blue line represent the lower limit which minimal required failure probability for a flood defence. The green line represents the alert value, if a flood defences exceeds this value, the secretary of infrastructure is informed and the reinforcement can be initiated. The yellow, grey, orange and dark blue line respectively represent the 1:100, 1:50, 1:30 and 1:10 slopes and illustrate the failure probability for different combinations of foreshore slope and foreshore height.

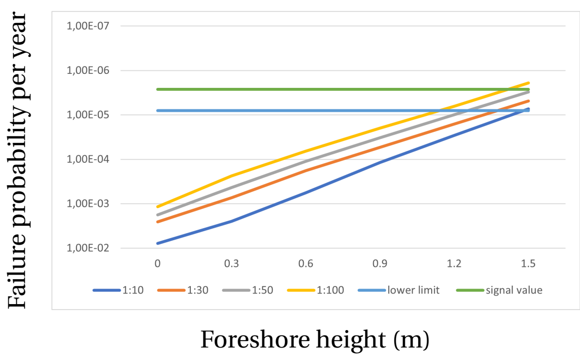


Figure 3.3: Failure probabilities of schematic profiles with 0.1 l/s/m of overtopping. Each line represents a foreshore slope and illustrates the failure probability of a foreshore with this slope and a foreshore height.

For 0.1 liters of overtopping, only the profile with a foreshore slope of 1:100 and a height of 1.5 meter suffices for the signal value and the minimal required safety standard is satisfied by foreshores with a height of 1.2 to 1.5 meters, depending on the slope.

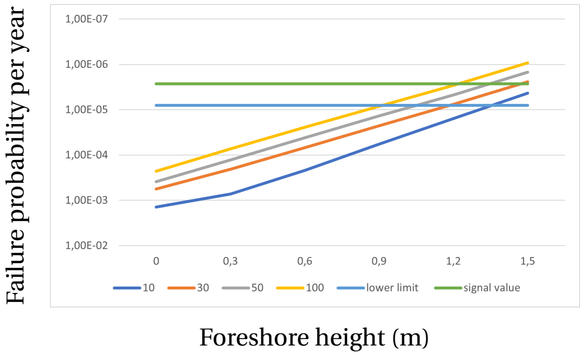


Figure 3.4: Failure probabilities of schematic profiles with 1.0 l/s/m of overtopping. Each line represents a foreshore slope and illustrates the failure probability of a foreshore with this slope and a foreshore height.

For 1.0 liters of overtopping, the profiles with a foreshore height of 1.2 to 1.5 meter suffices, depending on the slope and for the signal value and the minimal required safety standard is satisfied by foreshores with a height of 0.9 to 1.4 meters, depending on the slope.

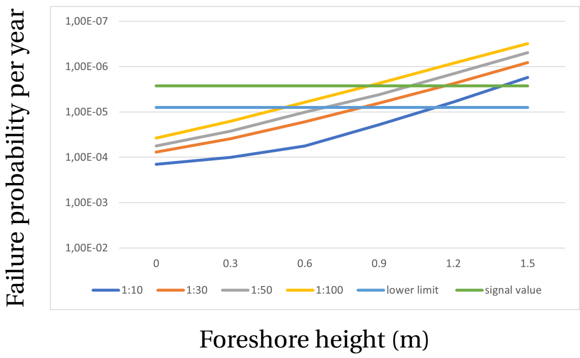


Figure 3.5: Failure probabilities of schematic profiles with 10 l/s/m of overtopping. Each line represents a foreshore slope and illustrates the failure probability of a foreshore with this slope and a foreshore height.

For 10 liters of overtopping, the profiles with a foreshore height of 0.9 to 1.4 meter suffices, depending on the slope and for the signal value. The minimal required safety standard is satisfied by foreshores with a height of 0.5 to 1.1 meters, depending on the slope.

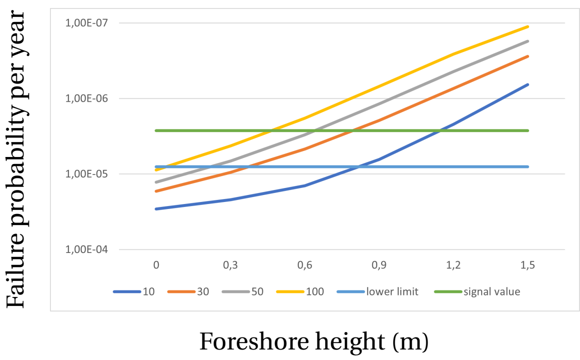


Figure 3.6: Failure probabilities of schematic profiles with a 'gesloten zode'. Each line represents a foreshore slope and illustrates the failure probability of a foreshore with this slope and a foreshore height.

Figure 3.6 illustrates the calculated overtopping for a “gesloten zode” respectively. A “gesloten zode” is allowed for a mean overtopping of 100 l/s/m with a lognormal deviation of 120 l/s/m for waves of 1 to 2 meters. For 0 to 1 meters a mean overtopping discharge of 225 l/s/m and a standard deviation of 250 l/s/m is allowed by the manual. For an ’gesloten zode’ a foreshore height of around 0 to 0.8 m would, depending on the foreshore slope, satisfy the minimal required safety standard. A foreshore height of 0.4 to 1.2 meters would meet the signal value.

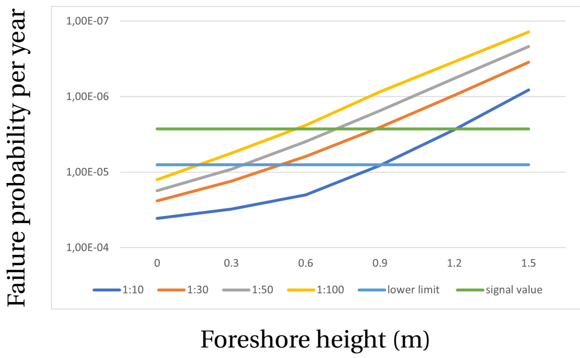


Figure 3.7: Failure probabilities of schematic profiles with a ’open zode’. Each line represents a foreshore slope and illustrates the failure probability of a foreshore with this slope and a foreshore height.

Figure 3.7 presents the calculated failure probability overtopping for an “open zode”. An “open zode” is allowed for a mean overtopping of 70 l/s/m with a lognormal deviation of 80 l/s/m for waves of 1 to 2 meters. For 0 to 1 meters, a mean overtopping discharge of 100 l/s/m and a standard deviation of 120 l/s/m is allowed by the manual. For an open zode, a foreshore height of around 0.1 to 0.9 would, depending on the foreshore slope, satisfy the minimal required safety standard.

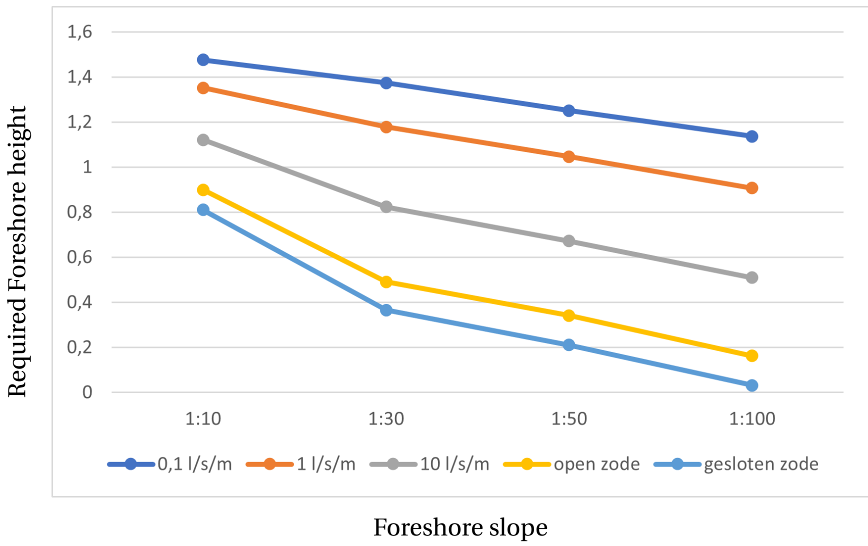


Figure 3.8: Required minimal foreshore height for a slope and discharge. Each line represents a discharge and illustrates required foreshore height for a slope and a discharge.

Figure 3.8 presents the required minimal foreshore height for each combination of overtopping discharge and foreshore slope.

3.1.4. Conclusion

From calculating the failure probability of the different cross sections, the following conclusions can be made: When the allowed overtopping discharge increases by a factor of 10, the failure probability decrease by a factor of 3, and increasing the foreshore height by 30cm decreases the failure probability by a factor of 3 to 5. To demonstrate this with a quick simplified calculation: For the open zode to satisfy the safety standard for a 1:100 profile, a foreshore height of 0.1 m is required and for a 1:10 profile is required. When assumed the depth is 3 meter (for more detail about the bathymetry see 1.8), 76 $\frac{m^3}{m}$ is required for the gentler slope and 480 $\frac{m^3}{m}$ is required for the steeper slope. Thus, to decrease the failure probability, increasing the foreshore height is more effective than decreasing the slope of the foreshore, in terms of cubic meter of sand required for the construction.

3.2. Sensitivity analysis of the morphological changes of the foreshore during an extreme event

3.2.1. Method

For the sensitivity analyses, the one at a time (OAT) method was used. This is a simple and common approach for sensitivity analyses. One input variable is changed, whilst keeping the others the same. Xbeach 2.3 is a complex numerical model with many different input variables. This analysis has the aim to get an insight in which of the parameters are of importance and how important they are. This will help to find the optimal beach foreshore profile.

Multiple simulations were performed where each time one input variable was changed, whilst keeping the others the same. Subsequently, the input variable was set back to its original value and another variable was changed. This was applied for all parameters. Unfortunately, the OAT method could not find the interaction between different input variables. It is important to notice when only changing one parameter, that parameters could influence each other. Thus these results should be interpreted carefully.

Parameter	Standard Value
Number of grid points	50
Angle of incidence	0
Water Density	1000 $\frac{kg}{m^3}$^3
Wave Height	1.5 m
Wave Period	5 s
Grain Size	230 μm
Storm Duration	35 hours
Storm shape	35 trapezoid
Peak of storm duration	2 hours
Setup level	2 m
Lake level	+ 0.1 m NAP
Gamma	3.3
Asymmetry (FacAs+)	Default (0.1)
Skewness (FacSk+)	Default (0.1)

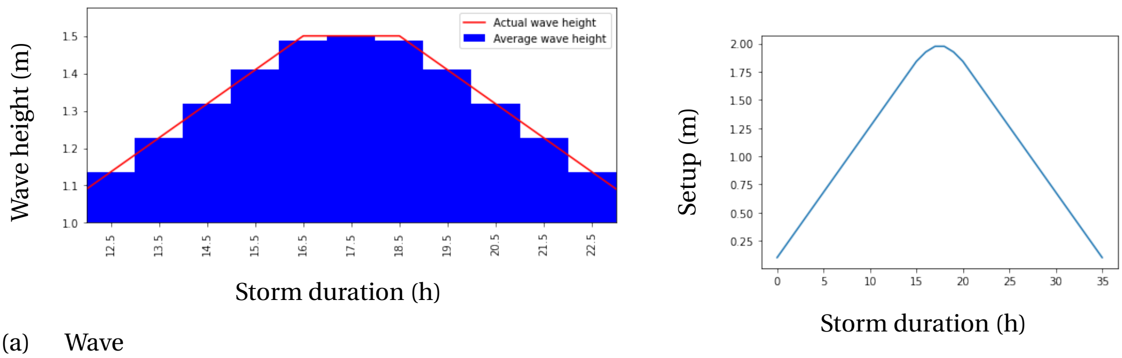
Table 3.1: Standard input for the sensitivity analysis

The standard input, shown in table 3.1 was mostly a first guess for the expected input at Almere Duin. First the standard setup was determined. The setup level was 2.0 meters, as this was close to the setup of the design point of the schematic profile, with a foreshore height of 1.5 meters and a 1:100 slope, with 0.1 l/s/m of overtopping (shown in 3.3). The wave height (H_{m0}) was set to 1.5 meter (the design point in Riskeer does not give the offshore wave height).

The WBI prescribes a storm duration of 35 hours for the Dutch water systems[13]. Also, they prescribe the water level during the storm. The water level will rise linearly at the beginning of the storm from the daily lake level until the peak level minus 0.1 m is reached two hours before the halfway point. Then to the peak level at the middle of the storm, continuing to the peak level minus 0.1 meter two hours after the halfway point and then back to the daily lake level at the end of the storm. The water level during the storm is shown in the figure 3.9b.

The daily lake level is raised 0.30 meters due to climate change until the year 2125. The daily lake level is assumed to increase equally and will be + 0.1 m NAP. The peak for the wave height during the storm will be 2 hours and has a trapezoid shape, with 0 wave height at t = 0 and t = 35 hours. For the input, the average wave height over an hour will be used. Thus, for the first hour of waves the wave height at t = 0.5 hours will be used and for the second hour of waves the wave height at t = 1.5 hours will be used etc. This is illustrated in figure 3.9a.

As calculated in chapter 3.1, most cross sections have a design point at 330°, the others had North (0°). However refraction is not taken into account, because a 1D Xbeach simulations does not calculate refraction 2.3. Therefore this is the angle at which the waves reach the beach. For the cross section, the schematic profile with a foreshore slope of 1:20 and a foreshore height at 1.5 meter is chosen. Rho is expected to be 1000 $\frac{kg}{m^3}$ in a fresh water lake like the Markermeer. For the Houtribdijk. another dike with a foreshore in the Markermeer, a d_{50} of 250 μm was used [39]. For the Houtribdijk local sand was used, thus, a similar grain size can be expected to be found in the Markermeer and used for this project as well. To be safe, a grain size of 230 μm and a D_{90} of 460 μm were used for this analysis. Finally, there is a hard layer present corresponding to the current block revetment and the dike above the revetment is modelled as sand.



(a) Wave height per hour

(b) Setup level during a synthetic storm according to the WBI 2017

Figure 3.9: Setup level and wave height during a synthetic storm

3.2.2. Results

In this section the results of the sensitivity analyses are presented. The results are divided into two parts: an analysis of the load and an analysis of the strength. In the load subsection, the wave height, wave period, setup, grain size and duration of the peak were examined. For the strength subsection, different variations of foreshore height and slope were tested, with and without revetment. An analysis of the minimal required grid points was performed beforehand.

Load

Wave Height

To determine the influence of the wave height, a run was performed with four different wave heights. The calculated 1:10.000 wave height is around 1.5 meters. Wave heights of 0.5, 1.0, 1.5, and 2.0 meters were used. The wave period corresponding with the wave heights are chosen to have a constant steepness = 0.07. The wave periods were calculated with the wave heights using intermediate depth wave formula (3.2).

$$L = \frac{gT^2}{2\pi} \tanh 2\pi \frac{d}{L} \tag{3.2}$$

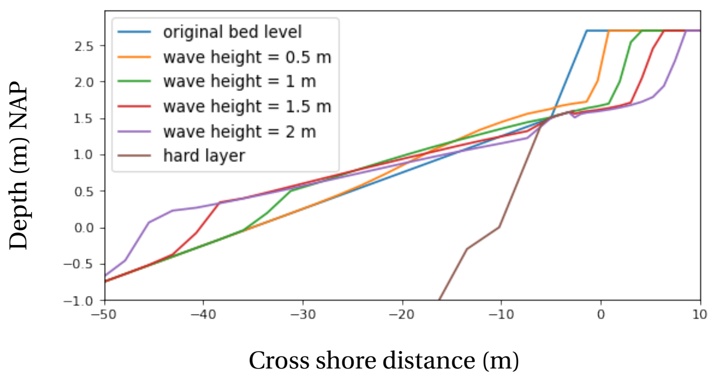


Figure 3.10: Dune erosion for different wave heights, simulated in Xbeach

Figure 3.10 shows the dune erosion for different wave heights after 35 hours of storm. It shows that under these circumstances the larger wave heights cause the dune to erode significantly faster. It is worth noting that the profile of the different wave heights have the same slope only the length of the profile differs. This data suggests that the wave height is an important parameter.

