Assessment and quantification of the waterside slope erosion safety of a fully sandy levee at Kloosterbos

BSc thesis

Author:	Luuk van Laar
Student number:	S2162911
Supervisors UT:	Daan Poppema
	Denis Makarov
Supervisor Vallei en Veluwe:	Reindert Stellingwerff
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Preface

Before you lies my bachelor thesis 'Assessment and quantification of the waterside slope erosion safety of a fully sandy levee at Kloosterbos'. This has been an educational and inspiring journey. I have been working the past 10 weeks in the footsteps of a levee specialist to research something completely new. What was nice to see, is that I am not the only one enthusiastic about this subject. I have spoken to many people (virtually) and therefore got a good idea of working at Vallei en Veluwe. Even during corona times.

I would like to thank my external supervisor Reindert Stellingwerff (Vallei en Veluwe) for bringing me close to the working environment. I have spoken much with his colleagues and other levee specialists, which he often brought me in contact with. I would also like to thank Adrie van Ruiten (colleague) for sharing his knowledge about the erosion mechanisms and wave models. Thirdly, I would like to thank Menno de Ridder (expert on XBeach) for helping me understand the model better and giving new insights about how the model works. I would also like to thank my internal supervisor Daan Poppema (University Of Twente) for his flexibility and insights he had. These often helped me get further in the process. Lastly, I would like to thank everybody who shared their knowledge with me. It really helped me get to the thesis as it is now.

Luuk van Laar

Enschede, June 28, 2021

Abstract

English

In this study, a fully sandy levee is quantitatively assessed on its waterside slope erosion safety. It is located within the levee segment 52-4, near Zwolle, and is managed by the water authority Vallei en Veluwe. The levee consists of sand only and does not contain a grass or clay layer. It is therefore considered to be susceptible for erosion of the waterside slope. In previous assessment rounds by Vallei en Veluwe, the levee was considered as concealed levee and wide enough to handle erosion. This is a qualitative judgement of the waterside slope erosion safety and a quantitative analysis was missing. In this study, several existing levee erosion models were investigated to quantify the amount of waterside slope erosion happening at Kloosterbos.

At first, two wave models were compared (SWAN and Bretschneider) on their applicability for Kloosterbos. From these, the Bretschneider equations was considered best applicable, since it provided sufficient input data for the erosion model and had a large range of available data sets. The SWAN model gives more options regarding the output, but no SWAN data was available at Kloosterbos. Therefore, the Bretschneider equations were used to provide the input for the erosion model.

XBeach1D is used in this study as erosion model for Kloosterbos. This model was compared to four other erosion models (DUROS+, D++, equations of Klein Breteler et al. and DUROSTA), from which XBeach1D came out best to use for Kloosterbos. The other models mainly contained limitations regarding the wave characteristics. Investigation was done on the translation from the coastal to river settings, together with a sensitivity analysis for several parameters within XBeach1D. In the translation to the river settings, the focus was mainly put on the wave generation by JONSWAP (originally meant for the North Sea). It followed that JONSWAP can be used for the Kloosterbos situation. Following from the sensitivity analysis, angled waves would cause more erosion than perpendicular waves. The extension of the angled waves in XBeach1D has not been calibrated nor validated, but is used in the final safety assessment.

Further, a weakest link and ultimate limit state (minimum profile left after erosion) was defined for the Kloosterbos situation, which were needed for the eventual safety analysis. The weakest link is defined as the levee location at which the levee is most likely to fail due to erosion of the waterside slope. The weakest link was found to be located east in Kloosterbos. The levee reaches failure at the moment on which the ultimate limit state is exceeded. In this study, the boundary profile used in dune safety is also used as ultimate limit state for the Kloosterbos levee. The boundary profile is the minimum profile that should be left after erosion has happened.

All things considered, the waterside slope erosion safety of the levee was determined, using the safety categories stated in WBI2017. The erosion safety of the levee at Kloosterbos complied to the III_v (1/20,000 years) safety category. This means that the levee complies to the lower threshold for the waterside slope erosion for the levee section. Therefore, no levee measure is needed for Vallei en Veluwe, when looking at the waterside slope erosion.

Dutch

In deze studie wordt een volledig zanderige dijk kwantitatief beoordeeld op de erosieveiligheid van het buitentalud. Het ligt binnen traject 52-4, nabij Zwolle, en wordt beheerd door waterschap Vallei en Veluwe. De dijk bestaat alleen uit zand en bevat geen klei- en grasbekleding. De dijk wordt daarom gevoelig beschouwd voor erosie van het buitentalud. In de eerdere beoordelingsrondes van Vallei en Veluwe werd de dijk geacht als een verholen kering en werd de dijk breed genoeg beschouwd om erosie van het buitentalud te verwaarlozen. Dit is een kwalitatieve schatting van de erosieveiligheid aan het buitentalud en een kwantitatieve analyse ontbrak. In dit onderzoek zijn verschillende bestaande dijkerosiemodellen onderzocht om de hoeveelheid erosie van het buitentalud te kwantificeren bij Kloosterbos.

Aanvankelijk werden twee golfmodellen vergeleken (SWAN en Bretschneider) voor hun toepasbaarheid op Kloosterbos. Hieruit werden de Bretschneider vergelijkingen beschouwd als de best toepasselijke, aangezien deze voldoende invoer leverde voor het erosiemodel en een ruime hoeveelheid beschikbare datasets had. Het SWAN model geeft meer opties met betrekking tot de modeluitvoer, maar er waren geen SWAN gegevens beschikbaar bij Kloosterbos. In deze studie zijn de Bretschneider vergelijkingen gebruikt als input voor het dijkerosiemodel.

XBeach1D is gebruikt in deze studie voor het erosiemodel. Dit model werd vergeleken met vier andere modellen (DUROS+, D++, vergelijkingen van Klein Breteler et al. en DUROSTA), waaruit XBeach1D het beste uitkwam voor Kloosterbos. De andere modellen bevatten voornamelijk beperkingen met betrekking tot de golfkarakteristieken. Er is onderzoek gedaan naar de vertaling van de kust naar rivierinstellingen, samen met een gevoeligheidsanalyse voor verschillende parameters binnen XBeach1D. In de vertaling naar de rivierinstellingen lag de focus vooral op de golfgeneratie door JONSWAP (oorspronkelijk bedoeld voor de Noordzee). Hieruit volgt dat JONSWAP gebruikt kan worden voor de Kloosterbos situatie. Uit de gevoeligheidsanalyse volgde dat schuine golven meer erosie veroorzaken dan loodrechte golven. De toepassing van schuine golven in XBeach1D is echter niet gekalibreerd of gevalideerd, maar wordt wel gebruikt in de uiteindelijke veiligheidsbeoordeling.

Verder werden voor de Kloosterbos situatie een maatgevend profiel en faaldefinitie gedefinieerd, die nodig waren voor de uiteindelijke veiligheidsanalyse. Het maatgevend profiel wordt gedefinieerd als de dijklocatie, waar de dijk het snelst faaldefinitie bereikt door erosie van het buitentalud. Het maatgevend profiel ligt in het oosten van Kloosterbos. In dit onderzoek wordt het grensprofiel in duinveiligheid ook gebruikt voor Kloosterbos als faaldefinitie. Het grensprofiel is het minimale profiel dat na erosie achter moet blijven om veiligheid te waarborgen.

De erosieveiligheid van de dijk is bepaald op basis van de veiligheidscategorieën in WBI2017. De dijk bij Kloosterbos voldoet aan de III_v (1/20,000 jaar) veiligheidscategorie voor erosie van het buitentalud. Dit betekent dat de dijk bij Kloosterbos voldoet aan de norm voor de erosie van het buitentalud voor het dijkvak. Vallei en Veluwe hoeft daarom geen maatregel uit te voeren, kijkende naar erosie van het buitentalud.

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List of Abbreviations

GEBU	The failure mechanism which focuses on the erosion of the grass revetment at the waterside slope.
GEKB	The failure mechanism which focuses on the overtopping waves and erosion of the landside slope and crest of the levee.
DA	The failure mechanism which focusses on the erosion of dunes (dune erosion).
WBI2017	The levee assessment methodology, which is used to assess the levees in the Netherlands for the assessment round 2017-2023
HBN	The minimum height of the levee for which the levee does not fail due to the failure mechanism GEKB.

Safety categories:

Iv	= Certainly meets the alert value
IIv	= Meets the alert value
IIIv	= Meets the lower threshold and possibly the alert value at cross-section level
IVv	= Possibly meets the lower threshold at cross-section level or lower threshold
Vv	= Does not meet the lower threshold
VIv	= Certainly does not meet the lower threshold

1. Introduction

1.1. Problem Context

1.1.1. History of flood assessment in the Netherlands

In history, numerous floorings have taken place in the Netherlands. An example is the storm surge of 1953. The combination of extreme weather conditions and high water levels led to 150 levee breaches in the southwestern part of the Netherlands. All in all, 1836 people died with a total monetary damage of 5.4 billion euro's (RWS, sd). Because of this storm, the flood protection philosophy was developed by the Delta Commission. The core of this approach was formed by assigning a design water level and discharge to the levees in the Netherlands (Wesselink, 2007).

Over the years, the flood protection philosophy has changed. From 1960 on, the levee assessment method was based on flood probabilities only. The Delta Commission stated in 1960 that the damage caused by the flooding should also be considered, but there was insufficient knowledge at that time (M.J. Booij, personal communication, 2019). In 2006, a new project called VNK2 started on how to calculate the flood risk, which includes the damage in the hinterland. The results were published in 2014 and are integrated in the current levee assessment methodology 'WBI2017' (Rijksoverheid, 2021; STOWA, 2019). Every 12 years, all water authorities must carry out a levee safety assessment using the WBI2017 (Rijksoverheid, sd).

1.1.2. Water authorities

The authorities responsible for the flood risk management of the primary and regional flood defences system are Rijkswaterstaat, the provincial authorities and the water authorities. Rijkswaterstaat works on a national level. They manage the major waters, such as the sea and the rivers. The provincial authorities are responsible for setting the targets of water management, together with the rules, standards and policy. The water authorities work on a more regional level on the regional rivers (ENW, 2017). In total there are 21 water authorities, each assigned to an area in the Netherlands (Dutch Water Authorities, sd).