Wave Period

To determine the influence of the wave period, a run was performed with three different wave periods. The wave period were 5, 7.5 and 10 seconds respectively. The wave height was kept constant in this analysis, therefore each wave period had a different steepness.

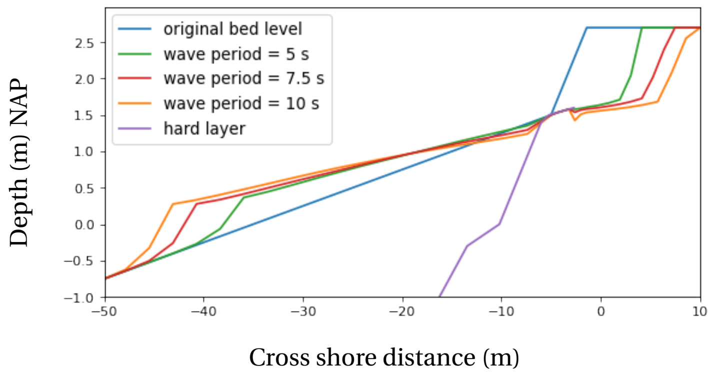


Figure 3.11: Dune erosion for different wave periods, simulated in Xbeach

Figure 3.11 illustrates the dune erosion for different wave periods after a full storm of 35 hours. It is clearly visible that the lower wave periods cause the dune to erode faster. This data indicates a likeliness that, under these circumstances, the wave period is an important parameter. Even for the same number of waves the dune erosion increases.

Setup

To determine the influence of the setup, a run was executed with three different wave heights. The setup heights were 1.5, 1.75 and 2.0 meters. The height of 1.5 meters is equal to the foreshore height, this is the minimal height for wave overtopping to still be within the scope of main failure mechanism.

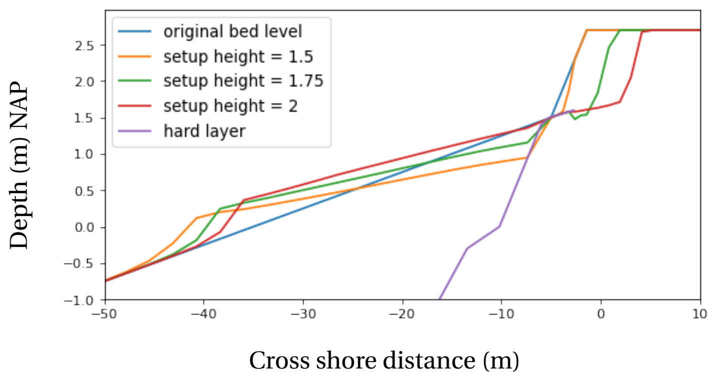


Figure 3.12: Dune erosion for different setup levels, simulated in Xbeach

Figure 3.12 shows the dune erosion for different wave heights after a full storm of 35 hours. It shows a higher setup causes the dune to erode significantly more. The data suggest that under these circumstances the setup is an important parameter.

Grain Size

To determine the influence of the grain size a run was executed with three different wave heights. The grain size were 200, 250 and 300 microns.

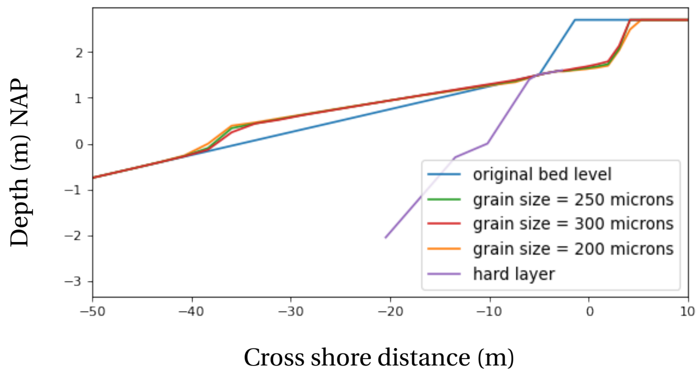


Figure 3.13: Dune erosion for different grain sizes, simulated in Xbeach

Figure 3.13 shows only a small difference in erosion for the chosen range of grain sizes. A small difference can be detected, where the smaller grain sizes erode a bit more than the larger grain sizes. The data suggests a likeliness that the grain size does not play a significant role of importance.

Duration of the Peak of the Storm

To determine the influence of the duration of the peak of the storm, an analysis was performed with three different peak durations. The durations lasted 2, 5 and 10 hours.

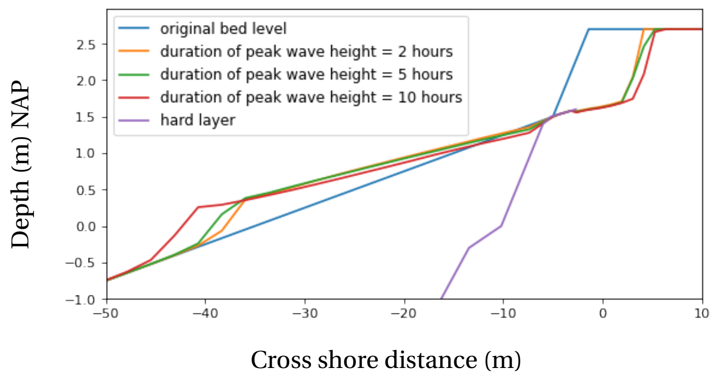


Figure 3.14: Dune erosion for different durations of the peak of the storm, simulated in Xbeach

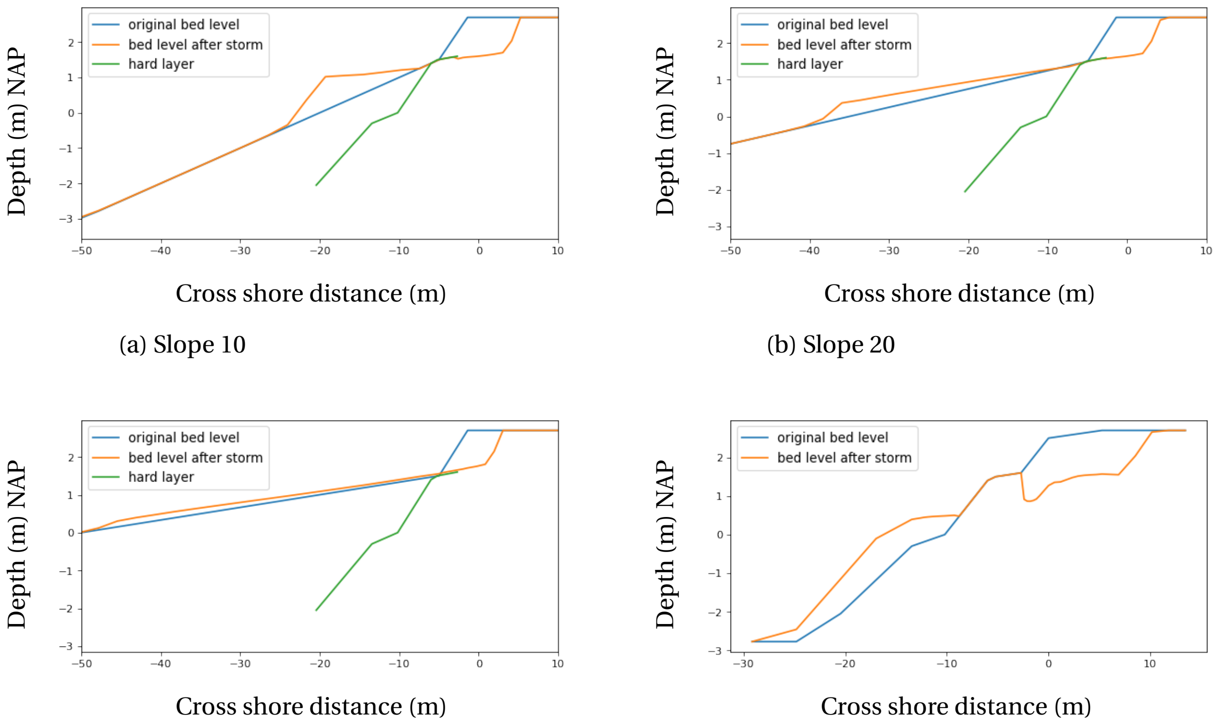
Figure 3.14 shows only a small difference in erosion for the chosen duration of the peak of the storm. A small difference can be detected, showing that the longer peak duration (10 hours) erodes a bit further than the shorter duration (2 hours). The data suggests a likeliness that the the duration of the peak wave height does not play a significant role.

Grid Variations

In this section different layouts of the dike were examined for dune erosion. First four different slope gradients were examined, with the dike modelled as a soft layer. Subsequently, different foreshore height were examined, with the dike modelled as a soft layer. Afterwards, different foreshore height were examined, with the dike modelled as a hard layer. Thereafter, different foreshore height were examined, with the whole structure modelled as a soft layer. Lastly, different foreshore heights were examined for dune erosion with a the whole dike modelled as a hard structure and a small dune of 5 meters width present in front of the dike.

Slopes

In this section different foreshore slopes were examined for dune erosion. Both the foreshore and dike were modelled as sand and a hard layer was present, where the current block revetment is situated. The foreshore height was set at 1.5 meters and the slopes are 1:10, 1:20 and 1:30 and the current bed profile of the case study. There is a hard layer present corresponding to the current block revetment.



(a) Slope 10

(b) Slope 20

(c) Slope 30

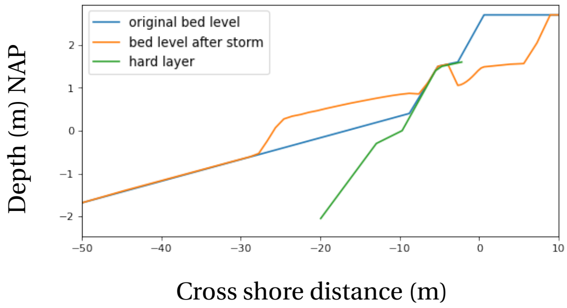
(d) Current cross shore profile

Figure 3.15: Different slopes of the foreshore during a synthetic storm, with both the foreshore and dike modelled as sand and a hard layer present, simulated in Xbeach

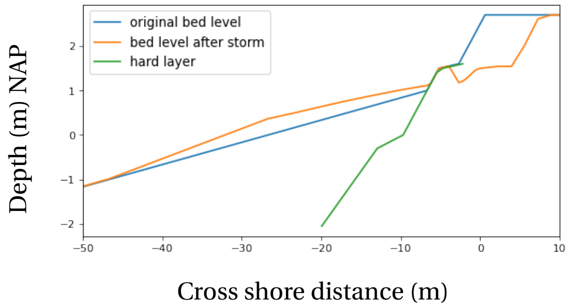
The figure 3.15d illustrates only a minor difference in horizontal erosion of the dike profile for the threes foreshore slopes. The main difference is the length of the foreshore over which the sand is distributed. The foreshores significantly reduce the erosion compared to the current layout without a foreshore.

Foreshore Height

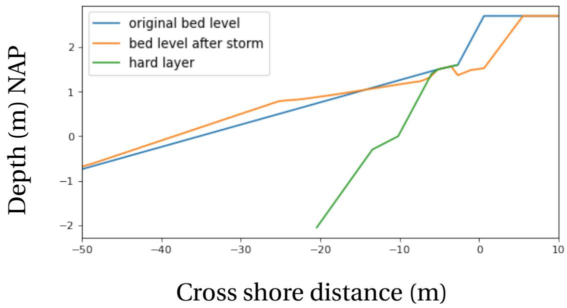
In this section different foreshore heights were examined for dune erosion. The foreshore slope was set at 1:20 and the foreshore heights are 0.5, 1.0 and 1.5 meter and additionally the current profile without foreshore is examined. There is a hard layer present corresponding to the current block revetment. Both the dike and the foreshore are modelled as a soft layer.



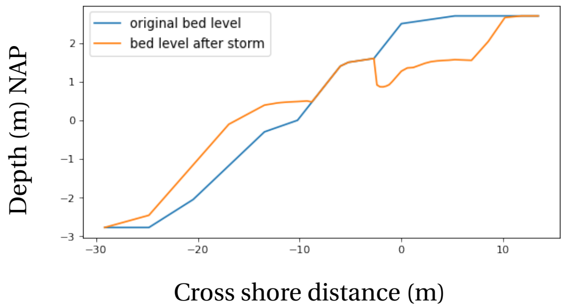
(a) Foreshore height at 0.5 m NAP



(b) Foreshore height at 1.0 m NAP



(c) Foreshore height at 1.5 m NAP



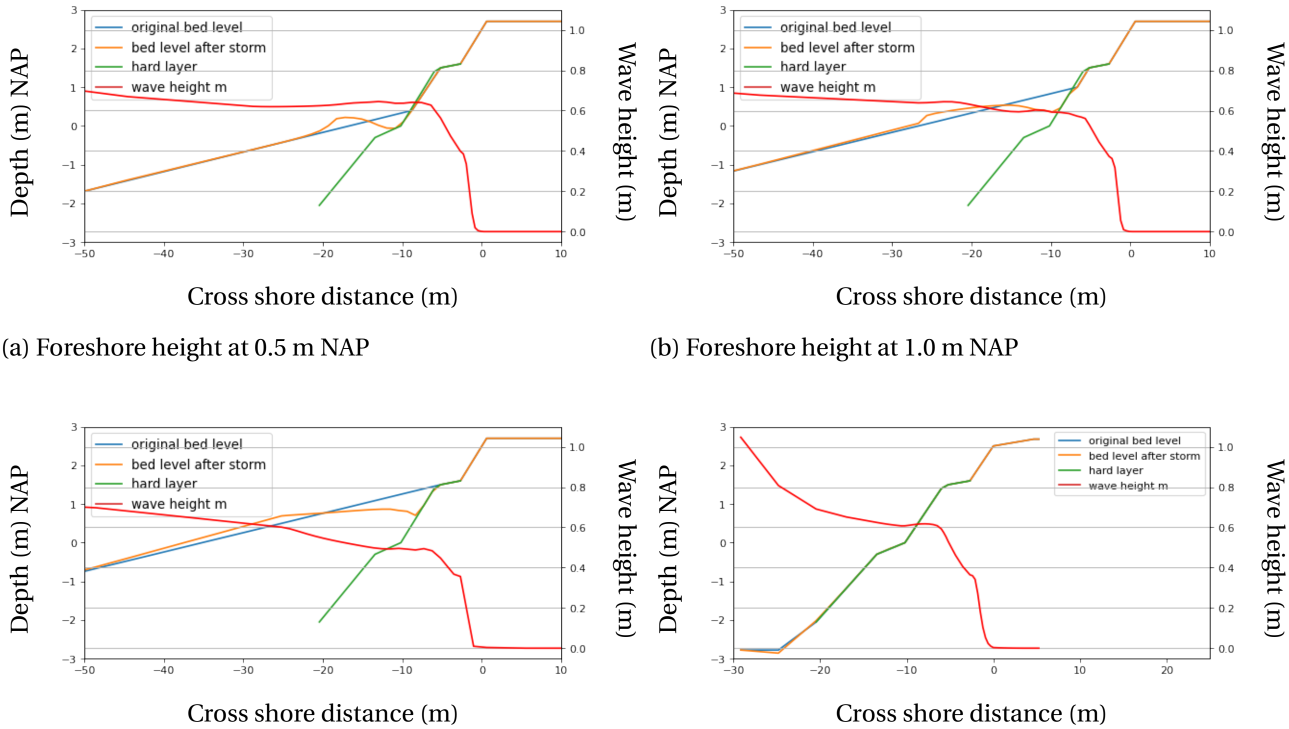
(d) Current cross shore profile

Figure 3.16: Different levels of foreshore heights, with both the foreshore and dike modelled as sand and a hard layer, which represents the block revetment. Simulated in Xbeach

Figure 3.16d shows a difference in horizontal erosion of the dike profile. The lower foreshore height of 0.5 meter results in approximately half the erosion of the foreshore height of 1.5 meters. The foreshores significantly reduce the erosion compared to the current layout without foreshore. There is a scour hole present after the hard layer. This hole would probably contribute to the collapse of the hard layer which could lead to more erosion, therefore concluding the simulation is inaccurate.

Grass Upper Layer modelled as hard layer

In this section, different foreshore heights were examined for dune erosion. The foreshore slope is set at 1:20 and the heights are 0.5, 1.0 and 1.5 meter. There is a hard layer present corresponding to the current block revetment. In Xbeach it is not possible to simulate grass. Therefore the grass was simulated as a hard layer and an extra check was performed to verify the wave height at the grass would not exceed 0.5 meters. The foreshore was modelled as a soft layer.



(a) Foreshore height at 0.5 m NAP

(b) Foreshore height at 1.0 m NAP

(c) Foreshore height at 1.5 m NAP

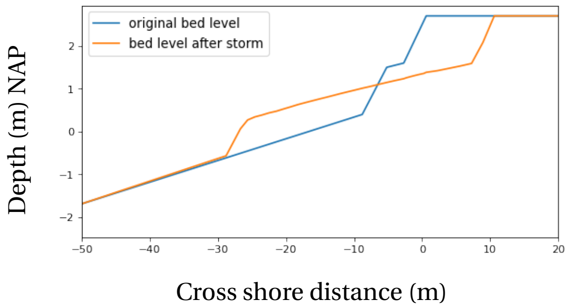
(d) Current cross shore profile

Figure 3.17: Different levels of foreshore heights, modelled in Xbeach. The grass dike was modelled as a hard layer and the foreshore modelled as sand.

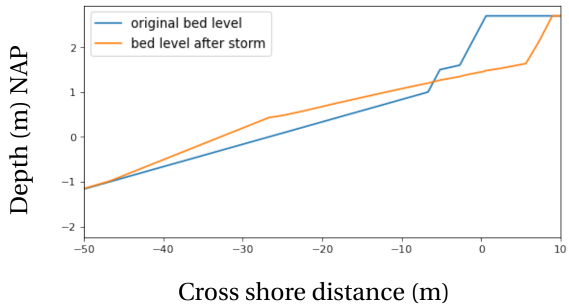
Figure 3.17d illustrates for each layout with a foreshore only a small scour hole emerges after the storm. For all three foreshore heights similar scour holes were found. The three scour holes all have a depth of approximately 0.5 meters. For the current layout without foreshore no scour hole was found. This was due to the fact that with the absence of a erodible foreshore no dune erosion could have taken place. Only a very tiny amount of dune erosion is visible at 2.5 meters depth. The red line illustrates the wave height over the foreshore, when the waves reach the end of the revetment, the wave height is reduced to approximately 0.4 meter. Hence, proving the plausibility to model the grass as a hard layer.

Soft Dike

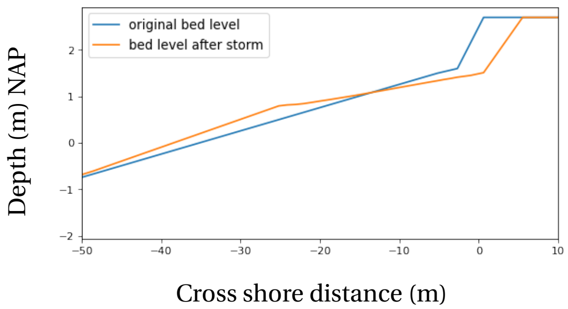
In this section different foreshore heights were examined for dune erosion with no hard layer present. The foreshore slope is set at 1:20 and the heights are 0.5, 1.0 and 1.5 meter and in the final simulation no foreshore was present. There is no hard layer present to simulate the amount of dune erosion of a dune.



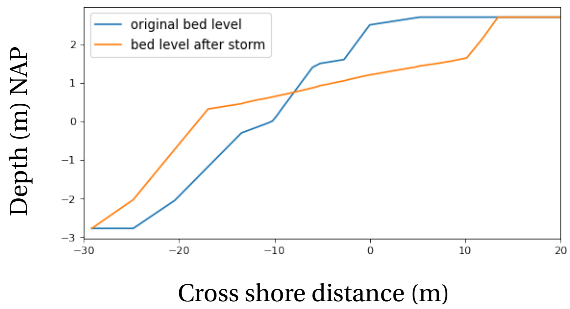
(a) Foreshore height at 0.5 m NAP



(b) Foreshore height at 1.0 m NAP



(c) Foreshore height at 1.5 m NAP



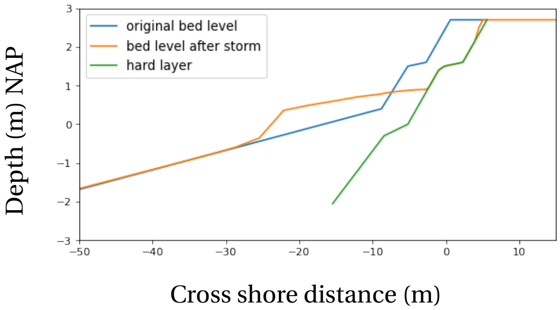
(d) No foreshore

Figure 3.18: Different levels of foreshores, with both the foreshore and dike modelled as sand, simulated in Xbeach

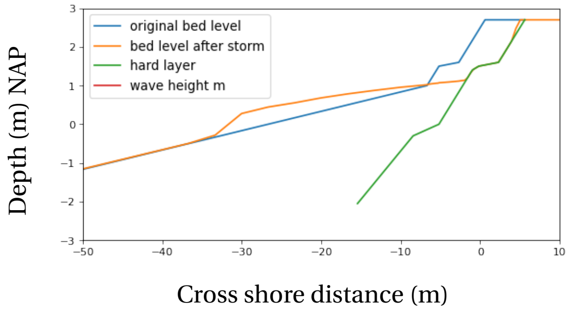
Figure 3.18d portrays there is a slight difference in dune erosion between the foreshores at 0.5 m NAP and at 1 m NAP and a larger reduction of dune erosion of the foreshore at 1.5 meter height compared to the simulation without a foreshore. This shows a foreshore does reduce the amount of dune erosion during a storm event.

Hard Layer with Sandy Dune

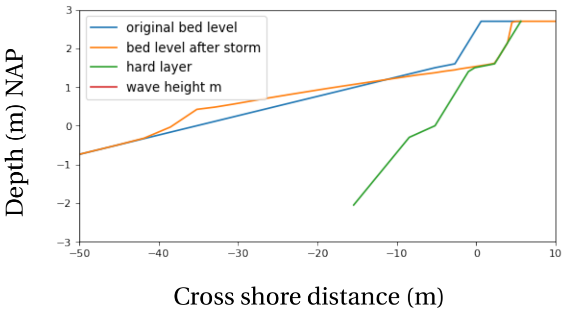
In this section, different foreshore heights were examined for dune erosion with a hard layer with a small dune of 5 meters width present in front of the dike. The dike is schematised as a hard layer and the foreshore and small dune as sand. The foreshore slope is set at 1:20 and the heights are 0.5, 1.0 and 1.5 meter and in the final simulation no foreshore was present.



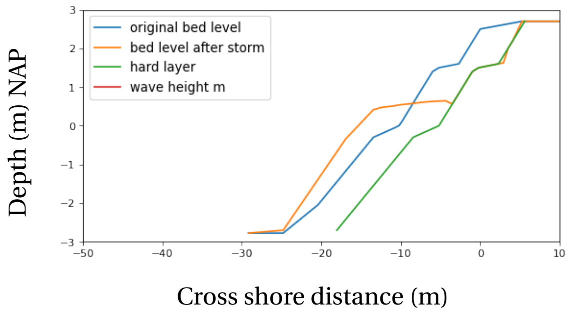
(a) Foreshore height at 0.5 m NAP



(b) Foreshore height at 1.0 m NAP



(c) Foreshore height at 1.5 m NAP



(d) No foreshore

Figure 3.19: Different levels of foreshore with both the foreshore and dike modelled as sand and a sand dune present 5 meters in front of the dike. Simulated in Xbeach

Figure 3.19d ilustrates that for each foreshore height the 5 meters of dune in front of the dike erodes. The foreshore at 1.5 and 1.0 meters remained the same after erosion. The foreshore at 0.5 meters the foreshore height increases by 0.5 meters and for the simulation without a foreshore a small foreshore appeared in front of the dune at around 0.5 m NAP, from the sand eroded from the top of the dune.

3.2.3. Conclusion

A sensitivity analysis of different parameters, foreshore layouts and hard structures was performed. This analysis gave insights into the importance of different inputs in Xbeach for dune erosion. The setup, wave height and wave period were found to be important parameters in Xbeach. The duration of the peak of the storm and the grain size were found not to be important parameters in Xbeach. This agrees with other findings.

Hard layers were found to influence the amount of dune erosion. The top of the foreshore only eroded when a complete hard structure was present. For the other layouts the foreshore height was higher after the storm, because the dike slope eroded onto the foreshore, which could decrease the failure probability. Also both increasing and decreasing the slope and foreshore height influenced the amount of dune erosion, especially compared to the absence of a foreshore. It is advised to use a layout where the dike slope is modelled as a non-erodible layer.

4

Probability of Failure

4.1. Introduction

In this chapter a step-wise calculation of the failure probability for wave overtopping of a hybrid structure was carried out. The step wise calculation consists out of four steps which are presented below. In this research method, overtopping was considered as the dominant failure mechanism and dune erosion as a sub-mechanism of overtopping.

1. Calculate failure probability for overtopping in Riskeer.
2. Calculate dune erosion.
3. Schematise the profile from Xbeach and calculate failure probability for overtopping again in Riskeer.
4. Check if design point matches the design point in step 1, if not repeat step 2 and 3 with new design point, until the new design points fits the old design point.

The first step is to calculate the failure probability of a hybrid levee in Riskeer for overtopping. Subsequently the offshore design point was calculated in Riskeer. The next step is to model the dune erosion of the profile in Xbeach for the design point calculated in the previous step. Lastly a schematization of the eroded profile was made and calculated again for overtopping in Riskeer. If the Failure probability increases or decrease too much, another iteration is necessary. In the next iteration the design point from the eroded profile is simulated again in Xbeach and then calculated again for overtopping. If the failure probability decrease or stays equal the calculation is done, otherwise another iteration is needed. The method is illustrated in figure 4.1

This method assumed the storm conditions, which lead to the critical overtopping discharge, also lead to the critical dune erosion. This assumption is examined in appendix A and it was suggested that the assumption was correct

In chapter 3.2.2, different layouts were assessed. In this chapter, the layout with the grass upper layer modelled as a hard layer and the sandy foreshore modelled as an soft layer was chosen for this simulation, as this is the most accurate schematization of the actual layout at Almere Duin. This layout showed significantly less erosion of the foreshore than the other layouts. Therefore, less erosion of the foreshore is expected in this chapter compared to the chapter 3.

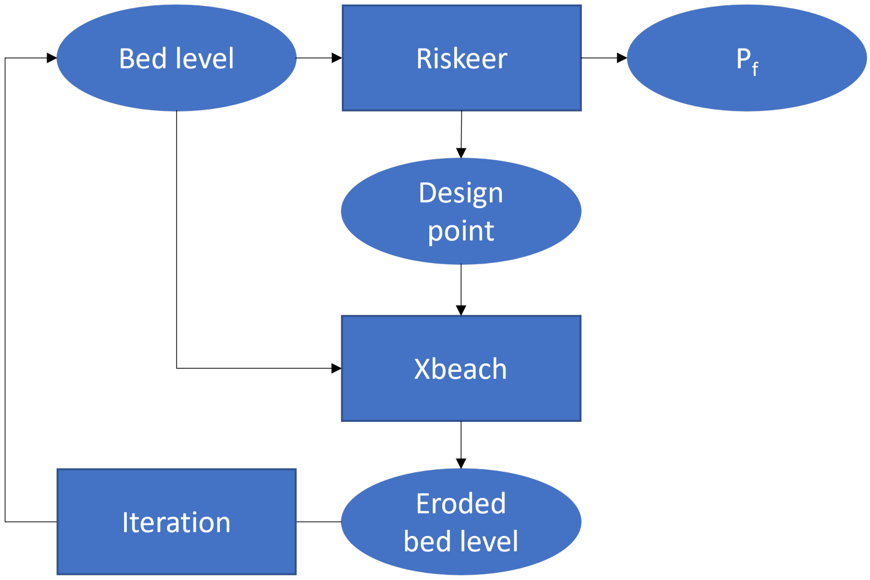


Figure 4.1: Flow chart presenting the method to calculate the failure probability and the iterative process to check the design point

4.2. Method

Foreshore Profiles

To test the step-wise calculation method of the failure probability for wave overtopping of a hybrid structure, the case study of Almere Duin is used. For Almere Duin the minimal volume of sandy foreshore, which meets the safety standard, is attempted to be found with this method. A simple foreshore with a triangular shape and a slope of 1:10 is modelled. Three different foreshore profiles with different foreshore heights are examined to meet the required safety standard of 1/125000 per year. The three profiles, shown in figure 4.2, had a dike slope and crest height equal to the current dike (1:3 and NAP+2,70m) and at the dike toe the foreshore connects to the dike and then the foreshore has a slope of 1:10 until the bottom at -2,50m NAP. The foreshore is modelled as a soft layer and the dike is modelled as hard layer, as explained in chapter 3, this is allowed when the wave height is reduced by the foreshore to less then 0.5 meter, when the waves reach the dike slope. The foreshore levels for the different calculations are summarized in table 4.1.

	Slope	**Height**
Foreshore profile 1	1:10	1.75 m
Foreshore profile 2	1:10	1.80 m
Foreshore profile 3	1:10	1.85 m

Table 4.1: Different profiles for assessment

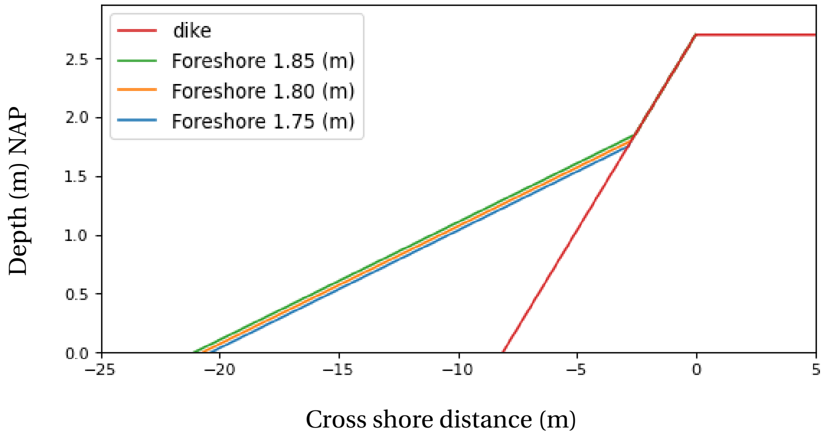


Figure 4.2: Bed profiles of the three different foreshores, which are calculated for wave overtopping

Step 1: Calculate failure probability for overtopping in Riskeer

The failure probability was calculated in Riskeer. Design criteria: the normative safety standard (P_f) for the IJmeerdijk was 1/10,000 per year and 24% of the failure probability share (ω) was for GEKB (overtopping). According to the WBI the length effect factor N = 3 was prescribed for this dike section. This probability of failure at cross section level was calculated with the formula (4.1). Hence the required maximum failure probability for the cross section for overtopping is 1/125,000 per year.

$$P_{crosssection:OT} = \frac{\omega_{OT} * P_f}{N} \tag{4.1}$$

The critical overtopping discharge had a mean of 0.1 l/s/m and a standard deviation of 0. The failure probability was calculated in Riskeer. The closest Hydraulic load was chosen, as explained in the section 3.1. Wave conditions from each design point were used for the next step.