1.1.3. Situation at Kloosterbos

One of the regional water authorities currently assessing the levees is Vallei en Veluwe. Vallei en Veluwe is based in Apeldoorn and works in the provinces of the Utrecht and Gelderland. They started with their fourth assessment round using WBI2017 for the period 2017-2023. One of the levee segments assessed is 52-4. It is located below Zwolle, near Hattem. In Figure 1, this levee segment is shown. Further, the progress in the assessment is shown in green (finished), light green (in progress) and grey (to be done). As can be seen, levee segment 52-4 still needs to be assessed.

Figure 1: Levee section 'Kloosterbos' in the red circle (IHW, sd)

Levee segment 52-4 is subdivided into levee sections. One of these levee sections is called 'Kloosterbos' (Figure 1, in the red circle) and is located in a small forest area. The Kloosterbos levee section is mainly a naturally formed levee, but the east of the levee section does seem potentially man-made (see Figure 2). This could mean that the soil composition might be different along the Kloosterbos levee section. More information about the soil composition can be found in section 3.2.4.4.

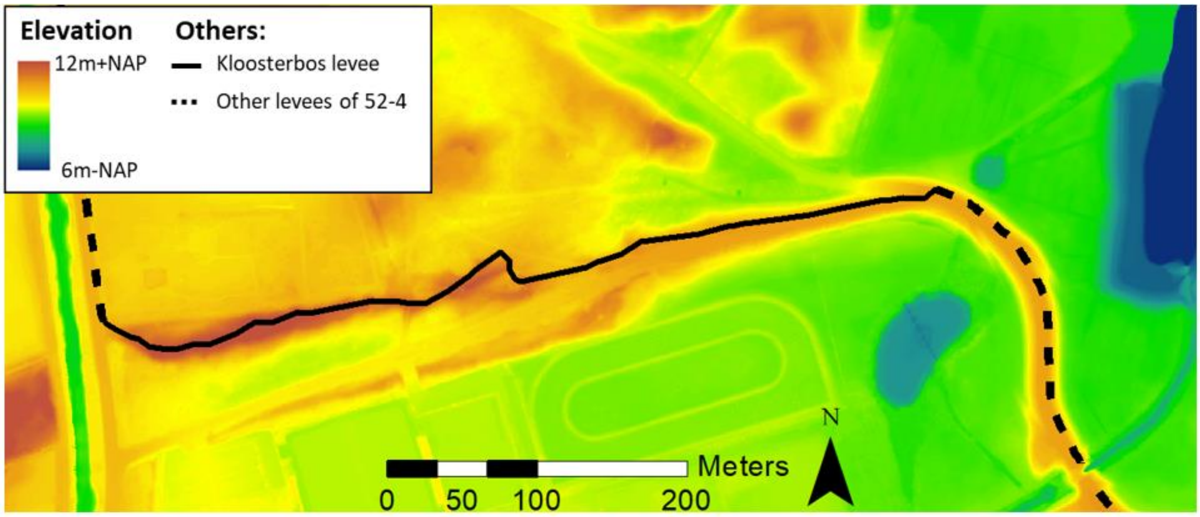


Figure 2: Elevation of at the Kloosterbos levee section

The levee section consists of a sand levee body without an erosion protective layer and has trees om top. The foreland consists of trees and vegetation. Normally, a river levee has a clay layer with a grass revetment. In Figure 3, a comparison is shown between a standard levee and the levee at Kloosterbos. The composition of the Kloosterbos levee is therefore rather different than from a standard levee, which makes assessing this levee more complicated.

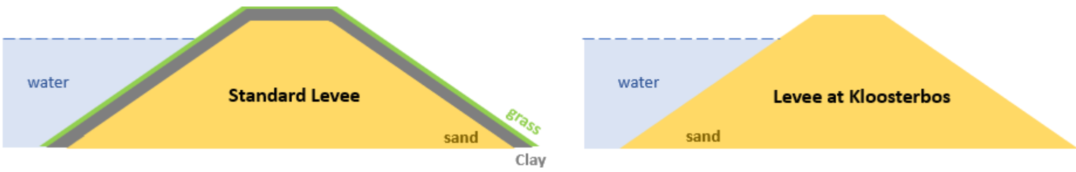


Figure 3: Standard river levee and the levee at Kloosterbos

In previous assessments, this levee section was identified as a concealed levee ('verholen waterkering'). This is a levee which is not recognisable as a levee body, but is part of a higher situated area (Waterschap Amstel, Gooi en Vecht, 2017). Based on qualitative expert judgement, this section was considered safe, due to the considerable amount of sand on top of the levee (Appendix A). However, a quantitative safety analysis was missing and the current methodologies available appear not to be suitable. To obtain more insight on the actual safety of the levee, a quantitative safety assessment should be carried out.

In this study, the waterside slope erosion mechanism is focused upon, since this is considered as one of the main threats for the levee at Kloosterbos. There is no erosive protective layer present, making the levee more susceptible for erosion of the waterside slope. This study will therefore contribute to the safety assessment of the levee section at Kloosterbos, by giving a quantitatively supported safety assessment for the erosion of the waterside slope of the levee at Kloosterbos.

1.2. Research objective

This study will focus on the safety assessment of the waterside slope erosion failure mechanism at the Kloosterbos levee section. This will be done by looking at current levee erosion models available and to see how they can be applied to this situation. The objective of this study is:

"To quantitively assess the fully sandy levee at Kloosterbos (Netherlands) on its waterside slope erosion safety by using existing levee erosion models"

A fully sandy levee is considered to be a levee to be completely consisting out of sand. The levee centre consists of sand as well as the levee coverage. There is no vegetation or clay layer on top of the levee. It may be comparable as a dune, since the levee body is in this case located under the sand. For the scope of this study, the trees on top of the levee are not included in the assessment. In Appendix A, a cross section is shown of the fully sandy levee at Kloosterbos.

In this study, erosion models are referred to as models which calculate a residual erosion profile after certain storm conditions. Most erosion models considered in this study are deterministic, determining the erosion profile for conservative values.

The levee at the Kloosterbos is susceptible for erosion, due to the loose soil on the levee. This situation is assessed with the erosion model found to be applicable for Kloosterbos. This model was adapted, where needed, to assess the Kloosterbos situation more correctly. The outcome of this study also has impact on the safety assessment of the other failure mechanisms. The other failure mechanisms (i.e. GEKB) fall out of the scope of this study, whereas these will be investigated by Vallei en Veluwe.

The waterside slope erosion safety is in this case determined by the safety categories used in WBI2017. These safety categories indicate how safe the levee is for waterside slope erosion and whether a measure is needed or not. In this study, a semi-probabilistic approach is used to get to the safety category corresponding to Kloosterbos. The wave characteristics are fully probabilistic determined and are used as input for the deterministic erosion model.

To quantitively assess the levee on the waterside slope erosion safety, the wave characteristics should be known. As mentioned in the problem context, the foreland of the levee consists of a small forest. From previous research, it follows that small forests on the foreland reduce the wave impact on the levee (Ren, et al., 2021; Suzuki, et al., 2010). However, when including vegetation in the safety assessment, a dependence on the quality of vegetation arises. The vegetation should always remain in the required quality. It may not be weakened or removed by natural processes or management plans. Further, the wave models used in WBI2017 do not have a function to incorporate vegetation easily yet. Therefore, in this study, the normative wave characteristics are taken excluding vegetation. From experiments, it was found that the current WBI2017 wave models do indeed overestimate the wave heights and underestimate the wave periods at the toe of the levee when there is a presence of vegetation in the actual situation (Steetzel, Groeneweg, & Vuik, 2018). This means that the failure probability will be overestimated and that the safety assessment of the Kloosterbos levee becomes more conservative.

1.3. Research Questions

The research objective is focussing on a quantitative safety assessment of the waterside slope erosion. To get to the safety assessment, it is needed to have an erosion model applicable for Kloosterbos. However, there are no erosion models yet, describing the levee situation at Kloosterbos. The main question of this research is therefore:

"How to model the waterside slope erosion for the fully sandy levee at Kloosterbos (Netherlands)?"

This question is subdivided into three sub-questions. The first focuses on the wave characteristics at Kloosterbos. These are needed as input for the erosion model. As mentioned, the wave characteristics were determined for the situation without vegetation on the foreland. There are several wave models which could model wave characteristics in general. It was investigated which model can adequately simulate the wave characteristics at Kloosterbos and could be applied for the waterside slope erosion model. This forms the first sub-question:

1- Which wave model can be used to determine the wave characteristics at Kloosterbos without including the vegetated foreland?

With the wave model known, an erosion model can be identified and adapted where needed, to get a quantitative safety indication of the waterside slope erosion at Kloosterbos. This forms sub-question two of this research:

2- Which erosion model can be used to provide a safety assessment for erosion of the waterside slope at the levee at Kloosterbos?

With both models, the safety assessment can be carried out for Kloosterbos, leading to sub-question three:

3- What is the waterside slope erosion safety of the levee at Kloosterbos?

2. Theoretical Framework

The theoretical framework mainly consists out of the ‘WBI2017 bijlage III’ methodology (RWS, 2017b). This regulation describes how the safety of the levees should be determined. This methodology is currently used as guideline for levee safety assessment. The failure mechanism to be studied, is erosion of the waterside slope of a levee. Two failure mechanisms were found in WBI2017, which could be applied to the Kloosterbos situation:

1. Grass revetment erosion waterside slope (GEBU)
2. Dune erosion (DA)

Below, both failure mechanisms are explained in more detail. Most failure mechanisms are assessed by using one of the three assessment levels. The first is the simple test, where it is tested whether the situation complies to certain basic rules. If so, the failure probability can be neglected. If not, the detailed test should be applied. This test uses calculation models to see whether the safety standards of the levee are met. If this model is not applicable for the levee section, the custom test should be carried out. This is a location specific analysis of the levee situation. The levee at the Kloosterbos is most likely to fall under the custom test, since this situation is not a regular levee section. However, insights from the simple and detailed test can be used to assess the Kloosterbos situation more accurately.

Further, the safety categories used in levee assessment are explained. These are used to indicate the level of safety of the levee.