Step 2: Calculate dune erosion in 1D Xbeach

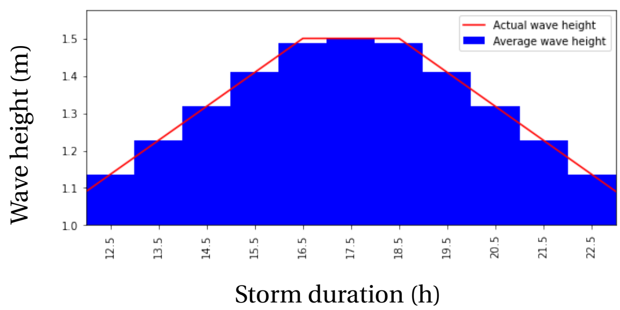
First the wave conditions from the design point in Riskeer are needed as input for the wave parameters. Riskeer only gives a onshore design point, which includes the wave conditions after the waves have passed the foreshore. To simulate the effect of the waves on the foreshore, the wave conditions at the offshore location are needed. To find these offshore design conditions, the GEBU (grass erosion outer slope) mode in Riskeer is used. As mentioned in the literature 2.3, the DaF module in Riskeer does not include the setup of water level over the foreshore. The GEBU mode can calculate the maximum wave height for a given setup level and probability of occurrence. Thus, for the setup level in the local design point the wave conditions offshore can be recalculated.

The number of grid points was 37. Rho is 1000 kg/m3 for fresh water. The wave height, wave period and setup was used from the design point in Riskeer. The wave period was divided by 1.1 to go from a peak period T_p to a spectral mean wave period $T_{m-1,0}$. For the beach of another dike, the Houtribdijk, with a foreshore in the Markermeer, a D_{50} of 250 µm was used [39]. Thus, such a grain size can be expected to be found in the Markermeer. To be safe, a grain size of 230 µm was used. The d_{90} was 460 micrometres.

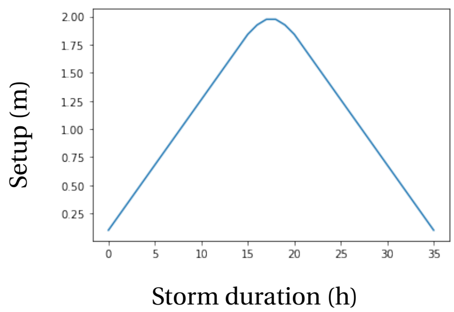
Parameter	Standard Value
Number of grid points	37
Angle of incidence	0
Water Density	1000 kg/m^3
Wave Height	From the design point in Riskeer (m)
Wave Period	From the design point in Riskeer (s)
Grain Size	230 μm
Storm Duration	35 hours
Storm shape	35 trapezoid
Peak of storm duration	2 hours
Setup level	From the design point in Riskeer (m)
Lake level	+ 0.1 m NAP
Gamma	3.3
Asymmetry (FacAs+)	Default (0.1)
Skewness (FacSk+)	Default (0.1)
Hard layer	On

Table 4.2: Input for Xbeach

The WBI prescribes a storm duration of 35 hours for the Dutch water systems. Also , they prescribe the water level during the storm. The water level rises linearly at the beginning of the storm from the daily lake level until the peak level minus 0.1 m is reached two hours before the halfway point, then to the peak level at the middle of the storm, then to the peak level minus 0.1 meter two hours after the halfway point and then back to the daily lake level at the end of the storm. The water level during the storm is shown in the figure below.



(a) Wave height per hour



(b) Setup level during a synthetic storm according to the WBI 2017

Figure 4.3: Setup level and wave height during a synthetic storm

The daily lake level was raised 0.30 meters for climate change until 2125. The daily lake level was assumed to increase equally and was + 0.1 m NAP. The peak of the wave height during the storm was 3 hours and has a trapezoid shape, with 0 wave height at t = 0 and t = 35 hours. For the input, the average wave height over an hour was used. Thus, for the first hour of waves the wave height at t = 0.5 hours was used and for the second hour of waves the wave height at t = 1.5 hours was used etc. A hard layer was modelled under the foreshore, which represents the current block revetment. Finally, wave spreading was turned off.

Step 3: Schematise the eroded profile from Xbeach and calculate failure probability for overtopping again in Riskeer

The profiles from Xbeach were schematised using the de Waal [40] and the failure probability was calculated in Riskeer using the same method as in step 1.

Step 4: Check if design points fit

This is a control step, if the design point in step 3 differs too much from the design point in step 1, another iteration was necessary. In the second iteration step 2 was repeated with the design point from step 3. Then step 3 is performed again. If the design point in step 3 differs too much from the design point from the previous step 3, another iteration was necessary. Iterations are performed until the design points do match. In this method it is assumed the critical design point from overtopping leads to the highest probability of failure. In Appendix A, a test was performed which suggested this was a valid assumption.

4.3. Results

In this section, the results of the simulations and calculations performed, following the steps explained in detail in the previous methods section, are shown.

Step 1

In step 1 the failure probability for wave overtopping was calculated in Riskeer. The table 4.3 shows the failure probability calculated in Riskeer for the three different foreshore heights. As aimed for, the failure probability is significantly lower than the safety standard of the dike, because it is likely the foreshore erodes in the next step.

Foreshore height	**Probability of failure**
1.75	1/466000
1.80	1/602000
1.85	1/771000

Table 4.3: Calculated failure probabilities of the sandy foreshore profiles

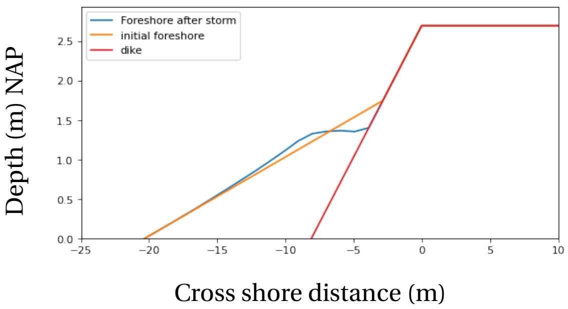
Step 2

In step 2 the design point offshore was calculated in Riskeer for each foreshore and afterwards dune erosion was simulated for the different foreshore heights. The design points calculated in Riskeer are shown in the table below. The maximum wave height was calculated for the water level, which was equal to the water level at the near shore design point, which was calculated in the step 1.

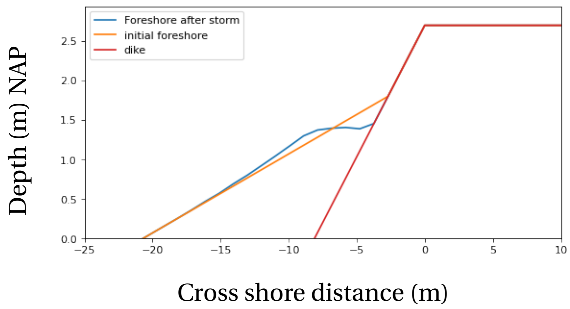
Foreshore height	Water level (m)	Wave height (m)	Peak wave period (s)
1.75	2.02	1.30	5.00
1.80	2.06	1.32	5.03
1.85	2.09	1.33	5.05

Table 4.4: Design points calculated in Riskeer

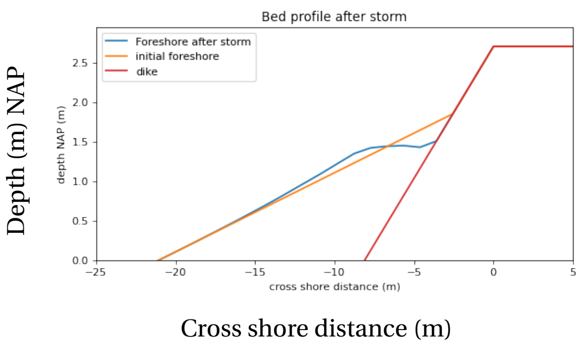
The figure 4.4, on the next page, shows the results of the simulation in Xbeach for the different foreshore heights before and after erosion. The inputs for the simulation are described in the methods section. During the peak of the storm, a platform with the length of approximately 5 meters is formed, 35 cm below the dike toe. All three simulations show a similar show a similar platform at 1.4, 1.45 and 1.5 meter NAP, respectively.



(a) Bed profile 1 after design storm



(b) Bed profile 2 after design storm

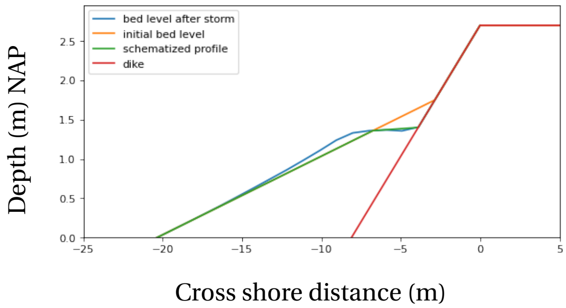


(c) Bed profile 3 after design storm

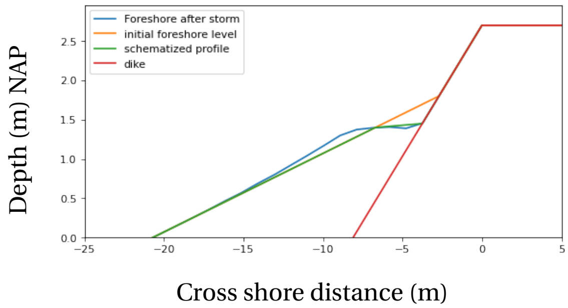
Figure 4.4: Bed levels after design storm simulated in Xbeach

Step 3

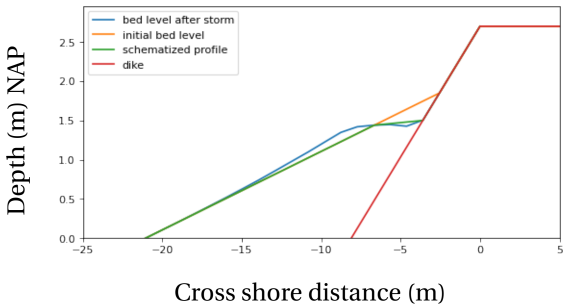
In the third step the eroded foreshore profiles were schematized, using the 'Schematiseringshandleiding hoogte' [40] and thereafter the failure probability for wave overtopping was calculated in Riskeer. The eroded profile had a slope steeper than 1/10, as a steeper slope than 1/10 is not allowed in Riskeer the maximum slope of 1/10 was used in these schematizations, (see chapter 3.1.2. The schematized profiles are shown in figure 4.5.



(a) Schematization of bed profile 1



(b) Schematization of bed profile 2



(c) Bed profile 3 after design storm

Figure 4.5: Figures with schematizations of the profiles simulated in Xbeach

The schematized profiles were calculated for wave overtopping and the probabilities of failure are shown in the table 4.5 below.

Foreshore height (m)	P_f before	P_f after
1.75	1/466000	1/88000
1.80	1/602000	1/112000
1.85	1/771000	1/140000

Table 4.5: Failure probabilities of the eroded profiles

Step 4

All foreshores had enough dune erosion to significantly lower the failure probability, therefore another iteration was deemed necessary. To begin the next iteration the design point from the failure probability of the eroded profiles was calculated in Riskeer and shown below.

Foreshore height (m)	Setup	Wave height (m)	Peak wave period (s)
1.75	1.76	1.19	4.80
1.80	1.79	1.20	4.83
1.85	1.83	1.22	4.85

Table 4.6: Design points calculated in Riskeer after erosion

The new design points from Riskeer were used to simulate the erosion of the foreshore in Xbeach, subsequently the eroded bed was schematized and calculated for overtopping in Riskeer. This resulted in the same failure probability, as was found in step 3. Thus no more iterations are needed as the design points from Riskeer and Xbeach are similar after one iteration. Table 4.7 shows the final failure probability for the different foreshore heights.

Foreshore height (m)	Final P_f
1.75	1/88000
1.80	1/112000
1.85	1/140000

Table 4.7: Final failure probabilities of the different foreshores, calculated in Riskeer

4.4. Conclusion

Combining Riskeer to calculate the probability of failure for overtopping and Xbeach to simulate dune erosion, was an effective way to approach the failure probability of a hybrid levee. When using this method it is important that both Riskeer and Xbeach have the same design points and eroded foreshore profile. It took multiple iterations to determine the failure probability. It is important what type of foreshore and hard layer combination is present. When the failure mechanism is wave overtopping, it is advised to model only the foreshore as sand and the dike as a hard structure. When determining the failure probability of a dune, a dune erosion calculation is advised over a wave overtopping calculation, to determine the probability of failure. It was found that the critical design point from wave overtopping led to the highest failure probability compared to other critical design storms.

5 Longshore Transport

5.1. Introduction

In this chapter a calculation was performed to predict the longshore sediment transport along the beach in Almere Duin. This calculation was performed using a wind rose, where for each cell of the rose the longshore transport was calculated and finally summed to find the average transport. To perform this calculation, the wind data from Schiphol was gathered and the occurrence of different wind directions was calculated. Then for each wind direction the flow velocity along the beach was calculated using a Delft3D model of the Markermeer. This data was combined to find the occurrence of different flow velocity along the beach. Where after with the van Rijn transport formula the gross north bound and south bound transport was calculated. This calculation is helpful to produce a maintenance strategy and to examine whether a slim foreshore or a stronger foreshore is a design optimal.

5.2. Method

Wind Data

First the wind data of the nearest weather station (Schiphol, about 25 km) was examined to find the occurrence of different wind direction and wind velocities. Then the wind data of Schiphol from 1951 - 2022 was downloaded from the KNMI website, where the hourly average wind data was available from 1971 to April 2022. Subsequently a wind rose was constructed with 16 wind direction and four layers. In each cell the vector of the occurrence of the 16 different wind directions and wind speeds from 0 -5 m/s, 5 - 10 m/s, 10 - 15 m/s and 15 - 20 m/s are presented (shown in figure 5.1). Hourly average wind speeds of more than 20 m/s (which only occurred 0.02%) were not included.

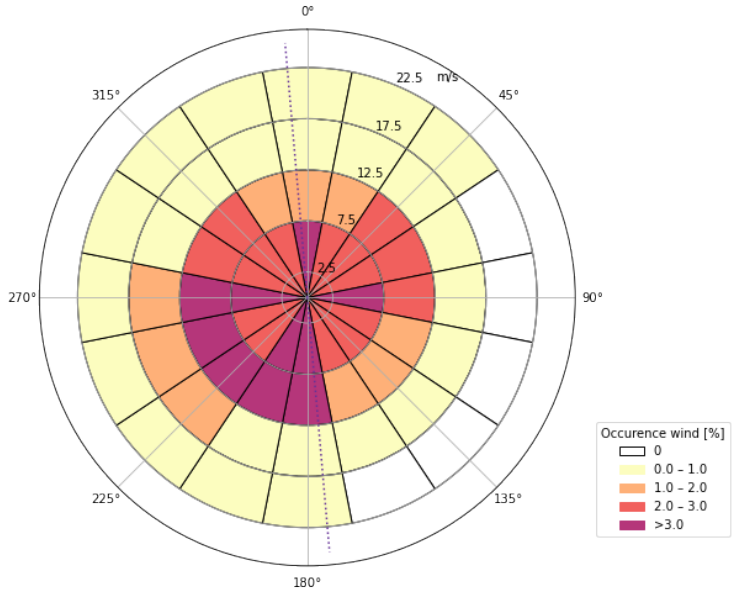


Figure 5.1: Occurrence of wind directions and wind speeds at Schiphol

Flow

Next the flow velocity for each vector in the rose was calculated. To perform this calculation, the flow direction and speed were retrieved from the Delft3D model of Ton et al.. The rose with the flow vectors is presented in figure 5.3a.