2.1. Grass revetment erosion of the waterside slope (GEBU)

One of the failure mechanisms described in WBI2017 is erosion of the grass revetment at the waterside slope (GEBU). Erosion of the waterside slope is caused by two types of loads: the wave impact load and wave runup load. The erosion caused by the river flow is neglected in the GEBU failure mechanism ('t Hart, De Bruijn, & de Vries, 2016). In Figure 4, the erosion by wave impact is shown. In this study, states 3 and 4 in Figure 4 are most interesting, since erosion of sand takes place. However, the GEBU models do not simulate sand erosion, since the moment of failure is defined as the moment when there is a point on the levee where the clay and grass revetment are completely eroded. Therefore, simulation the erosion of the sand core is not necessary.

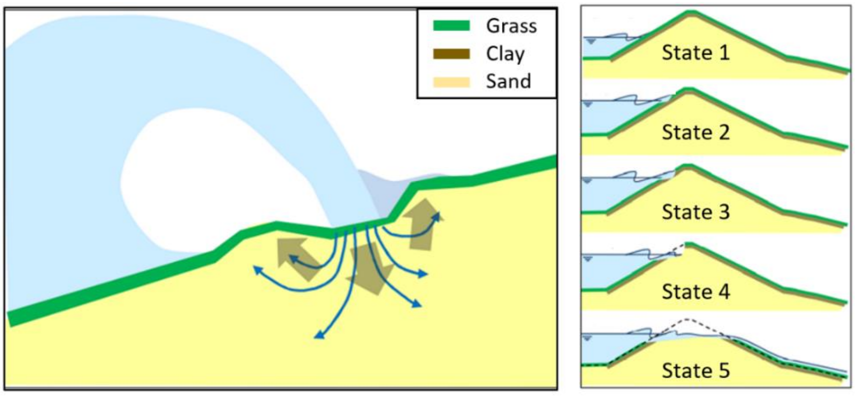


Figure 4: GEBU erosion due to wave impact ('t Hart, De Bruijn, & de Vries, 2016)

Within the GEBU assessment, the largest wave height is used to test the levee for. The levee should be able to withstand 12 hours with these extreme wave conditions (van Rinsum, Bisschop, Delhez, Lansink, & van Ruiten, 2020). If the clay and grass revetment are not completely eroded within this time, the levee complies to the corresponding safety category.

2.2. Dune erosion (DA)

This failure mechanism is focussing on the erosion of dunes. Failure by dune erosion is the situation where the dune is eroded up to the point where it does not protect the hinterland from flooding anymore(RWS, 2017b). The storm water level, wave height and wave period form the hydraulic boundary conditions. With the dunes, there is often a foreshore which breaks the waves. This decreases the wave height, but allows longer period waves to form (Figure 5). These longer period waves have most impact on the erosion of the levee, since the wind waves are often broken down by the foreshore before they reach the dune toe ('t Hart, De Bruijn, & de Vries, 2016).

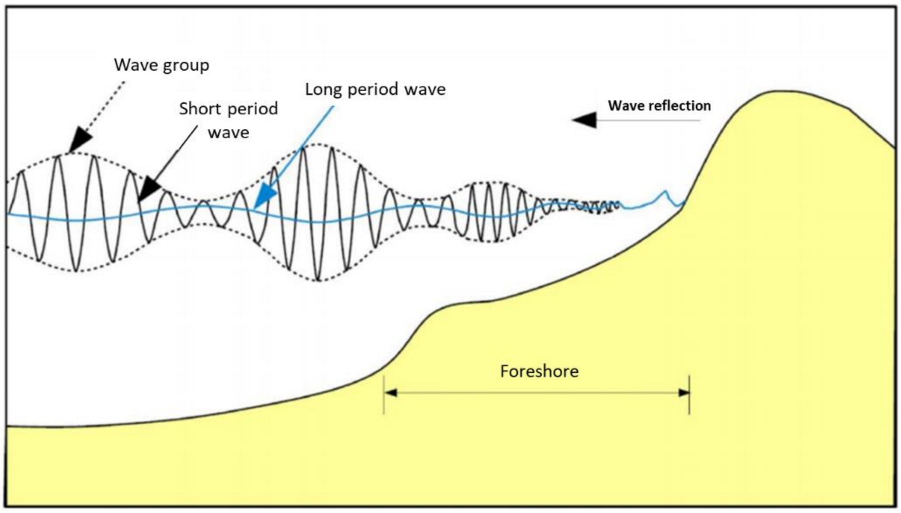


Figure 5: Long and short period waves ('t Hart, De Bruijn, & de Vries, 2016)

In Figure 6, the erosion process is shown for a dune flood defence. There is also a boundary profile ('grensprofiel') shown. The boundary profile consists of the same soil type as the rest of the dune. If the boundary profile is reached by the water, failure is reached. The dune will then fail due to erosion or other failure mechanisms. For this failure mechanism, there is only a detailed test available. Here, the erosion models DUROS+ and D++ are used. These models are mainly meant for the dunes located at the North Sea and therefore not yet applicable for river situations. More information on this failure mechanism can be found in the technical report for dune erosion (Van de Graaff, et al., 2007) and the schematic guideline for dune erosion (RWS, Water Verkeer en Leefomgeving, 2019).

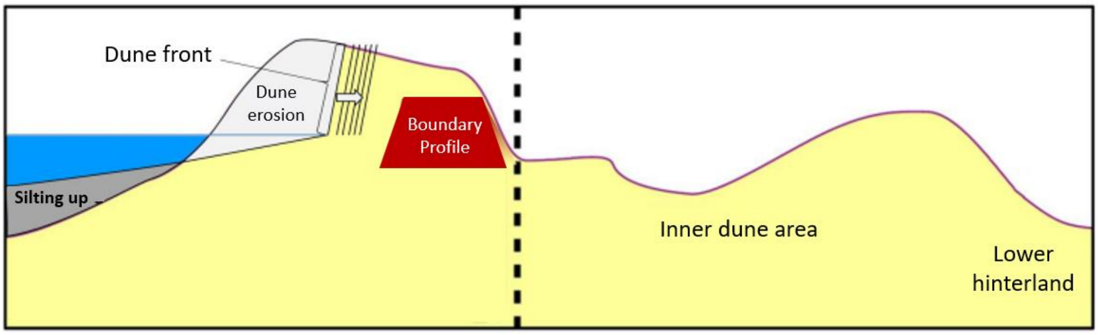


Figure 6: Dune erosion process ('t Hart, De Bruijn, & de Vries, 2016)

2.3. Safety categories in levee assessment

Within the Netherlands, river levees are given a lower threshold and alert value [1/years]. The lower threshold is maximum allowable failure probability for a levee. If the levee does not meet the lower threshold, then the levee is considered unsafe and a measure is required (ENW, 2017). The alert value is an extremer situation, which alerts the water authorities when exceeded. The levee is still safe, but not far off the lower threshold. The alert value is 1/3 of the lower threshold, so if a levee has a lower threshold of 1/1,000 years, the alert value is 1/3 of it: 1/3,000 years (RWS, sd).

The levee section Kloosterbos will be evaluated for the safety categories defined in WBI2017 (I_v, II_v, III_v, IV_v, V_v and VI_v). These safety categories each give a different safety indication, going from VI_v (certainly does not meet the lower threshold) up to I_v (certainly meets the alert value). The safety categories for all levee sections in 52-4 (I_v, II_v, etc.) will be assembled together into safety categories for levee segments (I_t, II_t, etc.). The levee segment safety categories indicate whether the levee segment meets the norm and whether a measure is needed. The Kloosterbos erosion situation complies to group 3 of failure mechanisms in WBI2017. For this group, it is prescribed that the weakest levee section represents the safety category for the whole levee segment (RWS, 2017b). If the levee at Kloosterbos is weakest and does not meet the norm (IV_v or lower), levee segment 52-4 neither meets the norm and a measure is needed. In this study, focus is on the levee section safety categories (I_v, etc.). The safety categories for levee segments will not be elaborated any further.

2.3.1. Relative failure contribution GEBU

Within the safety assessment of a levee, a failure probability budget is set up. This defines the relative contribution of each failure mechanism to the overall safety assessment of the levee. For each failure mechanism, a contribution factor (ω) has been set. This factor is used to obtain the so called failure probability at cross-section level. The sum of all failure contributions is equal to 1 (RWS, 2017b). In Table 1, the contribution factors are summarized in the overall failure probability budget.

Table 1: Failure contribution of each failure mechanism (sum=1)

Failure mechanism	Dunes (ω)	Levees (ω)
Height hydraulic structure (HTKW) or grass erosion of the levee crest and landside slope (GEKB)	0	0.24
Piping (STPH)	0	0.24
Macrostability landside slope	0	0.04
Grass erosion waterside slope (GEBU)	0	0.05
Other revetments waterside slope	0	0.05
Reliability closed hydraulic structure (BSKW)	0	0.04
Piping at hydraulic structure (PKW)	0	0.02
Strength and stability hydraulic structure (STKWp)	0	0.02
Dune erosion (DA)	0.70	0
Other failure mechanisms	0.30	0.30

The failure probability at cross-section level is the probability for which the failure mechanism is tested (ENW, 2017). In this study, the focus is put on the waterside slope erosion (GEBU), having a relative failure contribution factor of 0.05 (Table 1). One might consider the Kloosterbos situation as a river dune, giving a relative failure contribution factor of 0.70. However, the failure probability budget must be set up for a whole levee segment. In this case, levee segment 52-4 mostly consists out of standard levees, causing to work with the relative contribution factor for the 'Levees' in Table 1. Thus, the waterside slope erosion should be tested for GEBU ($\omega = 0.05$).

The failure probability at cross-section level can be determined with (RWS, 2017b):

$$P_{eis,dsn} = \frac{\omega P_{eis}}{N_{dsn}} \tag{1}$$

Where:

$P_{eis,dsn}$ $= Failure\ probability\ at\ dike\ sectional\ level\ [1/year]$
ω $= Failure\ contribution\ [-] = 0.05$
N_{dsn} $= Length\ effect\ factor\ for\ the\ dike\ section\ [-]$
P_{eis} $= Failure\ probability\ for\ the\ dike\ trajectory\ [1/year]$

The length effect factor (N_{dsn}) mainly accounts for the variability of spatial characteristics over a levee segment. If an important uncertain parameter is likely to change substantially from point to point, the length effect factor becomes higher (ENW, 2017).

2.3.2. Safety categories

In WBI2017, six different safety categories are defined for a levee section. They are described in Table 2. Here, P_f is the actual failure probability of the levee. These safety categories are used to give the safety assessments on levee sections. An example assessment is described below Table 2.

Table 2: The general safety categories in levee safety assessment

Category	**Explanation categories**	**Category range** P_f Failure probability levee [1/years] $P_{eis,dsn,alert}$ Alert value at cross-section level [1/years] $P_{eis,dsn,low}$ Lower threshold at cross-section level [1/years] $P_{eis,low}$ Lower threshold [1/years]		
I_v	Certainly meets the alert value		$P_f <$	$\frac{1}{30} P_{eis,dsn,alert}$
II_v	Meets the alert value	$\frac{1}{30} P_{eis,dsn,alert}$	$< P_f <$	$P_{eis,dsn,alert}$
III_v	Meets the lower threshold and possibly the alert value at cross-section level	$P_{eis,dsn,alert}$	$< P_f <$	$P_{eis,dsn,low}$
IV_v	Possibly meets the lower threshold at cross-section level or lower threshold	$P_{eis,dsn,low}$	$< P_f <$	$P_{eis,low}$
V_v	Does not meet the lower threshold	$P_{eis,low}$	$< P_f <$	$30P_{eis,low}$
VI_v	Certainly does not meet the lower threshold	$30P_{eis,low}$	$< P_f$	

Example:

A levee has a lower threshold of 1/100 years ($P_{eis,low}$) and an alert value of 1/300 years ($P_{eis,alert}$). The levee is tested for GEBU ($\omega = 0.05$) and the length effect factor is 1. The lower threshold and alert failure probability at cross-section level are then $P_{eis,dsn,low}$ =1/2,000 years and $P_{eis,dsn,alert}$ =1/6,000 years. The levee is tested for the 1/100; 1/2,000 and 1/6,000 years situation. Here are some possible outcomes:

Table 3: Example safety categories

	Situation [1/years]			Safety category	Measure required?
	$P_{eis,low}$ 1/100	$P_{eis,dsn,low}$ 1/2,000	$P_{eis,dsn,alert}$ 1/6,000		
Failure reached?	YES	YES	YES	I_v or II_v	No
	YES	YES	-	III_v	No
	YES	-	-	IV_v	Yes, the levee is not far off the lower threshold
	-	-	-	V_v or VI_v	Yes, the levee does not meet the lower threshold anymore

3. Methodology

3.1. Wave model used for Kloosterbos

To determine the erosion safety at Kloosterbos, the wave characteristics are needed. These are necessary as input for the erosion model. The type of wave characteristics depends on the erosion model (i.e. wave characteristics at the levee toe or at deeper waters?). In this study, the Bretschneider equations were used to determine the wave characteristics. The Bretschneider equations are currently used in the WBI2017 assessment for the upper river area (including Kloosterbos) and has a wide dataset available to work with (RWS, 2017a). Below, the choice of the wave model is further elaborated upon, together with more elaboration on the structure of the Bretschneider equations.

3.1.1. Selection of wave model for Kloosterbos

A short literature study was carried out to see which wave models could be used to determine the wave characteristics at Kloosterbos. Within the current levee safety assessment, two main wave models are used. These are the SWAN model and Bretschneider equations. Since this study mainly focuses on the erosion model, no further wave models were considered.

3.1.1.1. Bretschneider

The Bretschneider equations are based on three main parameters, namely the wind speed, effective fetch and average water depth. The Bretschneider equations determine the significant wave height (H_s) and peak period (T_p) at one location. The Bretschneider equations are based on only these three parameters mentioned and do not directly include other processes playing a role (e.g. due to a sudden change in bed level). They do however estimate wave propagation rather well for the river areas in the Netherlands (Camarena Calderon, Smale, van Nieuwkoop, & Morris, 2016). Thus, the Bretschneider equations are limited regarding wave propagation processes, provide data at only one location and only provide H_s and T_p. However, they do estimate wave propagation rather well and there is already a dataset available for the Kloosterbos site.

3.1.1.2. SWAN

The SWAN model is a 2D model developed by TU Delft, which calculates the wave propagation for larger areas (Liang, Gao, & Shao, 2019). The SWAN model is often applied in the dune erosion models and may therefore be more applicable for the erosion model used at Kloosterbos (Gautier & Groeneweg, 2012). The SWAN model extensively describes wave development processes, whereas Bretschneider only looks at three parameters (wind, fetch and water depth). Further, SWAN delivers a 2D output format of the wave characteristics, meaning the wave characteristics can be easily read at deeper and shallower waters. However, there is no database yet available for the Kloosterbos situation, whilst there is one for Bretschneider.

3.1.1.3. Choice of wave model for Kloosterbos

The Bretschneider equations provided sufficient output parameters for the erosion model (XBeach1D, see section 3.2.3) and had a dataset available for Kloosterbos. However, no dataset was available for the SWAN model and setting up such a 2D model, does cost a large amount of time. Therefore, the wave characteristics were used from the Bretschneider equations.

3.1.2. Bretschneider equations

The Bretschneider equations are based on the wind speed at 10m height (u), effective fetch length (F) and water depth (d). The Bretschneider equations are as follows (Camarena Calderon, Smale, van Nieuwkoop, & Morris, 2016):

$$\bar{H} = 0.283\tanh\left(0.530\bar{d}^{0.75}\right)\tanh\left[\frac{0.0125\bar{F}^{0.42}}{tanh(0.53\bar{d}^{0.75})}\right]$$

$$\bar{T} = 2.4\pi\tanh\left(0.833\bar{d}^{0.375}\right)\tanh\left[\frac{0.077\bar{F}^{0.25}}{\tanh(0.833\bar{d}^{0.375})}\right] \quad (2)$$

$$\bar{d} = \frac{dg}{u^2}, \bar{F} = \frac{Fg}{u^2}, \bar{H} = \frac{H_S g}{u^2}, \bar{T} = \frac{T_S g}{u}$$

Where:

$g =$ Gravity acceleration $[\mathrm{m/s^2}] = 9.81\mathrm{m/s^2}$
$u =$ Wind speed at 10m height $[\mathrm{m/s}]$
$d =$ Average water depth $[\mathrm{m}]$
$F =$ Effective fetch length $[\mathrm{m}]$
$H_s =$ Significant wave height $[\mathrm{m}]$
$T_s =$ Significant wave period $[\mathrm{s}]$

The wind speed at 10m height is one of the driving factors for developing the waves in rivers. As can be derived from the equations, the stronger the wind, the higher and longer the waves. The probability of the waves is related with the probability of the wind speed.

The effective fetch length is the length on the water over which the wave is developing (Camarena Calderon, Smale, van Nieuwkoop, & Morris, 2016). The longer the fetch, the more time the waves have to develop. This means that the significant wave height and peak period increase with the length of the fetch, which can also be derived from the equations. Possible fetches for a point on a levee is shown is Figure 7.

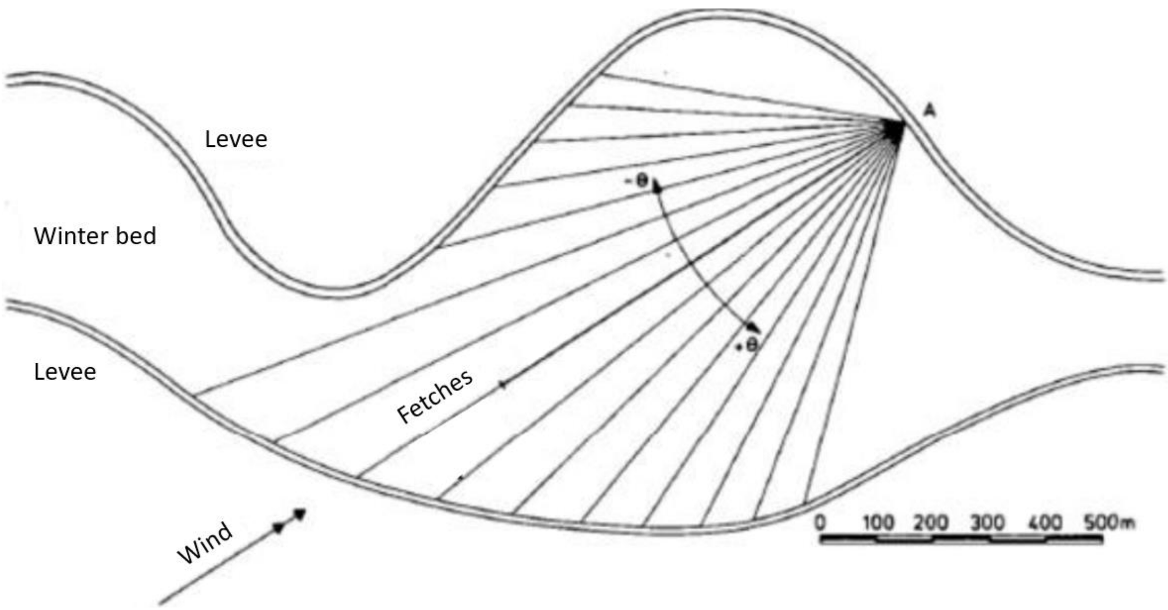


Figure 7: Example of fetches for point A on a levee (Camarena Calderon, Smale, van Nieuwkoop, & Morris, 2016)

The water depth also plays a role in determining the wave characteristics. The shallower the water is, the smaller the waves will be. In the Bretschneider equations, the averaged water depth is taken for the representative fetches.

3.2. Erosion model used for Kloosterbos

Next to the wave model, an erosion model is needed to simulate the erosion of the waterside slope. The erosion model found to be applicable for Kloosterbos is XBeach1D (version: Kingsday) in the supporting module MorphAn (version: 1.9.0). Xbeach1D is a numerical and deterministic model, which simulates hydrodynamic and morphodynamical processes. MorphAn is the supporting module, providing the input and results of XBeach1D in a user-friendly environment. In this section, more elaboration will follow on how other water authorities are dealing with fully sandy levees, which erosion models were considered for the Kloosterbos situation, how XBeach1D works and how the sensitivity analysis for XBeach1D is carried out.