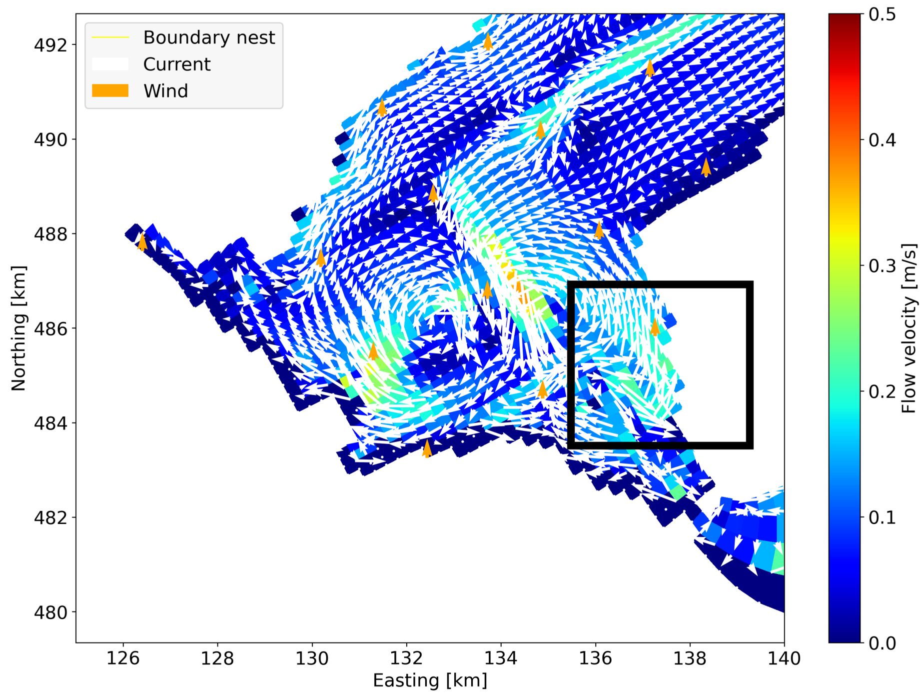
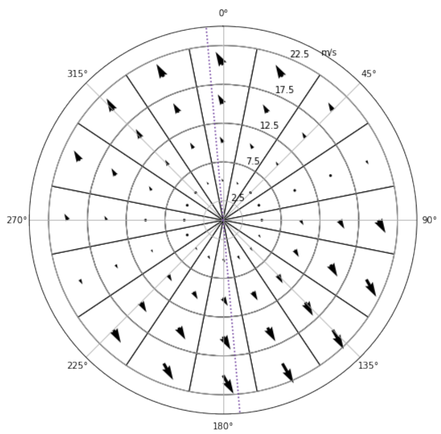
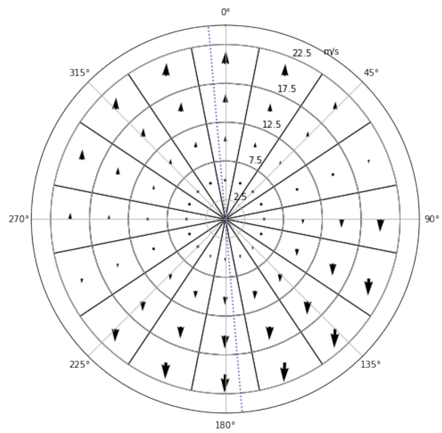


Figure 5.2: Flow velocities and direction of Delft3D model for wind from the south. (Black square illustrates location of Almere Duin)

Delft3D calculates the flow velocity vector for squares of the size of 250 meters. The beach is 1.25 km long so multiple squares are offshore off the beach. The variability of the direction and velocity of the flow is high, as seen in the figure 5.2 above. The grid cell which lead to the greatest brute longshore transport was assumed to be critical and chosen for the calculation. Subsequently, the alongshore component of the flow vector was calculated, shown in figure 5.3b. The u component was positive in the south direction and the v component was positive in the west direction. The normal of the coastline was 265 degrees, thus the alongshore component was calculated with: - u * cos(5) v * cos(85) and was chosen positive in southbound direction.



(a) Vector of the flow velocity, for each wind direction and wind speed



(b) Alongshore component of the vector of the flow velocity, for each wind direction and wind speed

Figure 5.3

Finally, the flow vectors were multiplied by the occurrence of the wind for each wind velocity and wind direction to find the occurrence of the alongshore flow, shown in figure 5.4.

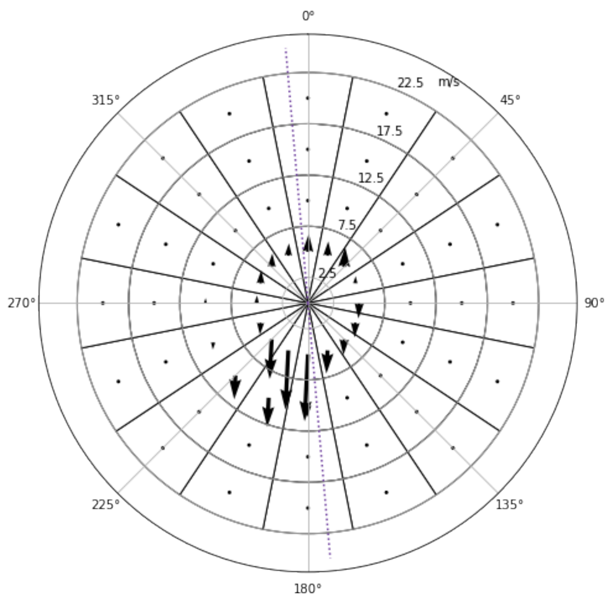


Figure 5.4: Alongshore component of the vector of the occurrence of the flow velocity, for each wind direction and wind speed

Longshore Transport

the CROSMOR LST formula was calibrated for higher energy coastal conditions, which have been demonstrated to

The Longshore transport rate was calculated with the Van Rijn 2014 CROSMOR formula. The CROSMOR LST formula is assumed to be correct, because a formula for lower energy conditions was not found in the literature. Even tough it was found in the literature that the formula could differ by 1 or 2 orders of magnitude.[3] The CROSMOR formula was proposed by Van Rijn [36] and is a process based transport formula which establishes a relationship between wave height, alongshore flow velocity, particle size, beach slope and longshore transport:

$$Q_{t,mass} = 0.006 K_{swell} \rho_s (tan_\beta)^{0.4} (d_{50})^{0.6} H_{s,br}^{2.6} V_{wave} \tag{5.1}$$

Where Q_t is the alongshore sand transport (in kg/s; dry mass). K_{swell} is a swell correction factor, because of the absence of swell waves on the Markermeer, $K_{swell} = 1$. ρ_s is the sediment density, $\tan_\beta$ is the slope of the beach or surf zone. $H_{s,br}$ is the significant wave height at the breakerline, which was calculated with the Bretschneider formula. Finally, V_{wave} is the wave-induced longshore current velocity (m/s), averaged over the cross-section of the surf zone. Additional flow velocities in the surf zone due to tide and wind could be included by adding them to the V_{wave}. More information about the formula can be found in the literature 2.6.

The effective alongshore velocity at mid surf zone (m/s) for flow velocity, which was calculated with the Delft3D model and the occurrence of the wind direction and speed, was explained in the paragraph above. Formula (5.1) was used to calculate the transport for each combination of wind direction and velocity and the transport vectors are presented in figure 5.5. Finally, the LST for each negative cell and positive cell was summed to find the bulk southbound and northbound bulk alongshore transport and those were deducted to find the net transport.

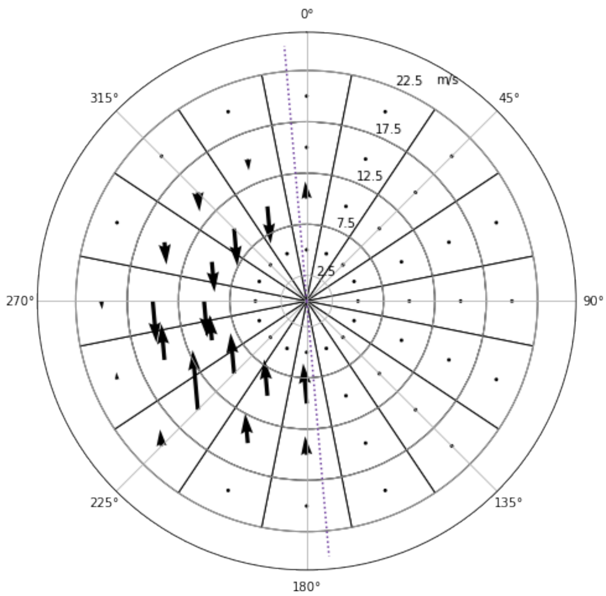


Figure 5.5: Calculated occurrence of transport, for each wind direction and wind speed

5.3. Result

The CROSMOR2013 transport formula gives a dry bulk transport in kg/s. To find the yearly transport the results was multiplied by 3600 seconds, 24 hours and 365 days. Then to find the volume of transport the outcome was divided by the density of sand, 2650 kg/m^3. The density of fresh water is 1000 kg/m^3 and for offshore winds the wave height was set to zero. The result is then multiplied by the dry weight conversion factor of 1.61 to find a yearly longshore transport of 929 m^3/y southbound, 1246 m^3/y northbound and a net transport of 317 m^3/y northbound. [41]

$$Q_{t,mass} = \frac{\rho_s}{\rho_s - \rho} * Q_{t,immersedmass} \tag{5.2}$$

$$\frac{\rho_s}{\rho_s - \rho} = \frac{2650}{2650 - 1000} = 1.61 \tag{5.3}$$

Uncertainty in Transport

With the same method as outlined in section 5.2, the net yearly longshore transport was also calculated for each of the 51 years between 1971 and 2021. The average yearly transport was found to be 321 m^3/y northbound with a standard deviation of 326 m^3/y. The transport for each year from 1951 to 2021 is plotted in a histogram in figure 5.6 below and this shows the yearly transport uncertainty matches a normal distribution. The goodness of fit of some distributions was examined, resulting in a best fit for the normal distribution. The goodness of fit was tested with the Kolmogorov-Smirnov test, which suggested it had a 84% probability the yearly longshore transport was drawn from the fitted normal distribution.

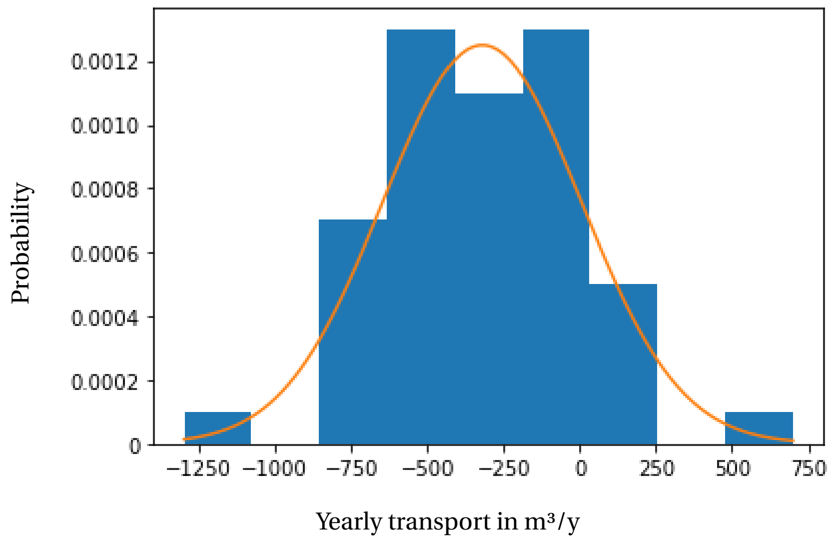


Figure 5.6: Net yearly transport over the period 1971-2021 with fitted normal distribution

5.4. Conclusion

The aim of this chapter was to find the order of magnitude of transport, which can be expected at Almere Duin. The amount of transport found was 321 m^3/y northbound with a standard deviation of 326 m^3/y. The uncertainty of the transport is very high due to the flow close to the beach turning in the Delft3D model.

Using wind to predict flow promises to be a good tool because the wind data is better accessible than wave data. However, a calibrated model or flow measurements are necessary for this method. If a Delft3D method is available this is a good option. If this is not the case, than flow measurements need to be performed which would take approximately the same time and costs as performing wave measurements. The underestimation of transport by the CROSMOR2013 LST formula of 1 or 2 orders of magnitude [3], leads to a large uncertainty in the prediction, which is common in coastal morphology.

6

Life Cycle Costs

6.1. Introduction

In this chapter, the life cycle costs of different maintenance strategies are calculated and compared. Almere Duin functions as a good case study to calculate the life cycle costs, six different maintenance interval periods of 1, 4, 10, 25, 50 and 100 year, represented the different maintenance strategies. To calculate the life cycle costs the alongshore transport, as estimated in chapter 5, was used. Together with cost estimates the net present value was calculated to determine the life cycle costs for each maintenance strategy.

6.2. Method

The life cycle costs were calculated of six different maintenance strategies. The different maintenance strategies have an interval period between nourishments of 1, 4, 10, 25 and 50 and 100 years. The strategy with a 100 year maintenance interval has no maintenance during its lifetime, because the design lifetime is 100 years.

The alongshore sediment transport is assumed to lead to erosion at the northern part of the beach and sediment accretion at the southern part of the beach, shown in figure 6.1. Currently no beach is present at the location, so it was assumed the beach is to be constructed in between two hard structures. Even though all sediment is likely to remain in the system, longshore transport will transport the grains from the south side to the north side of the beach. Because there is no continuous supply of sand, due to two hard structures on both sides, this leads to changes in the cross shore profile, which in turn leads to erosion on one side and accretion on the other side.

To find out how much sand would be eroded per meter of sandy foreshore, the length over which the beach erodes and the shape of the erosion needs to be known. There was no good method available to predict these parameters, therefore the length was estimated to be 100 meters and the erosion was assumed to be constant over this length. The conclusion in chapter 2.6 was that the estimated net yearly longshore transport was equal to 317 m^3/y and directed to the north and this resulted in a loss of 3.2 $m^3/m/y$.

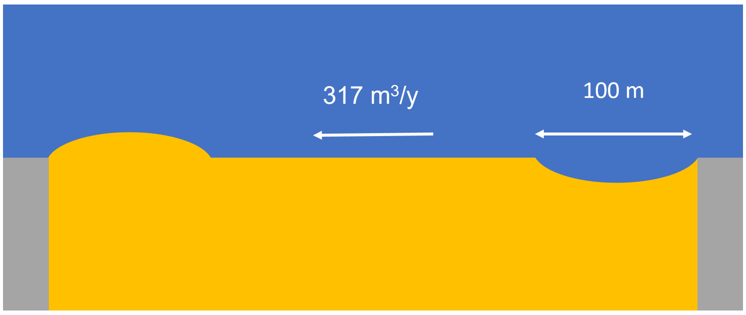


Figure 6.1: Alongshore transport at Almere Duin

Foreshore Components

In this thesis, the foreshore consists out of three components. The most nearshore part of the foreshore, is the safety profile, which is the minimal profile necessary to meet the required safety standard. The second piece of the foreshore is the maintenance volume, which is the part of the foreshore which erodes under daily conditions. Lastly, below the maintenance volume and the bottom of the lake, a fill volume is required. Figure 6.2 presents the cross section showing the different layers. It is possible the average lake level rises 30cm over the next 100 years, this water level rise was anticipated for in the calculation of the failure probability, but was not taken into account in the LCC in this chapter. Because this would make the calculation excessively complicated and is not expected to lead to significantly different results.

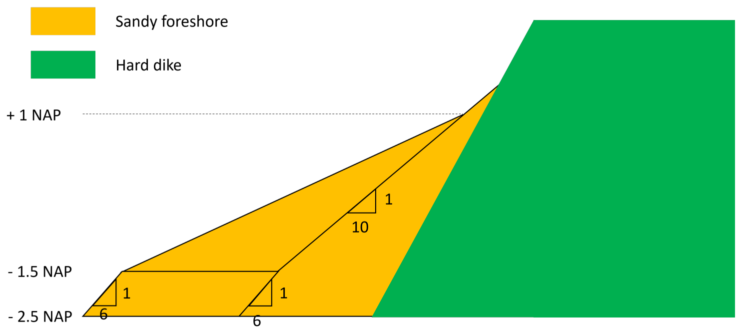


Figure 6.2: Cross section of the beach

Volume

To be able to calculate the volume of the foreshore components, the boundaries of the profiles need to be known. The foreshore height of the safety profile was calculated in chapter 4 and is equal to 1.85 m and the bottom of the lake is assumed to be flat. Figure 6.3 shows the depth of the lake parallel to the shore, 100 meters offshore. In onshore and offshore direction, the depth does not change much. More information about the bathymetry can be found in 1.8 and is about 2.5 meters below NAP. The safety profile has a slope of 1:10 until the offshore boundary of the surf zone. Ton et al. [35] found that the active surf zone of beaches at the Markermeer was between +0.95 m NAP and -1.55 m NAP. It is assumed the slope outside the surf zone can be equal to the internal angle of friction, which is 15-30 degrees for sand under water [42], thus a 1:6 slope is assumed. Next, the maintenance volume was calculated by multiplying the yearly erosion by the interval period between maintenance. The maintenance volume attaches offshore to the safety profile and is vertically restricted by the limits of the surf zone, thus the maintenance buffer is located in-between -1.5 and +1 NAP. Possible settlements were considered out of the scope for the calculation of the volumes.