3.2.1. Water authorities

In total, four water authorities (including Vallei en Veluwe) shared their methodology used for the erosion safety assessment of fully sandy levees. These approaches were also considered for the Kloosterbos situation. The external water authorities which were contacted, are Waterschap Limburg, Waterschap Rijn en IJssel and Waterschap Drents Overijsselse Delta. Each water authority assessed their levees in different ways. Their methods are summarized below and a comparison is made to see whether it may be applicable for Kloosterbos.

3.2.1.1. Waterschap Limburg – High grounds

The water authority 'Waterschap Limburg' had to assess higher located grounds. These high grounds were not made by men, but were naturally formed. Before the assessment round 2017-2023, these high grounds did not have to be assessed on their safety. However, since the start of 2017, things have changed (Ministerie van Infrastructuur en Milieu, 2017). Some of the high grounds have been included in the levee segments (see Figure 8). The approach of Waterschap Limburg for these high grounds is described in (van Rinsum, Bisschop, Delhez, Lansink, & van Ruiten, 2020).

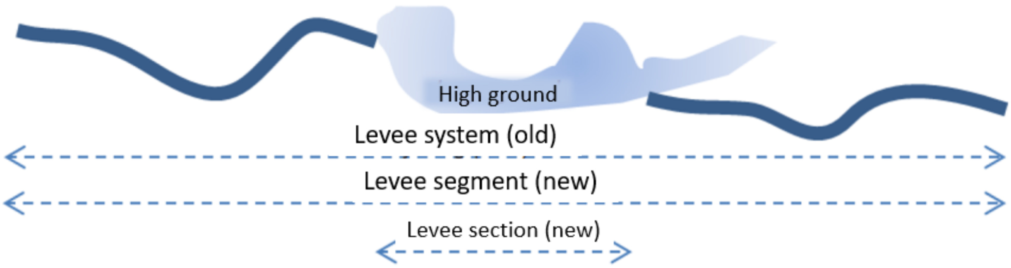


Figure 8: High ground located inside of the current levee segment (van Rinsum, Bisschop, Delhez, Lansink, & van Ruiten, 2020)

The high grounds were assessed by looking at their height and testing whether this complies with the GEKB failure mechanism. The GEKB failure mechanism focuses on the overtopping waves and erosion of the landside slope and crest of the levee. A maximum overtopping discharge of 0.1l/s/m is taken. With this, the HBN ('Hydraulisch Belastings Niveau' [m+NAP]) is determined for the lower threshold and alert value. The HBN is the minimum height of the levee for which the levee does not fail due to GEKB. If the stretch of high ground is completely above the HBN for the alert value, the high ground is considered safe and no measure is needed. If there is a point where the high ground is lower than the HBN for the alert value, the high ground is planned to be reinforced in a next levee reinforcement project (van Rinsum, Bisschop, Delhez, Lansink, & van Ruiten, 2020).

This method is partly taken up in the Kloosterbos levee too. The GEKB method will be used to determine the height of the boundary profile of the Kloosterbos levee (see section 4.2.2).

3.2.1.2. *Waterschap Rijn en IJssel*

Waterschap Rijn en IJssel focused on the width of the levee instead of height. Their methodology is described in (Huis in 't Veld, 2020). There are three sections located in the levee segment 50-1, marked as concealed levee (levee covered with sand). Waterschap Rijn en IJssel assessed these three sections, by looking at the width of the levee. The remaining width of the concealed levee was measured for the hydraulic boundary conditions used for GEBU. For GEBU, the largest wave should be taken with the corresponding water level. It was not mentioned what situation the GEBU characteristics were derived from (norm, alert value etc.). With the GEBU conditions, the remaining width of the levee sections were measured (see Table 4).

Table 4: Remaining levee width for hydraulic boundary conditions GEBU (Huis in 't Veld, 2020)

Levee section	**Maximum wave height for GEBU**		**Remaining levee width**
	Water level	Wave height	
Concealed levee 1	9.5m+NAP	0.28m	ca. 40m
Concealed levee 2	9.5m+NAP	0.50m	ca. 60m
Concealed levee 3	9.5m+NAP	0.55m	>100m

From the relatively large remaining levee widths, with all waves being smaller than 0.55m, it was reasoned that the levee would not fail. This is a rather similar method as was used in previous assessment rounds for Kloosterbos. This method is therefore not further considered in this study.

3.2.1.3. *Waterschap Drents Overijsselse Delta*

Waterschap Drents Overijsselse Delta (WDOD) had just finished the assessment of the 9-1 levee segment, near Dalfsen and Zwolle, which is described in (WDOD, 2019). East of Dalfsen, the 9-1 levee segment mainly consists of fully sandy levees. WDOD looks at the lower threshold water level and checks whether the remaining levee profile, above water level, is 100m wide. If the remaining levee profile is wider than 100m, the failure probability is considered to be negligible. If the remaining levee profile is smaller than 100m, the levee is considered unsafe and measures are needed. Again, this method is rather qualitative and will not be included for the Kloosterbos levee.

3.2.1.4. *Waterschap Vallei en Veluwe*

Another methodology used to assess fully sandy levees, is the use of a minimum required profile described in Handreiking Constructief ontwerpen (den Adel, Barends, de Groot, & Heemstra, 1994). This minimum required profile is derived from the dune erosion mechanism. If this minimum profile fits within the current levee profile, the levee can be considered safe for outer slope erosion (see Figure 9, in red). This method has been used by Vallei en Veluwe as simple test to see whether fully sandy levees would fail (van Ruiten, 2021). If the minimum profile did not fit, more investigation followed. In this study, the focus is on the detailed investigation, rather than the simple test. Further, the minimum profile did not fit in the weakest link at Kloosterbos and is thus not further considered.

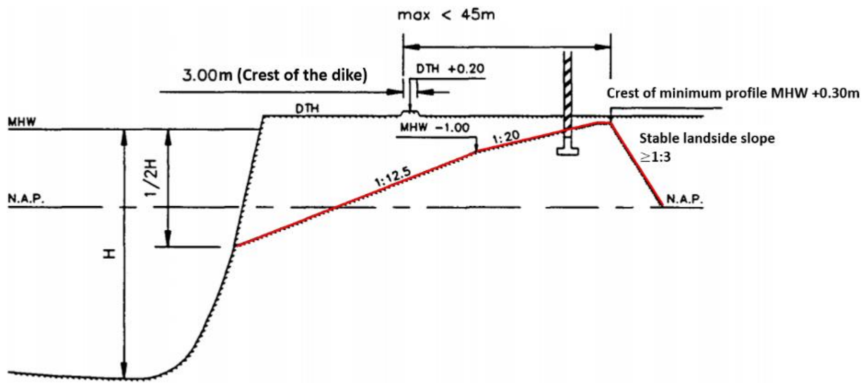


Figure 9: Minimum required profile (in red) used in the simple test of Vallei en Veluwe (den Adel, Barends, de Groot, & Heemstra, 1994)

3.2.2. Selection of erosion model for Kloosterbos

A literature study was conducted to see which erosion models could be applicable for Kloosterbos. In Appendix B, more detailed elaboration on each considered erosion model and its applicability for Kloosterbos can be found. It was found that the erosion models used for river levees are not applicable (i.e. BM Gras Buitentalud), since these assume levees with grass and clay layers. They define the moment of failure as the moment where the grass and clay revetments are fully eroded and the sand core is reached. This means that the erosion of sand is not included in BM Gras Buitentalud (RWS, 2017b), whilst this is needed for the fully sandy levee at Kloosterbos. There are other models, which do consider the erosion of the sand core. An example is Het Bekledingsmodel, applying the equations of Klein Breteler et al. 2012) for sand erosion.

Another representation of the Kloosterbos situation is to consider the levee as a small dune. There is a wide variety of dune erosion models available, which describe the waterside slope erosion process. For the assessment round 2017-2023, the DUROS+ and D++ model are used to assess the dunes for the coastal regions of the Netherlands (Boers, 2015). These two models are integrated into the supporting module MorphAn. Further, two other dune erosion models were considered to use for Kloosterbos: DUROSTA and XBeach.

Table 5 summarizes the (dis)advantages of each of the five erosion models regarding their applicability for Kloosterbos. XBeach was found to be best applicable for Kloosterbos, since the other models mainly had restrictions regarding the wave characteristics. The only disadvantage XBeach has regarding the Kloosterbos situation, is that it is meant for the coastal region. However, this is the case for most of the considered erosion models.

DUROSTA and the equations of Klein Breteler et al. (2012) also seem quite reasonable to use for Kloosterbos. However, the equations of Klein Breteler et al. are only valid for higher waves larger than 0.7 meters (Klein Breteler, Capel, Kruse, Mourik, & Kaste, 2012), which are not present at Kloosterbos (waves are ~0.3m high). DUROSTA does not have wave restrictions, but is an outdated model, performing worse than XBeach for waterside slope erosion (van Santen, Steetzel, van Dongeren, & van Thiel de Vries, 2012). Hence, XBeach is considered best applicable for Kloosterbos.

Table 5: The possible erosion models for Kloosterbos

Model	**Applicable for Kloosterbos?**	
	Yes	**No**
DUROS+	• Simulates erosion of dunes	• HBC* at ~20m depth • Simulates short storms of 4-6 hours • High T_p** range of 12-20s • Meant for the coastal region
D++	• Simulates erosion of dunes • HBC* at shallow waters	• Simulates short storms of 4-6 hours • High T_p** range of 12-20s • Meant for the coastal region
DUROSTA	• Simulates erosion of dunes • T_P** range <12s	• Outdated model • Performs worse than XBeach for determination erosion profile • Meant for the coastal region
XBeach	• Simulates erosion of dunes • Allows HBC* at shallow waters • No specific T_p** range	• Meant for the coastal region
Equations of Klein Breteler et al. (2012)	• Simulates erosion of sand levee cores • No specific T_p** range	• Only valid for waves higher than >0.7m • Validated for levees only with clay layer

* Hydraulic Boundary Conditions
** T_p = Peak period of waves

Within XBeach, there are several versions which one can opt for. In XBeach, it is possible to work in a 1D or 2D environment. In this study, the 1D environment is chosen. It was considered best not to apply an already detailed 2D model, but more a simplified model to see whether the general model estimates erosion for river dunes well or not. Within the 1D model, the assumption is made that there is no sediment transport alongshore. The Kloosterbos levee has a high foreland with vegetation lying rather far from the summer bed (Figure 10), which makes it a reasonable assumption that there is no substantial current present. Further, it is assumed in this study that the alongshore foreland at Kloosterbos is rather uniform, in order to neglect 2D processes. A rather deep foreland will be taken, which will cause higher waves with more erosive impact (RWS, 2018) and make sure that the output erosion profile of the model is not underestimated.