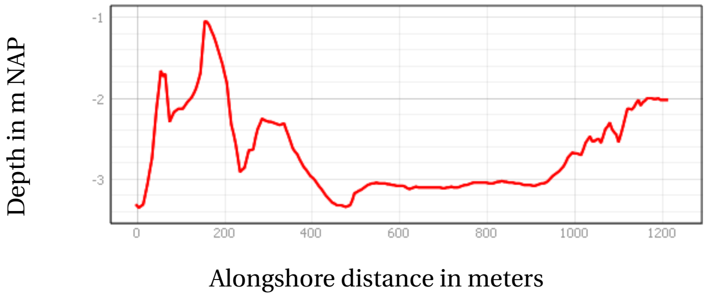


Figure 6.3: Depth along the beach 100 meters offshore

Formula (6.1) was used to calculate the volume of the foreshore, where d is the depth of the lake, h_F is the height of the foreshore, α_F is the slope of the foreshore, V_{dike} is the volume of the dike under the foreshore in m^3/m, and α_O is the of slope offshore of the surf zone. All of the volumes of the foreshore components are calculated per unit of width.

$$V_{Safe} = (\frac{(d+h_F)^2}{\alpha_F * 2} - V_{dike} - \frac{\alpha_F^{-1} - \alpha_O^{-1}}{2} \tag{6.1}$$

To calculate the maintenance buffer formula (6.2) was used. Where V_{Main} is the volume of the maintenance buffer in m^3/m, T_M is the time period in-between the nourishments in years, $V_{erosion}$ is the yearly erosion rate calculated in chapter 2.6. To calculate the fill layer under the maintenance layer, formula (6.3) was used. When all volumes are known, the total volume of the foreshore can be calculated with formula (6.4).

$$V_{Main} = \frac{T_M * V_{erosion}}{L_{erosion}} \tag{6.2}$$

$$V_{Fill} = \frac{V_{Main}}{1.25} \tag{6.3}$$

$$V_{Tot} = V_{Safe} + V_{Main} + V_{fill} \tag{6.4}$$

The cost of each nourishment is calculated by multiplying the maintenance buffer by the cost of sand per cubic meter, as shown in formula (6.5). L is the length of the dike section.

$$Cost = V_{tot} * C_{Sand} * L \tag{6.5}$$

Costs

To calculate the Life Cycle Costs, the bulk price of sand per cubic meter and the discount rate need to be known. The costs of nourishments strongly depend on the method with which the sand will be brought on the beach and the distance the sand has to travel. The construction of a sandy foreshore uses bigger volumes and is more likely to be allowed a sand fill nearby, compared to a nourishment. This could reduce the costs per cubic meter to 3 €/m³. [43]

The investment costs (C_I) consits out of the costs for the construction of the beach, which were calculated by multiplying the volume of the beach by the costs per volume, shown in formula 6.6. Where V_{Fill} is the volume of the beach and C_{Sand} is the cost of sand per cubic meter in Euros, L is the length of the beach, which is 1.255 km and $L_{erosion}$ is the length over which erosion occurs.

$$C_I = (V_{Safe} * L + (V_{Main} + V_{Fill}) * L_{erosion}) * C_{Sand} \tag{6.6}$$

During maintenance of the beach, the volumes which will be applied to the beach are significantly smaller. Nourishments for maintenance are in the order of 20 - 300 m³/m, making the costs per cubic meter significantly higher. For small volumes lots of different equipment needs to be mobilized and sand might need to be shipped by barge from further away. This pushes the cost up to 15 - 20 €/m³ or even more. In theory the sand which has been carried away by the waves to sediment traps or dams at the edge of the beach could be dredged and sprayed back on the eroded shore of the beach. However, whether this is possible, varies per case, due to nature developments and strict dredging regulations. Therefore, in this thesis, the LCC was calculated with 20 €/m³ for nourishments [43].

Due to the small maintenance volumes, it was assumed that the cost per cubic meter of sand is equal for all nourishment sizes. Only for each nourishment the costs of mobilization are used. The mobilization costs include the cost to mobilize the equipment, to execute the nourishment and also other costs e.g. writing the contractual documents, monitoring the beach development and preparing construction.

The cost per nourishment can now be calculated with formula (6.7), where C_N, is the cost of the nourishment, V_{Main} is the maintenance volume, C_{Sand}, is the cost of sand in €/m³, $L_{erosion}$ is the length over which erosion occurs and C_{Mob} are the mobilization costs.

$$C_N = V_{Main} * C_{Sand} * L_{erosion} + C_{Mob} \tag{6.7}$$

According to the 'Factsheet Discontovoet' [44], LCC analyses, which are used for cost benefit analyses and have only expenses, were advised to use a discount rate of 1.6%. This is for both initial costs of construction of infrastructure and for management and maintenance, of which the costs are inevitable and sunk. There is no exception for smaller projects.

Present Value of Total Costs

Lastly, the total costs of the complete life cycle were calculated. The calculations are presented in formula (6.8) , where $PV_{Maintenance}$ is the present value of the maintenance cost, $C_{Nourishment}$ is the cost per nourishment in Euros, r is the discount rate, T_m is the maintenance interval period and T_L is the lifetime of the project.

$$PV_{Main} = \sum_{n=1}^{T_L/T_M - T_M} \frac{C_N}{(1+r)^{n*T_M}} \tag{6.8}$$

The total life cycle costs are calculated by combining the investment costs (C_I) and the maintenance costs (C_{Main}). (6.9).

$$C_{Tot} = C_I + C_{Main} \tag{6.9}$$

6.3. Results

In table 6.1 below a summary of all the input parameters is given. The beach will need to satisfy the safety limit until 2125, without having to reinforce the dike, as was mentioned before in the background 1.8, this is about a hundred years from now. Therefore, the lifetime of the construction was set at 100 years.

Parameter	**Value**
Lifetime	100 years
Depth	-2.5 m NAP
Longshore Transport	317 $m^3//y$
Length of loss	100 m
Cost sand construction	3 €/m^3
Cost of beach nourishments	20 €/m^3
Length	1.255 km
Cost of Mobilization	€10000
Length	1.255 km
Discount rate	1.6%

Table 6.1: Input parameters for LCC

The table below presents the results of the LCC for each maintenance strategy. The costs are presented in millions of Euros. The Lowest LCC was for the maintenance strategy with no nourishments and the highest for the maintenance strategy with a 1 year maintenance interval. This data suggests that the longer the maintenance period is, the lower the LCC will be. However if a longer lifetime is considered (illustrated in image), it was found that a 100 year maintenance interval was optimal for this specific case study.

Maintenance interval period	**1**	**4**	**10**	**25**	**50**	**100**
Initial costs (10^3 **€**)	140	140	150	180	220	310
Size of Nourishments (**10^3 m^3**)	0.3	1.2	3.2	7.9	15.9	31.7
Costs of Nourishments (10^3 **€**)	16.3	35.4	73.4	168.5	327	644
Costs of maintenance (10^3 **€**)	809	422	324	241	148	0
Life Cycle Costs (10^3 **€**)	946	564	476	418	368	310

Table 6.2: Results of the LCC

Figure 6.4 presents the total life cycle costs for maintenance intervals ranging from 1 year to 100 years. The result suggests the longest possible maintenance interval would be optimal, which for the case study of Almere Duin is a maintenance interval period of 100 years. Thus, for this specific case study, a strategy without any maintenance was found optimal.

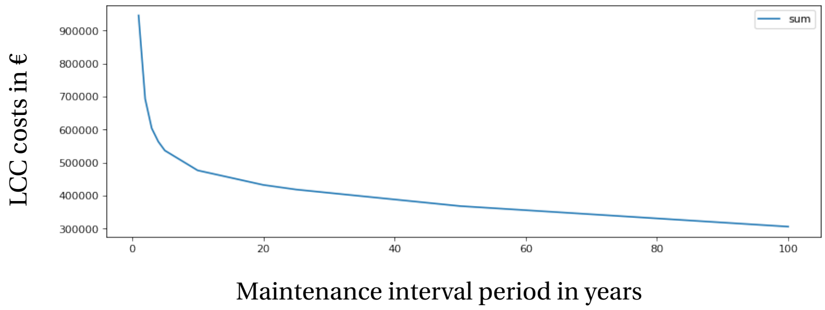


Figure 6.4: Total life cycle costs for different maintenance intervals, from yearly to 100 year interval periods

However, an optimal maintenance interval period was observed, when for example, a longer design lifetime was chosen. This is illustrated in figure 6.5. An optimal maintenance interval period was also observed, for a higher interest rate or for a lower price of sand, per maintenance volume.

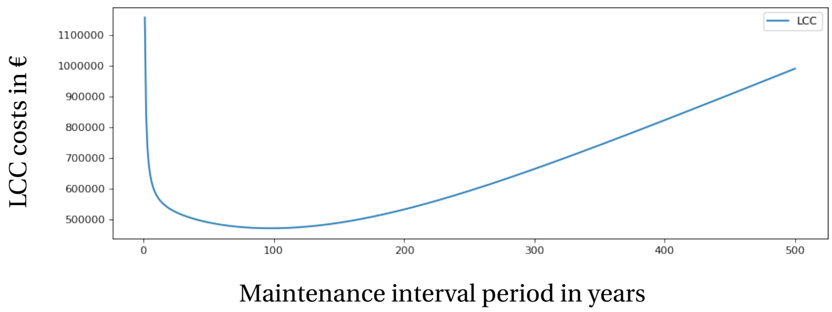


Figure 6.5: Total life cycle costs for different maintenance intervals, from yearly to 500 year interval periods, with a lifetime of 500 years

6.4. Monte Carlo Simulation

In this section, the influence of a Monte Carlo Simulation on the results of the LCC is examined. The formulas used for the Monte Carlo simulation are the same formulas as described in section 6.2 and in total 10000 simulations were executed. Only the physical parameters were probabilistic and their uncertainties are presented in table 6.3. The yearly alongshore transport has a mean of 321 m^3/y and a standard deviation of 326 m^3/y (which was predicted in chapter5.3). The length over which the beach erodes is assumed to be lognormally distributed with a mean of 100 meters and a standard deviation of 50 meters. The CROSMOR2014 LST formula overestimates storm conditions by a factor of 1.5 and underestimates low wave conditions by a factor of 2. Therefore the yearly erosion rate is multiplied by the model uncertainty of the transport formula, which had a lognormal distribution with a coefficient of variation of 1 and a coefficient of variation of 100% [36].

Parameter	Distribution	μ	σ	Coefficient of variation
Yearly alongshore transport	Normal	321	326	X
Model uncertainty transport formula	Lognormal	1	X	1.0

Table 6.3: Distribution of the probabilistic input parameter

Results

The resulting mean total life cycle costs of the six different maintenance strategies and 10% and 90% probability values are presented in table 6.4. The life cycle costs appear to be 21% to 34% higher with the Monte Carlo simulation, nevertheless the ranking, from lowest to highest LCC of the strategies, does not appear to change.

Maintenance Interval	Deterministic LCC (10^3)	Mean Probabilistic LCC (10^3)	10% Probability Value (10^3)	90% Probability Value (10^3)
1 year	856	1144	950	1350
4 years	467	676	571	789
10 years	369	576	487	622
25 years	302	522	444	606
50 years	271	489	433	556
100 years	310	483	445	537

Table 6.4: Life cycle costs of different maintenance strategies, calculated deterministic compared to life cycle costs calculated probabilistic and their 10% and 90% probability value

To quantify the uncertainty, also a Monte Carlo simulation was performed. In figure 6.6, for each maintenance interval, the calculated LCC is plotted without any uncertainty, with only the uncertainty of the LST rate, with only the model uncertainty and with both uncertainties.

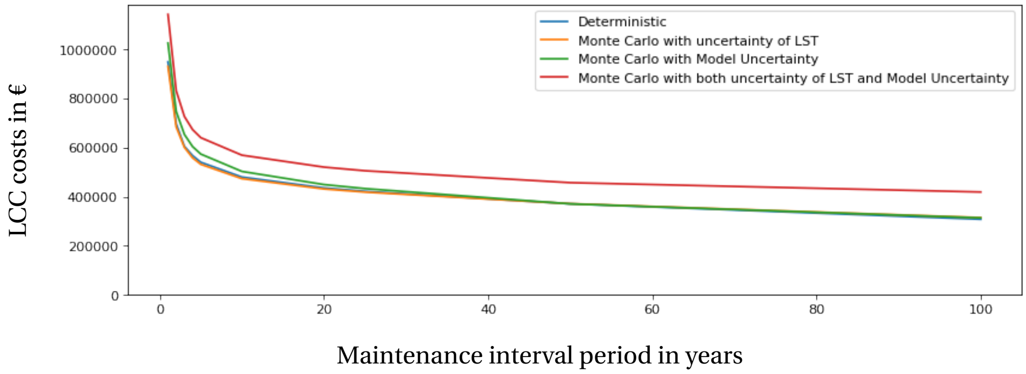


Figure 6.6: Total life cycle costs for different uncertainties, from yearly to 100 year interval periods

Figure 6.7 illustrates the resulting cumulative density function of the six maintenance strategies, with both the model uncertainty and the uncertainty of the erosion rate.

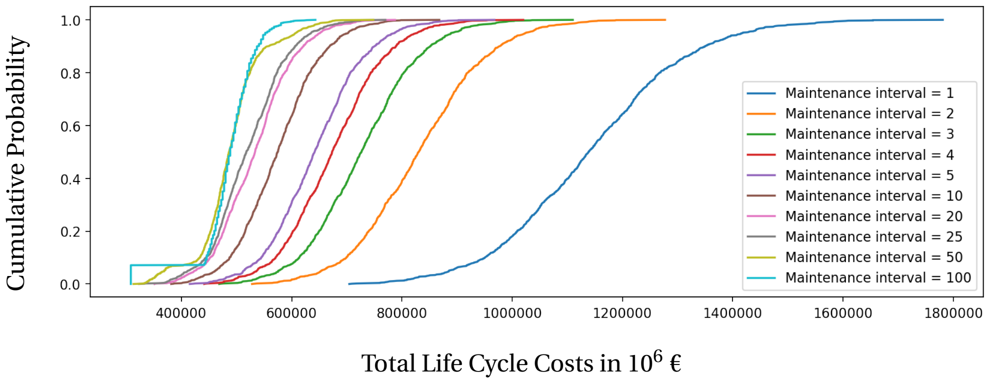
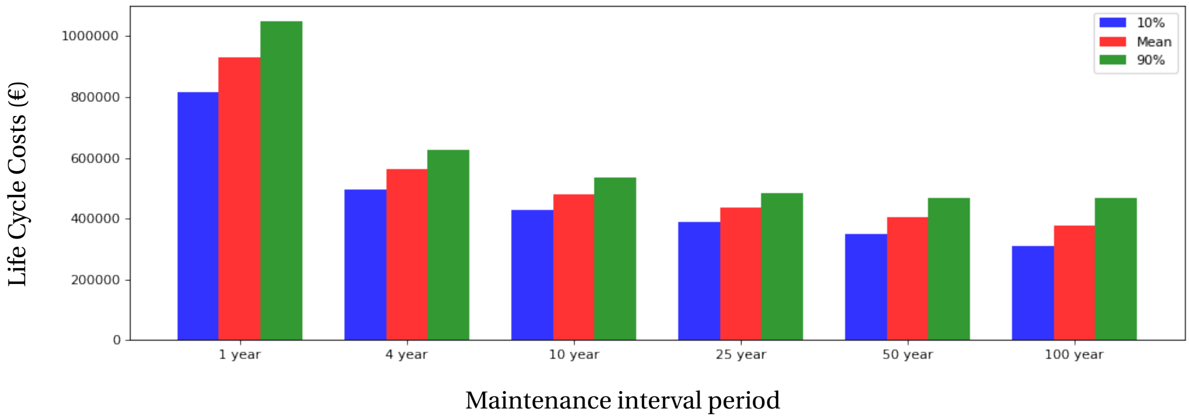
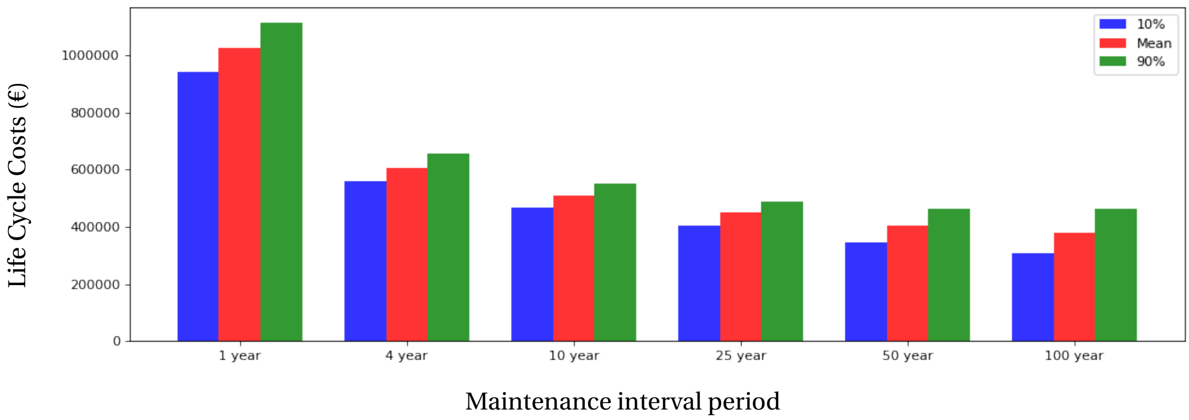


Figure 6.7: Cumulative density function of different maintenance strategies

A Sensitivity analysis was carried out and the resulting sensitivity for the different maintenance strategies is presented in the in figure 6.8 below. For each variable two calculations are made, one with the input variable set that it matches the exceeding probability and one that it matches the non-exceeding probability. All other variables are set to their expectation value. The blue bars illustrate the result for a 10% probability value and the green bars illustrate the result for a 90% probability value. It was found that the maintenance strategies with a higher maintenance interval had a higher relative uncertainty but a similar absolute uncertainty.



(a) Sensitivity of the LST rate



(b) Sensitivity of the Model Uncertainty

Figure 6.8: Sensitivity analysis of the yearly erosion and model uncertainty for each maintenance strategy

Monitoring

Because sandy foreshore in low-energy non-tidal environments are relatively new, not much is known about their behaviour in the long term. A monitoring project could be an import aspect for maintenance and could also help to gain knowledge about the behaviour of sandy foreshores in the long term.

To track the development of the sandy foreshore, one could use a method comparable to the method used for following the developments of the Dutch coastline could be used. Such a method could reduce the uncertainty of the erosion of the beach and help develop a better maintenance strategy. However, the monitoring campaign comes at a price. According to the E et al. [43], monitoring would cost around €5000 per km of shoreline. The Almere Duin beach has a length of around 1.25 km, which would result in a yearly expenditure of €6250 for the monitoring. The predicted yearly erosion of the beach is around 317 m^3 per year. The most expensive price per cubic meter of sand, for a small project, is €20 per cubic meter, which would result in a total loss of value of €6340 Euros per year. Thus making the monitoring more expensive than the erosion, therefore monitoring was not considered. However, it is very much possible that for beaches with a greater amount of yearly erosion, monitoring could lead to a more optimal strategy.

6.5. Conclusion

The aim of this chapter was to develop a method to optimally calculate the life cycle costs of a sandy foreshore. The life cycle costs of six maintenance strategies, with a maintenance interval period of 1, 4, 10, 25, 50 and 100 years, were calculated together with the uncertainty and sensitivity of the life cycle costs.

The life cycle costs of different maintenance strategies were calculated and it was observed that optimizing the maintenance strategy lowers the life cycle costs. For this specific case, it was found that the largest maintenance buffer was optimal. Therefore it is concluded that assessing different maintenance strategies is important, to minimize the life cycle costs.

Lastly, a Monte Carlo simulation was performed, which did not lead to a different optimal maintenance strategy, however the probabilistic calculation did find higher life cycle costs of 20% to 34%. The model uncertainty was found to be slightly more influential than the yearly erosion rate and the length of the erosion, was found to have no influence. The uncertainty of the strategies was found to be between 9% and 27 % and to reduce the uncertainty it is suggested to nest the Delft3D simulation and calibrate the LST formula for low-energy non-tidal lake environments.

7

Discussion and Recommendations

In this chapter each of the four tracks is discussed followed by a general discussion.

Chapter 3

An assessment of the influence of a foreshore on the failure probability of a dike in a low-energy non-tidal shallow lake environment was made. These results help to optimize the design for a hybrid levee with a foreshore because this gives an insight into the failure probability for different combinations of foreshore heights and slopes. A limitation was that no calculation without a foreshore was performed to find the effect of different foreshores on the reduction of the failure probability. This was however not necessary for the research scope of this thesis. Previous studies such as Verheij et al. [23], found the foreshore should reduce the necessary crest height by 0.5 to 1.5 meters and Oosterlo [7] who found a reduction factor in the probability of failure between 30 and 1000. Despite this, these findings were expected and reinforce the general consensus that the foreshore reduces the hydraulic load on levees and reduces the probability of failure.

These findings also help to determine which parameters are important for dune erosion. This helps to calculate the failure probability of a hybrid levee in chapter 4. There were no unexpected results. It was found that the wave period, wave height and setup were of more importance than the grain size and storm duration. These findings also help to determine what influence different combinations of hard layers and foreshores have on dune erosion. It was found that different foreshore heights influence dune erosion and different hard layers also influence dune erosion. It is important to know the effect of different hard layers on erosion is to determine what failure mechanisms can be used to determine the failure probability.

Chapter 4

In this study corresponding design points and bed level inputs were used to use Riskeer and Xbeach in parallel. This worked when the setup was higher than the foreshore height. Otherwise, the foreshore loses its wave breaking function and the failure mechanism shifts to dune erosion. It was found that the dike must be hard, for the reason that the sand from the dike slope could not be eroded onto the foreshore. If the dike would be modelled as soft, the failure probability would lower significantly, due to the cross shore processes bringing the sand from the dune onto the foreshore, and thereby significantly heightening the foreshore. This is problematic, because this would change the scope of the thesis. From approaching the failure probability, to analyzing the residual strength. Furthermore, it would be easier to shift the failure mechanisms to dune erosion, and assess the strength of the dike and foreshore combination as a dune.

Not much research on overtopping of dunes could be found, which makes sense as dunes are often very wide and normally nobody walks over them during storms. Due to the limit of <0.1 $l/m/s$ of wave overtopping set by the city council, an overtopping calculation for flood defence remained necessary. It was found that a wave overtopping calculation can be performed with a hard dune and a soft sandy foreshore or small dune to limit the reduction of the failure probability . The former option was chosen because it was previously found that this led to lowering of the foreshore height. Nevertheless it is advised to use the failure mechanisms of dune erosion instead of wave overtopping for determining the failure probability of a dune.

Using Riskeer and Xbeach in parallel took a while to master, but was easy once acquainted with the software. The calculation times were low compared to other methods [45] [8]. The Riskeer calculation time was 2 minutes and Xbeach calculation time was 25 minutes. It was convenient that probabilistic calculations were already integrated in the Riskeer software. All together, Riskeer plus Xbeach was a good method to approach failure probability.

It was found to be inconvenient that Riskeer only presents the onshore design point resulting in an extra step to find the offshore design point. The offshore design point is known to Riskeer and can be found if looked for carefully in the program but a good understanding of the software is necessary to perform this task. Therefore, it is advised to add an offshore design point to the Riskeer GEKB mode.

Although refraction over the foreshore was calculated by the DaF module of Riskeer, it would still be interesting to see if oblique waves would cause the erosion of the foreshore to be significantly different. It would have been possible to research the difference in erosion of the foreshore with 2D Xbeach. However in this research only 1D simulations were performed due to the significant extra calculation time of 2D Xbeach.

In future research more parameters, foreshore heights and slopes could have been tested. Also it would be interesting to perform the same analysis with 2D Xbeach instead of 1D Xbeach. This would allow for Xbeach to simulate alongshore transport and oblique waves, and could lead to different results, especially for this case study, where most waves arrive at a 65 degree angle.

These findings are similar to Oosterlo (2015) [45], who found similar little influence of grain size and storm duration, and larger influence of offshore conditions such as setup, wave height and wave period [45].

Chapter 5

The aim of this chapter was to find the order of magnitude of longshore transport which could be expected at Almere Duin. No unexpected results were found, except the uncertainty of the alongshore component from the Delft3D model was larger then anticipated. This method needs more research because it is likely this could be useful to predict longshore transport, when limited data is available. It was very helpful that a Delft3D model was already made, otherwise it would have been a very time consuming method to model the Markermeer in Delft3D.

There was no data available to validate this method for the Almere Duin case study. In the literature, it was found that LST formulas for coastal areas could yield results with a difference of order of magnitude of 1 to 2. However, no LST formula for lake environments was found in the literature. Thus, to predict the transport rate at Almere the method applied was correct, as only wind data was available. The predicted transport rate could be more precise when data of sand traps and flow measurements would be collected. It is advised to validate and calibrate the CROSMOR2013-formula for non-tidal low-energy shallow lake environments.

Multiple longshore transport formulas, including the CROSMOR2014 formula, which was used in this thesis, do allow a flow velocity to be included in the formula. However, there was no longshore transport formula found in the literature which has an alongshore flow parameter which is calibrated by alongshore flow induced by wind, only longshore transport induced by either waves or tide was found. Jackson found the Bagnold LST formula to be optimal for a fetch-limited case, if there was more time available, it would have been interesting to compare the results of the Van Rijn LST formula to the Bagnold formula [37].

There were multiple grid cells, with a high variability of the direction and velocity of the flow, representative for Almere Duin in the Delft3D model. The grid cell which led to the greatest overall longshore transport was assumed to be critical, for more detail and less uncertainty. It is advised for further research to nest the Delft3D model.

Chapter 6

These findings help to minimize life cycle costs of a sandy foreshore. The assumptions made were good enough to see good results. It was found different foreshore options have different life cycle costs, and the percentage of maintenance costs of total costs range from 0% to 85% . The outcome depends on what maintenance strategy is used. This suggests a large range of maintenance strategies could be relevant and thus should be considered when optimizing LCC.

A Monte Carlo simulation was performed, which did not lead to a different optimal maintenance strategy, it also did not change the order of the LCC of the maintenance strategies. This could have been expected as the probabilistic input parameters were similar for each maintenance strategy. However, the probabilistic calculation did find a 20% to 34% increase of the expected life cycle costs.

In the Monte Carlo simulation, a similar random sample of the yearly erosion was used, for each year in the lifetime of the structure. In stead of using a different random sample per year. The use of a different random sample per year might have led to a different uncertainty, which probably would have been smaller. Therefore, it is advised for further research, to compute a different random sample each year.

The model uncertainty of the LST formula and yearly erosion rate were found to be very significant. The CROSMOR2013-formula model uncertainty can be reduced by having more data, especially data in non-tidal shallow lake environments is missing.

General Discussion

This research could not only be used to calculate the probability of failure for wave overtopping, of a sandy foreshore in a non-tidal low-energy shallow environment. It also could be used the predict the longshore transport when only wind data is available, and be used to calculate the life cycle costs of a sandy foreshore.

The maintenance strategy can be optimized by calculating the life cycle cost of different maintenance strategies. These strategies should range from low maintenance to high maintenance. Boundary conditions such as longshore transport and the length over which the beach erodes, will be critical to determine the outcome. However, uncertainty of the models used should be reduced, to gain a clearer insight in which strategy is optimal.

As discussed in chapter 7 the time varying protection level could be approached by using a combination of Riskeer and Xbeach when the design points and schematization of the foreshore profile are identical.

It might be possible to calculate the failure probability with one software. This could be done by adding a parametric extension to Riskeer, which would include the reduction of the foreshore height in Xbeach for different combinations of wave height and wave period.

The foreshore profiles used to calculate the failure probability and subsequently the life cycle cost could probably have had an even steeper slope. Unfortunately, Riskeer only allows a maximum slope of 1:10, because a foreshore is defined to have a maximum slope of 1:10. However this definition originates from studying coasts where slopes are generally gentler than in lake environments

In earlier attempts, a slope was chosen which was very stable, because a maintenance buffer was already present. This brought about a few problems, thereupon it is advised to first calculate a minimum profile, and then add different maintenance buffers to this profile. This lead to a better method, where the calculations, to find the the minimum profile and the life cycle costs, were separated. Moreover, this made the option available to choose the length of the maintenance interval periods.

8

Conclusion

In this thesis sandy foreshores in non-tidal low-energy shallow lake environments were studied.

First the sub questions and then the research question are answered.

Sub-Question 1

- What is the influence of different foreshore heights and slopes on wave overtopping and how do different foreshores and parameters influence dune erosion?

From calculating the failure probability of different slopes and foreshore heights in Riskeer, the following conclusions can be made: When the allowed overtopping discharge increases by a factor of 10, the failure probability decrease by a factor of 3, and increasing the foreshore height by 30cm decreases the failure probability by a factor of 3 to 5. To decrease the failure probability, increasing the foreshore height is more effective than decreasing the slope of the foreshore, per cubic meter of sand required for the construction.

A sensitivity analysis of different parameters, foreshore layouts and hard structures was performed. This analysis gave insights into the importance of different inputs in Xbeach for dune erosion. The water level, wave height and wave period were found to be important parameters in Xbeach. The duration of the peak of the storm and the grain size were found not to be important parameters in Xbeach. This agrees with other findings by P and [23].

Hard layers were found to influence the amount of dune erosion. The top of the foreshore only eroded when a complete hard structure was present. For the other layouts the foreshore height was higher after the storm. Because the dike slope eroded onto the foreshore, which could decrease the failure probability. Also both increasing and decreasing the slope and foreshore height influenced the amount of dune erosion, especially compared to the absence of a foreshore. It is advised to use a layout where the dike slope is modelled as a non-erodible layer.

Sub-Question 2

- How to determine the failure probability of a hybrid levee due to wave overtopping?

Combining Riskeer to calculate the probability of failure for overtopping and Xbeach to simulate dune erosion, was an effective way to approach the failure probability of a hybrid levee. When using this method it is important that both Riskeer and Xbeach have the same design points and eroded foreshore profile. It only took one extra iteration to determine the failure probability. It is important what type of foreshore and hard layer combination is present. When the failure mechanism is wave overtopping, it is advised to model only the foreshore as sand and the dike as a hard structure. When determining the failure probability of a dune, a dune erosion calculation is advised over a wave overtopping calculation, to determine the probability of failure. It was found that the critical design point from wave overtopping led to the highest failure probability compared to other critical design storms.

Sub-Question 3

- What order of magnitude of transport can be expected at Almere Duin?

The alongshore flow velocity for different wind speeds and different wind directions was modeled with Delft3D. The flow velocities were multiplied by the occurrence of each wind direction and wind speeds and then for each combination of wind speed and direction the alongshore transport was calculated with the CROSMOR2013 LST formula. Then the transport for all combinations was summed to predict the yearly transport. The average yearly alongshore transport found was to be 321 m^3/y with a standard deviation of 325 m^3/y. The uncertainty the Delft3D model gave was very high due to the current close to the beach not being constant. The underestimation of transport by the CROSMOR2013 LST formula of a factor of 2, is common in coastal morphology.

Sub-Question 4

- How can a sandy foreshore be designed to minimize the life cycle cost?

The life cycle costs of six maintenance strategies, with a maintenance interval period of 1, 4, 10, 25 50 and 100 years, was calculated and the uncertainty and sensitivity of the life cycle costs, has been analysed. It was observed that optimizing the maintenance strategy lowers the life cycle costs. It was found that the largest maintenance buffer is optimal. Therefore it is concluded that assessing different maintenance strategies is important, to minimize the life cycle costs.