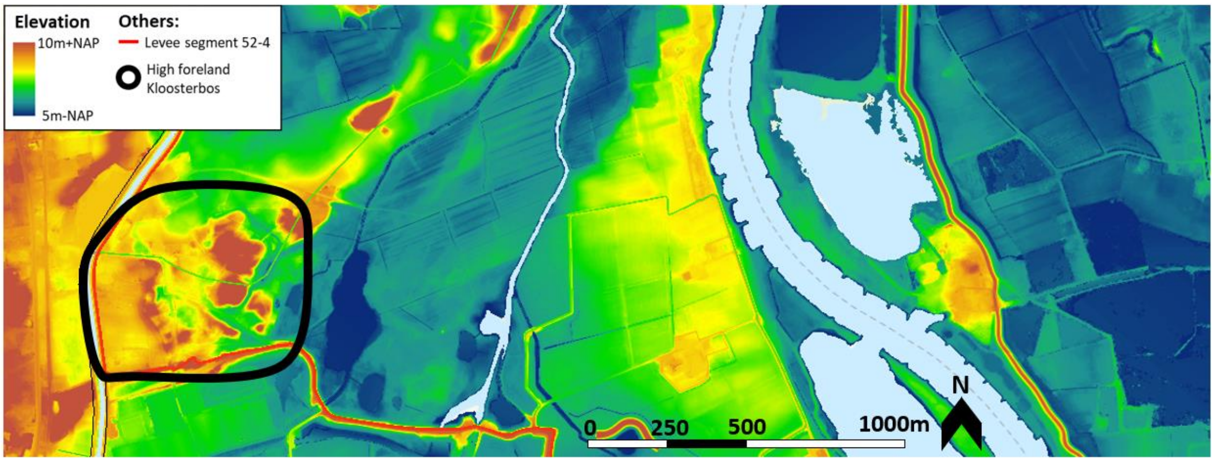


Figure 10: Elevation of the foreland at Kloosterbos, from GeoWeb 5.5

In the supporting module MorphAn (v1.9.0), XBeach1D is available. This is the Kingsday version of XBeach and is used in this study. The newest version XBeachX was also considered, but is not implemented in MorphAn and is therefore less user-friendly to use. The updates made in XBeachX after the Kingsday version, were mostly based on gravel morphodynamics, bermslope effects (steep foreland) and the updated sediment transport by Van Rijn (1993) (Deltares, 2018). The Kloosterbos levee does not contain gravel and does not have a steep foreland. Further, XBeach1D uses a different set of sediment transport equation than the Van Rijn equations, namely the Van Thiel-Van Rijn equations (2009). Therefore, the older XBeach1D model is considered sufficient to use.

Concluding, the XBeach1D version Kingsday can be used to provide a safety assessment for erosion of the waterside slope at the levee at Kloosterbos.

3.2.3. XBeach1D

XBeach1D is a 1D wave-group resolving model for wave propagation, infragravity waves, sediment transport and morphological changes of dunes and coastal beaches. XBeach1D solves the shallow water equations, the roller action balances (short wave action), sediment transport equations and updates the bed level following from these (see Figure 11).

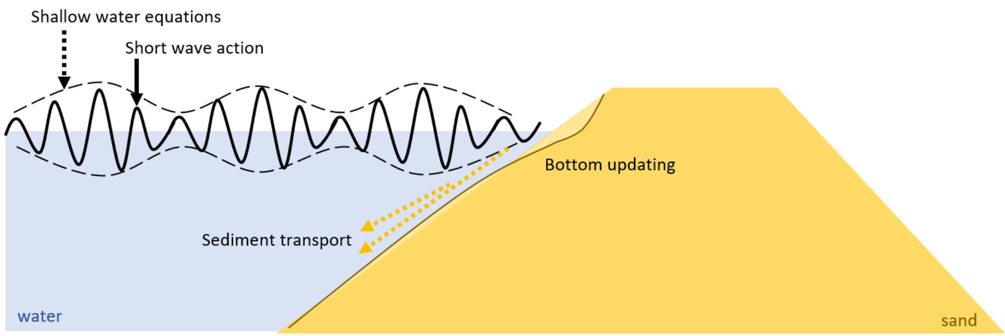


Figure 11: XBeach1D general components

In XBeach1D, the shallow water equations describe the infragravity waves flow below the water surface. The infragravity waves are formed by the shorter waves. For the propagation of shorter waves, much detail is given to the shape, dissipation, roller energy and current interactions (Deltares, 2020a). To include the short wave induced mass-flux and the return flow in shallow water, Xbeach1D applies the depth-averaged Generalized Lagrangian Mean formulation (Andrews & Mcintyre, 1978). To include the directional spreading of the waves in the 1D model, a reduction factor is implemented in the short-wave group variance (Deltares, 2020a). The sediment transport rates are computed by an advection-diffusion equation (Galappatti & Vreugdenhil, 1985) and the sediment equilibrium concentration (source-sink) is calculated using the Van Thiel-Van Rijn transport equations (2009). Important to note is that the alongshore sediment gradient is neglected in the 1D environment. Further, the inundated and dry areas are checked whether avalanching will happen or not. In XBeach1D, the critical inundated slope is 1:4 (z:x) and dry slope 1:1. When the slope is steeper than the critical slope, the avalanching process takes place where the sand will slump until the critical slope is reached again. More elaboration on the model structure can be found in the XBeach manual (Deltares, 2020a).

3.2.3.1. Assumptions XBeach1D

Within XBeach1D, several assumptions are made. The relevant assumptions for this study are listed below:

1. *Alongshore currents are neglected*
   In XBeach1D, the alongshore currents are neglected. Therefore, the model is made for situations where there are no substantial currents are present. More elaboration on the alongshore currents at Kloosterbos can be found in section 3.2.2.
2. *The JONSWAP spectrum represents the waves*
   The JONSWAP spectrum is a wave spectrum originally made to represent the North Sea. XBeach1D uses this spectrum for the wave input. More elaboration on the relation to Kloosterbos can be found in section 3.2.4.2.
3. *No trees on top of the levee*
   The effects of trees on top of the levee are not included in XBeach1D, whilst these are present at Kloosterbos. In this study, the effect of trees on top of the levee is not further considered, but is shortly mentioned in the discussion.
4. *No saturated soil*
   XBeach1D does not include soil saturation. This could be more relevant in the case of a river situation, due to the long duration of high water events. This assumption is further discussed in the discussion.

3.2.4. Input parameters XBeach1D

To obtain the erosion profile of the levee at Kloosterbos, the input parameters are necessary. In MorphAn, the input parameters can be defined in four different sections, which are further explained on the next pages:

- Profile of the levee
- Wave characteristics
- Water level
- Material characteristics

3.2.4.1. Profile of the levee

For the XBeach1D model, a 1D cross-section of the levee profile was required for the weakest link at Kloosterbos. The levee profile was obtained from the AHN3 database and imported in XBeach1D, see Figure 12. The output of the wave characteristics is given at around 100m distance from x=0m. Therefore, the profile has been extended from x=-55m to x=-100m. In reality the foreland rises a small bit and is bumpier. However, for now it is considered as flat, as the foreland is not uniform alongshore of the Kloosterbos levee. Further, this low and flat foreland will give more conservative erosion profiles, since this deeper foreland will overestimate the wave impact and therefore increase the erosion impact (Roode, Maaskant, & Boon, 2019; Steetzel, Groeneweg, & Vuik, 2018).

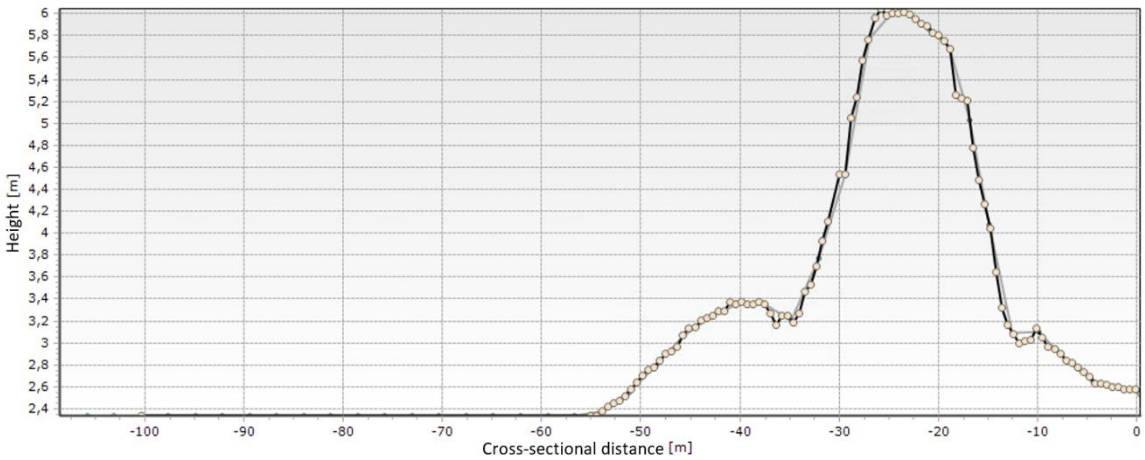


Figure 12: The profile of the weakest link at Kloosterbos implemented in XBeach1D

Moreover, the minimum and maximum grid size can be determined. Within the given grid size range, XBeach1D calculates a grid size to work with. This is the distance over which XBeach1D will calculate the morphodynamical and hydrological processes. For dunes at the coast, the grid cells can have a larger distance, since the dunes are often wide. However, at Kloosterbos the river levee is rather small (~15m width). Therefore, a smaller grid size range is used to calculate with. The minimum grid cell distance is set to 0.1m (default is 2m) and the maximum set to 0.5m (default is 60m).

3.2.4.2. Wave characteristics

XBeach1D works with the JONSWAP spectrum for the wave input. This is a wave spectrum originally designed for the North Sea (Joint North Sea Wave Project). The wave spectrum is a representation of the wave sequence in the water. This is visualised in Figure 14, where a combination of waves leads to the combined wave sequence in the water. In Figure 13, a wave spectrum is shown for i.e. JONSWAP. On the y-axis, the wave energy is shown (the contribution to the eventual wave sequence) and on the x-axis the frequency of the wave ($1/T_p$) (Michel, 1999).

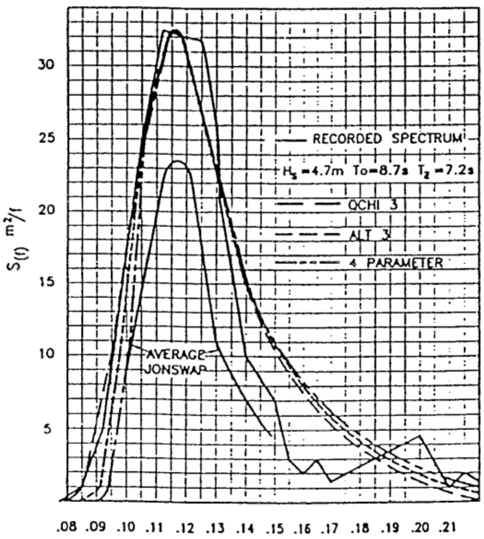


Figure 13: Wave spectrum, with the average JONSWAP spectrum (Michel, 1999)

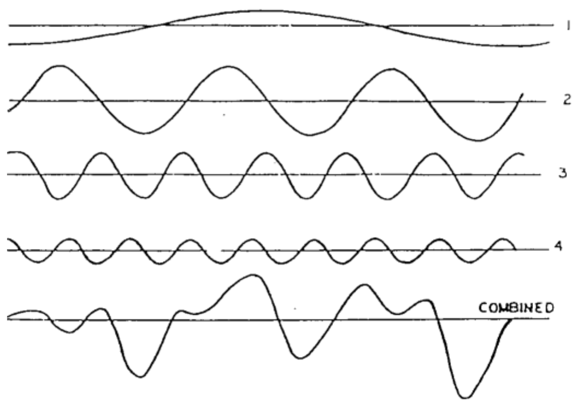


Figure 14: Combination of the waves leading to the actual wave composition (Michel, 1999)

The JONSWAP is a model originally designed to model waves on the North Sea. The question arises whether the JONSWAP spectrum is also valid for the Kloosterbos situation. On rivers, wind induced waves play an important role in wave propagation(RWS, 2017a). The JONSWAP spectrum has been validated for these wind induced waves (Babanin & Soloviev, 1998) and has also been used and validated for smaller scale shallow lakes of ~1.5m deep (Homoródi, Józsa, & Krámer, 2012; Liu, 1987). The Kloosterbos situation is rather similar to these shallow lakes, since there are almost no currents present and the foreland is also shallow for high waters (1-3m deep). The fetch length is the main limiting factor, since the lakes have long fetches (4-15km). However, at Kloosterbos the fetch length is also already rather large (~3.5km, Figure 20, page 23). Therefore, the JONSWAP spectrum is considered to be applicable for the Kloosterbos situation.

XBeach1D requires five input parameters for the JONSWAP spectrum. The significant wave height (H_s) and peak period (T_p) are obtained from the wave models. The directional spreading coefficient is normally used in 2D and is kept on its default value (10,000). The wave angle is kept perpendicular on the levee (default), since perpendicular waves have most wave energy on the levee to exert (van der Meer, 2002). Further, perpendicular waves are also used as for the GEBU failure mechanism, since these proved to give conservative outcomes for levees with grass and clay revetment (RWS, 2019). The last parameter is the peak enhancement factor (γ). This factor determines the peakedness of the JONSWAP wave spectrum. Often, when there is no data available for the wave spectrum, the value γ =3.3 is taken (Grainger, Sykulski, Jonathan, & Ewans, 2021). Therefore, γ has been set to 3.3 for the Kloosterbos situation. The sensitivity of this parameter is evaluated in section Sensitivity analysis4.2.3 to obtain more insight on the effect of this parameter on the erosion profile.

For the GEBU failure mechanism, the levee is tested for a 12 hour duration with the peak storm conditions (RWS, 2019). However, the storm situation would occur differently in reality. The storm would have a shorter peak period with more time available to build up and cool down from this peak storm situation (Gautier & Groeneweg, 2012). However, taking this 12 hour peak storm situation is considered in WBI2017 to be on the conservative side (RWS, 2016). For the Kloosterbos situation, the levee will be tested for a 12 hour peak storm situation.

3.2.4.3. Water level

The water level is also an input needed for the XBeach1D model. It is possible to describe a tide for the water level (water level fluctuation). The water levels in the upper river area (Kloosterbos) are dominated by discharge and also influenced by storm surge effects of the IJsselmeer. The storm surge effects are incorporated in the hydraulic boundary models of the WBI2017(RWS, 2017a). The duration of a high water event in the upper river area (including Kloosterbos) can hold up from days to weeks (RWS, 2017a). Since the model is tested for a period of only 12 hours, the water level is considered to be stagnant in the model.

3.2.4.4. Material characteristics

Finally, the material characteristics are needed. The material characteristics consist of the density of water ($\rho_w = 1000kg/m^3$), particle density of the sediment (ρ_s) and the median grain size (D_{50}). Below, the particle density and median grain size are further elaborated upon.

Particle density of the sediment (ρ_s)

Particle density is defined as the density of the solid particles in a soil sample (Blake, 2008, p. 504). For the Kloosterbos situation, the particle density is yet unknown. The general particle density range among soils is 2550-2700kg/m^3 (Blake, 2008). A value of around $\rho_s = 2650$kg/m^3 is generally taken for sand (Niroumand, 2017; Blake, 2008). This is also the default value recommended by XBeach1D. Therefore, a particle density of 2650kg/m^3 is used. In section 4.2.3, the sensitivity of this parameter is investigated to obtain more insight on the effect of this parameter on the model results.

Median grain size (D_{50})

The median grain size (D_{50}) value was determined by looking at borehole measurements. These measurements indicate what type of sand (fine, coarse etc.) is present at Kloosterbos, with which an indication for the D_{50} value was made.

In Figure 15, the KLK (Kloosterbos Kerkhofdijk) borehole locations are shown, which are used to determine the D_{50} value at Kloosterbos. These borehole measurements were made by an external party during a reinforcement project (A. van Ruiten, personal communication, June 22, 2021). The weakest link is also shown, which is the levee profile for which the levee is tested (see section 4.2.1 for more elaboration). In this case, the D_{50} is needed for the levee material. The WVK borehole measurements at the clay levees are not considered, since these do not represent the Kloosterbos levee material. There are some borehole measurements rather close to the weakest link mentioned by TA at the end (i.e. KLK-647TA). These are carried out on the waterside slope of the levee and are only 1 meter deep. The KLK-649 borehole measurements reach a depth of 5 meter below ground level and therefore add valuable information for the deeper lying sand.

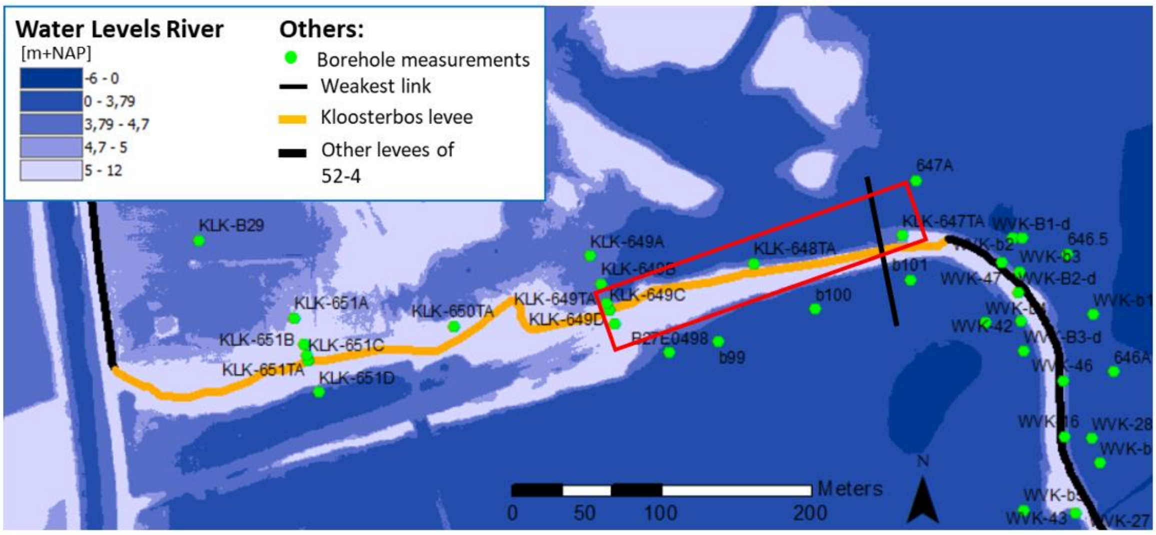


Figure 15: Borehole measurements used to determine D_{50}

Following from the KLK-647TA and KLK-648TA measurements, the sand is a mix of rather fine (150-210um) and rather coarse sand (210-300um). The measurements at KLK-649C/D also show a mix of rather fine and coarse sand for the deeper lying sand layers (2-5m beneath ground level), see Appendix C. Since the soil consists of an equal mixture of rather fine and coarse sand, the median grain size is set to $D_{50} = 210um$. This parameter is also investigated on the sensitivity due to the large variability in the sand type (150-300um).

As mentioned in section 1.1.3, the east of Kloosterbos might be man-made, causing potential differences in soil types along the Kloosterbos levee. However, the borehole measurements show the same soil type along the levee, indicating that the levee might be fully naturally formed.

3.2.5. XBeach1D errors (XBeach.exe)

Within the XBeach1D environment, errors came up during the simulations. For several simulations, XBeach1D stopped computing results when reading and writing wave mass flux and wave energy files. To obtain further insight on what was going on, the underlying XBeach.exe file available on https://oss.deltares.nl/web/xbeach was investigated (version Kingsday). It appeared that the XBeach.exe file worked with a few different internal settings than XBeach1D in Morphan (v1.9.0). This should not be the case, since it is stated in the MorphAn manual that MorphAn worked with the XBeach.exe model version Kingday (Deltares, 2020b). A few differences found, were in the factor of bedslope effect (facsl+) and the water depth at which the angle of repose is switched from dry to wet critical slope (hswitch+). The bedslope effect is in this case a factor reducing the upward sediment transport along the slope (M. de Ridder, personal communication, June 18, 2021).