Lastly, a Monte Carlo simulation was performed, which did not lead to a different optimal maintenance strategy, however the probabilistic calculation did find higher life cycle costs of 20% to 34%. The model uncertainty was found to be slightly more influential than the yearly erosion rate. The uncertainty of the strategies was found to be between 9% and 27 % and to reduce the uncertainty it is suggested to nest the Delft3D simulation and calibrate the LST formula for low-energy non-tidal lake environments.

Main Research Question

- How can the time-varying protection level and maintenance strategy of flood defences with sandy foreshores in low-energy non-tidal shallow lake environments be optimized?

To answer the main research question different sub-questions were asked and each sub-question led to answering a part of the main question or helped to answer the next sub question.

The time varying protection level can be optimized by calculating the failure probability due to wave overtopping with Riskeer and calculating the erosion of the foreshore with Xbeach. To use this method, Riskeer and Xbeach should have the same design point, and if the design point shifts, multiple iterations are necessary.

A maintenance strategy with a four year maintenance interval period was found to be optimal in this study and a probabilistic calculation found similar results. It is therefore not deemed necessary, to carry out a probabilistic calculation, to find the optimal maintenance strategy. However the models used, as well as the local boundary conditions, such as the longshore transport and the cost of sand, were found to influence the life cycle costs significantly. For this reason, it is concluded that a location specific analysis is important to optimize the maintenance strategy.

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A

Check design point

A.1. Introduction

In this step Riskeer was be used to calculate different storms, with the same probability of occurrence as calculated in the final third step, which could possibly lead to a lower probability of failure. The next step was to model the dune erosion of the profile in Xbeach for the storms calculated in the previous step. Finally a schematization of the eroded profile was made and calculated again for overtopping in Riskeer.

For this calculation no hard layer was present. This led to more erosion than a layout with a hard dune which will make the difference in erosion more clearly and lead to possible observing difference between generated storms.

1. Calculate different critical storms in Riskeer.
2. Calculate dune erosion.
3. Schematise different profiles and calculate failure probability for overtopping again in Riskeer.

A.2. Method

Step 1: Calculate different critical storms in Riskeer for each profile

First was be evaluated if another storm will not erode the foreshore, which will weaken the profile, which in turn will make the newly emerged profile the new critical profile for overtopping. Also, dune erosion is evaluated to not be a critical failure mechanism.

To achieve this, different variations of a critical storm will be calculated in Riskeer. Meaning, for different setup levels the maximum wave height is calculated. This is done in Riskeer with the GEBU (grass erosion outer slope) mode. Here maximum wave heights will be calculated with 1/125.000 probability of occurrence, which is the permissible failure probability for overtopping at cross-section level. For this wave height also a wave period and angle of incidence are given in the design point in Riskeer. The wave period will change with the wave height to keep a constant wave steepness. The different wave height and setup combinations will be analysed and the combinations which could be critical will be simulated in 1D Xbeach with the same input parameters as explained before. This will result in the most critical combination.

Step 2: Calculate Dune Erosion

Calculate the dune erosion again with Xbeach, as explained in step 2. If the dune erosion is too much, it will be necessary to perform a 2D Xbeach simulation or 1D Xbeach with a SWAN model to calculate the refraction.

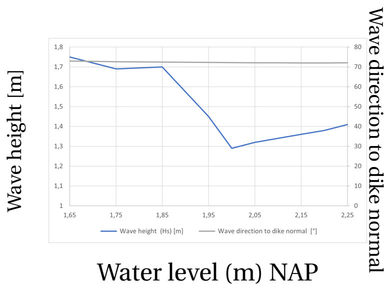
Step 3: Schematise different profiles and calculate failure probability for overtopping again in Riskeer

If necessary, the Xbeach profiles will be schematised in Riskeer to recalculate the failure probability for overtopping based on the eroded profiles. The failure probability should still be less than the standard.

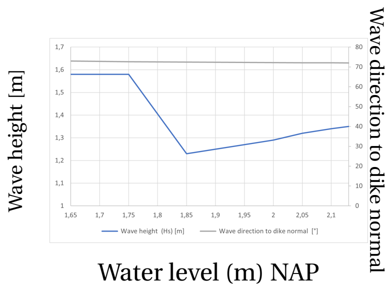
A.3. Result

Step 1

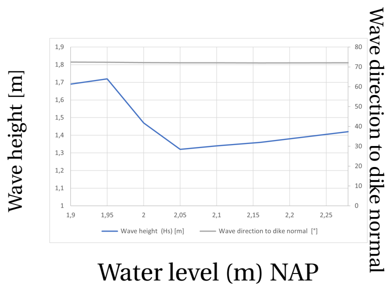
It could be possible that the critical storm is not the critical storm for wave overtopping, but a storm which leads to more dune erosion and thus decreases the wave breaking effect of the foreshore. First, to examine this, the different wave heights for each water level are shown below. The results are different than expected. A declining wave height was expected for the higher water levels, as there is less failure probability space left after generating a high setup to also generate a high wave height. However, the wave height seems to increase a little for the higher water levels.



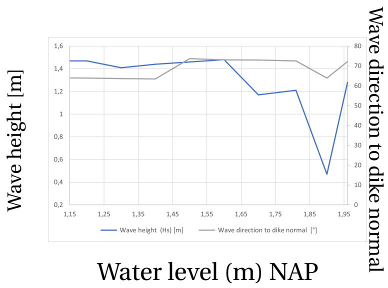
(a) Maximum wave height and angle of incidence for different setup levels for profile 1



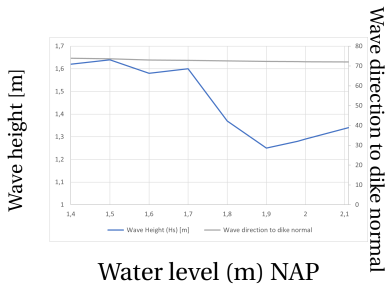
(b) Maximum wave height and angle of incidence for different setup levels for profile 1



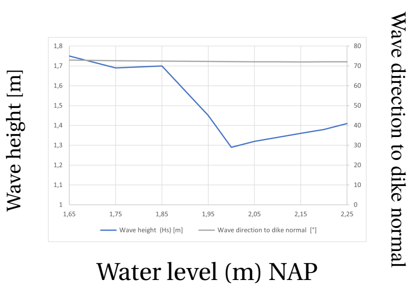
(c) Maximum wave height and angle of incidence for different setup levels for profile 3



(d) Maximum wave height and angle of incidence for different setup levels for profile 4



(e) Maximum wave height and angle of incidence for different setup levels for profile 5



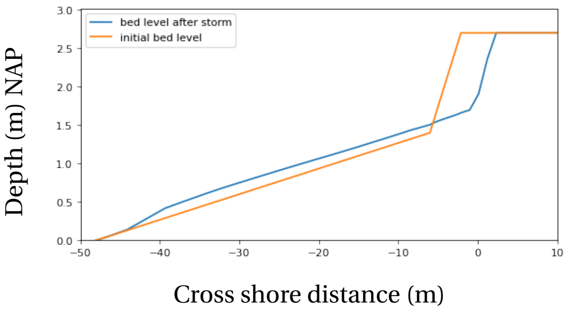
(f) Maximum wave height and angle of incidence for different setup levels for profile 6

Figure A.1: Figures with wave height per setup level generated in Riskeer

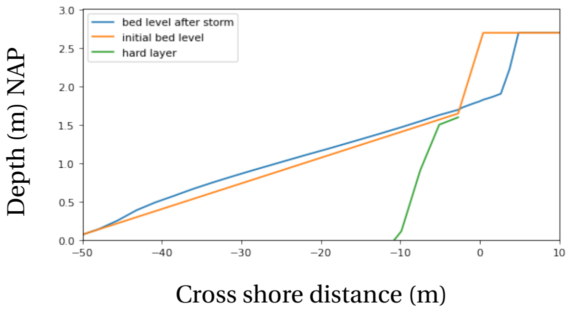
From this data two possible critical points were checked to have a lower failure probability than the safety standard. Firstly, the highest water level was examined, because a higher water level could lead to less wave breaking and higher wave height to more erosion. Secondly the option where the water level is at the toe of the dike was examined, because this usually leads to more erosion.

Step 2

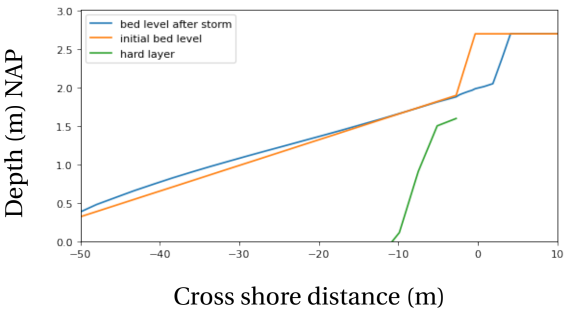
Figures showing the eroded profile with maximum water level. The foreshore height goes up expecting decreased probability of failure after the storm, just as the outcome before.



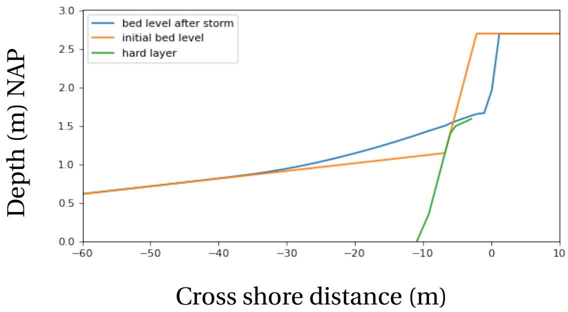
(a) Bed profile 1 after storm



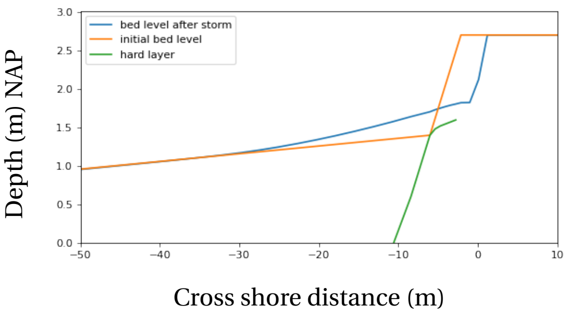
(b) Bed profile 2 after storm



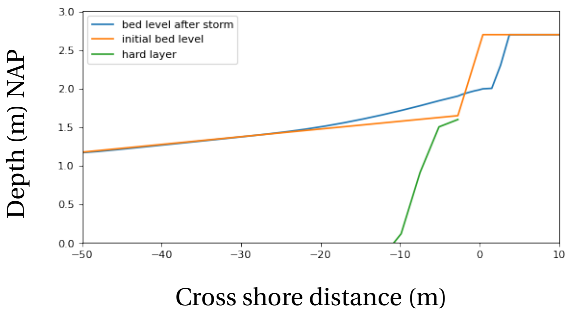
(c) Bed profile 3 after storm



(d) Bed profile 4 after storm



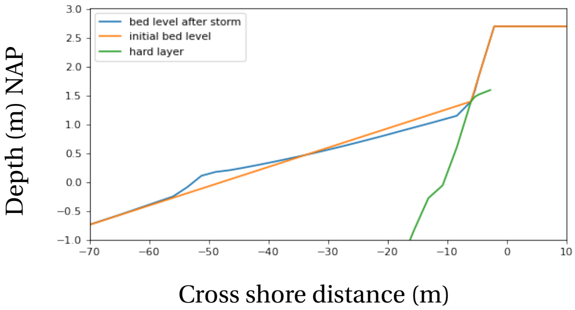
(e) Bed profile 5 after storm



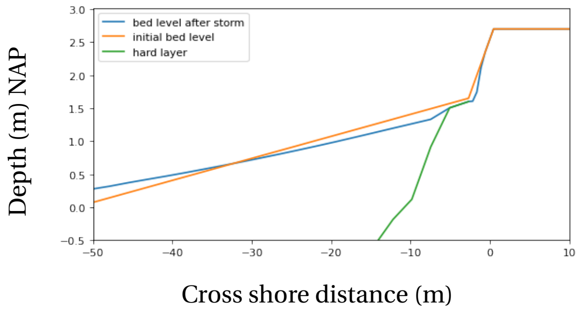
(f) Bed profile 6 after storm

Figure A.2: Figures with schematizations of the profiles, with maximum setup, simulated in Xbeach

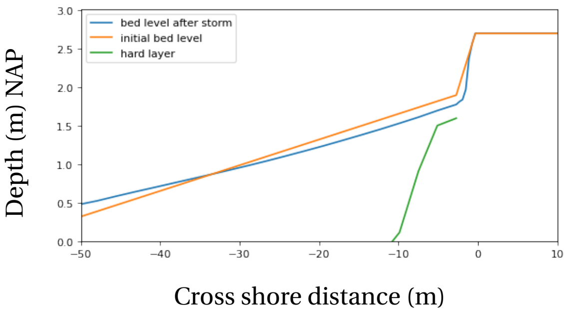
The figures below show the Xbeach simulations for water level at dike toe level for maximum erosion. There does not seem to be enough erosion to recalculate the failure probability, as it is probably lower or equal to what was calculated before.



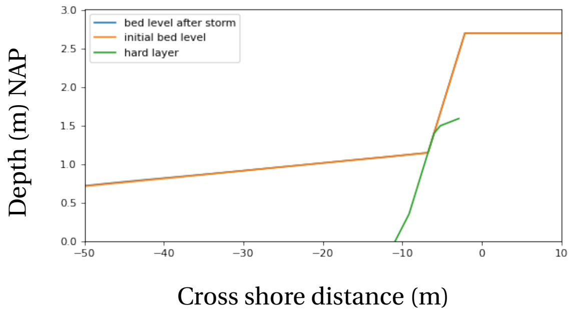
(a) Bed profile 1 after storm



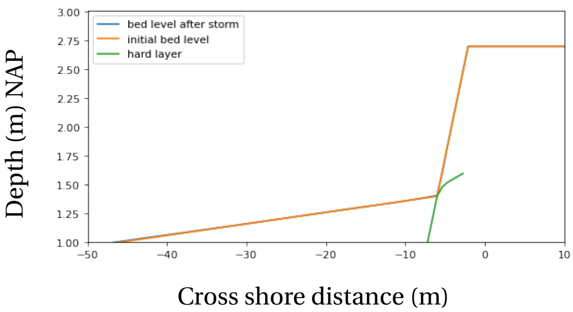
(b) Bed profile 2 after storm



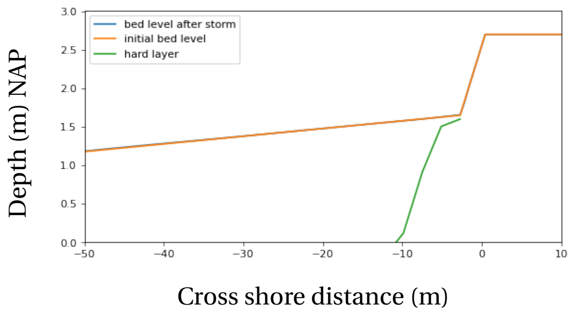
(c) Bed profile 3 after storm



(d) Bed profile 4 after storm



(e) Bed profile 5 after storm



(f) Bed profile 6 after storm

Figure A.3: Figures with schematizations of the profiles, with water level at the dike toe. Simulated in Xbeach

Step 3

As neither the high setup, nor the setup at the dike toe, caused significant erosion, where the dike toe is lowered or the dike slope is steepened, there was no need to schematize the profiles one more time and calculate the failure probability for wave overtopping.

A.4. Discussion and Conclusion

The aim of this appendix was to find if the design point in Riskeer leads to the highest failure probability. In the literature no study was found which shed light on this question. Multiple design points with equal probability of occurrence were calculated and tested for dune erosion. A design point with the highest setup and wave height and a design point with setup at the dike toe were compared to the design point from Riskeer for maximum wave overtopping.

The design point with high setup showed no significant difference in erosion to the design point with maximum overtopping discharge. The design point with setup at the dike toe showed little erosion of the foreshore, because only a little erosion was found and low setup leads to less overtopping no direct need was seen to calculate the probability of failure. Although it is suggested for later research to perform this calculation.

This research was important because a potential higher failure probability could have been found, which would suggest the method of approaching the failure probability used in this thesis would need to be altered.

To conclude these results suggests the design point from Riskeer can be used and no other design points need to be examined.