In Figure 16, the differences in erosion profiles between XBeach.exe and XBeach1D are shown for the I_v situation (1/1,800,00 years) with H=5.5m+NAP, H_s=0.42m and T_p=2.43s after a 12 hour storm duration. The used MATLAB file can be found in Appendix F. There is a slight difference between the erosion profiles of XBeach1D and XBeach.exe. XBeach1D calculated 10.5% more erosion to happen compared to XBeach.exe. This difference is partly due to the different settings and the random wave generation. From the JONSWAP spectrum, random wave sequences are generated, which could cause these different erosion profiles.

The difference of erosion between the models is not further investigated. When the profile would be drawn more to scale, almost no difference in erosion profiles could be observed.

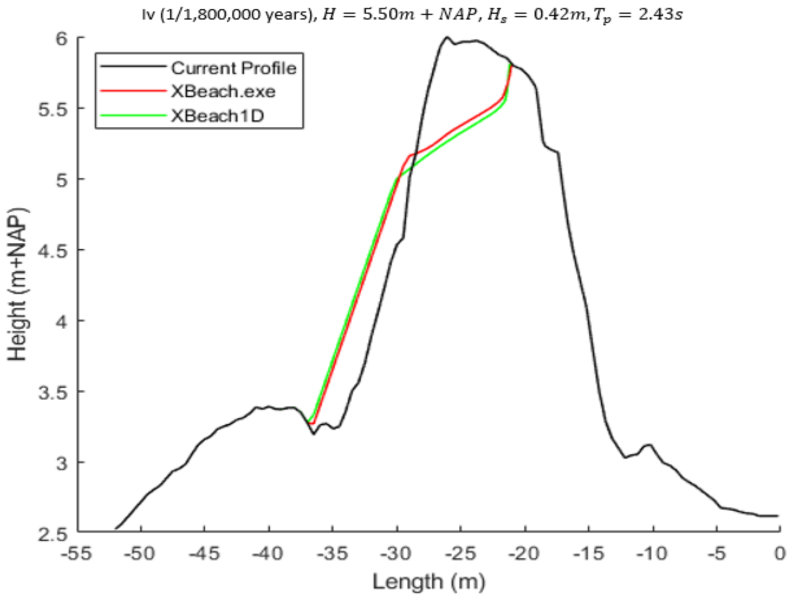


Figure 16: XBeach1D (MorphAn) vs XBeach.exe erosion profile for I_v (1/1,800,000 years)

The error in XBeach1D was located in wave energy and mass flux files. For every hour (3600s), the wave energy and mass flux were calculated. Differentiating the time span for which the wave files are made did not solve the problem. What did work was to define a .txt file stating only the static JONSWAP wave input (no time step definition). However, this was not possible to implement in XBeach1D. Therefore, the XBeach.exe version is used to calculate the erosion profiles, for which XBeach1D gives errors.

3.2.6. Sensitivity parameters

To obtain more insight on the effect of the XBeach1D input parameters on the final erosion profile, four parameters were tested on their sensitivity (Table 6). These parameters contained most uncertainty in their value, due to limited knowledge. For these (except grid size), the default values were used. It is therefore interesting to see what impact a change of value would give, if these parameters differed.

Table 6: Parameters sensitivity analysis

Parameter			Used value in XBeach1D	Range
Maximum grid size	-	[m]	0.5	0.5-60m
Peak enhancement factor JONSWAP	γ	[-]	3.3	$1<\gamma<5$
Particle density	ρ_s	[kg/m^3]	2650	$2550<\rho_s<2700$
Median grain size	D_{50}	[um]	210	$150<D_{50}<300$
Wave angle	-	[^{o}]	0	0-45

The sensitivity analysis was carried out for the safety category I_v (1/1,800,000 years, certainly meets alert value), with H=5.5m+NAP, H_s=0.42m and T_p=2.43s. This is one of the situations for which the Kloosterbos levee is tested (Appendix D). This situation was chosen, since it caused much erosion, to which the impact of each parameter could be observed more clearly.

The sensitivity of each parameter is determined by focusing on the differences in eroded volume of the levee [%]. The situation with the 'Used values' in Table 6 is set as base profile. In Figure 17, the corresponding erosion profile is shown for the base profile. The eroded volume for this profile is equal to 3.77m^3.

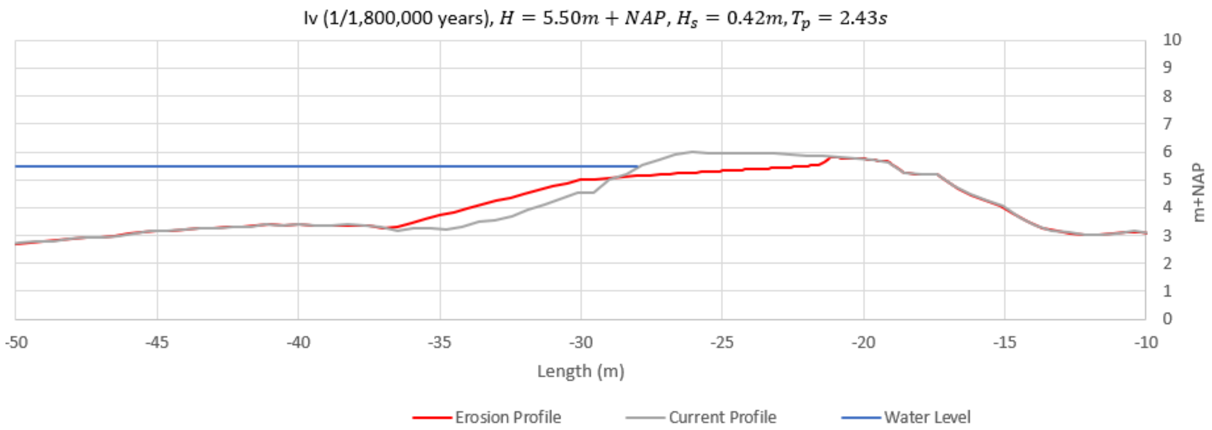


Figure 17: Base Erosion Profile used for the Sensitivity Analysis

The preliminary results of erosion of the outer slope at Kloosterbos implied that the levee was safe for waterside slope erosion (no measure needed). It was therefore most interesting to see whether any surplus erosion was caused by the possible deviation in parameter values.

The results of the sensitivity analysis are discussed in section 4.2.3.

4. Results

4.1. Wave characteristics at Kloosterbos

The wave model used for Kloosterbos is the Bretschneider equations. The Bretschneider equations provide the hydraulic boundary conditions for one output location. For Kloosterbos, eight output locations are available. In Figure 18, the output locations are shown for the Kloosterbos levee ('HR data'). As can be seen, the output locations are located 50-100m from the toe of the levee.

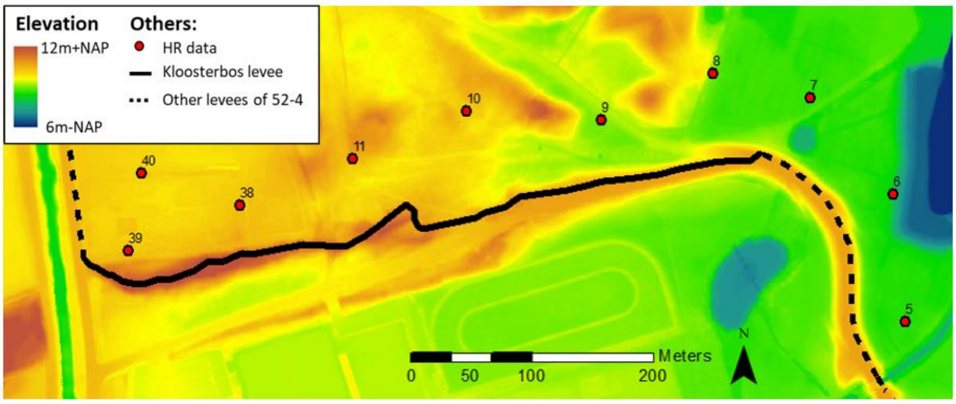


Figure 18: Output locations of the Bretschneider equations at Kloosterbos

To obtain the wave characteristics for XBeach1D, more information is needed about the probabilities for which the levee is tested. The levee at Kloosterbos has a lower threshold of 1/1,000 years and an alert value of 1/3,000 years and will be evaluated for the safety categories for a levee section as described in section 2.3.

4.1.1. Safety categories linked to Kloosterbos

As mentioned, the lower threshold is 1/1,000 years and alert value 1/3,000 years. Using equation 1 and Table 2 (page 8), the probabilities were determined for which the levee is tested. Equation 1 gives a lower threshold and alert failure probabilities at cross-section levels of $P_{eis,dsn,low}$ =1/20,000 years and $P_{eis,dsn,alert}$ =1/60,000 years ($\omega = 0.05$, $N_{dsn} = 1$). Filling in Table 2, gives Table 7 with the probabilities for each safety category at cross-section level for Kloosterbos.

This study applies the deterministic erosion model XBeach1D. Therefore, only the hydraulic boundary conditions will be fully probabilistically determined. For example, safety category I_v is tested with hydraulic boundary conditions for a 1/1,800,000 year situation.

Table 7: Safety categories for the Kloosterbos levee

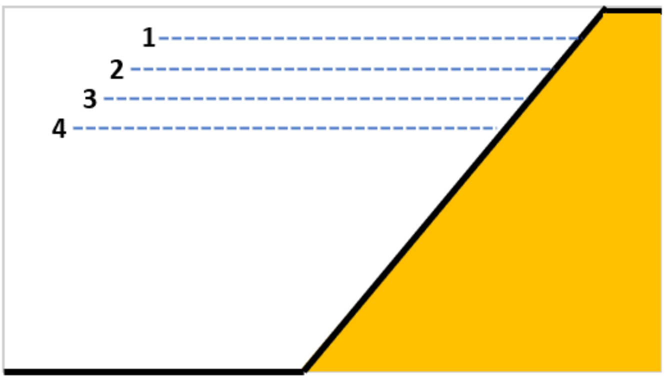
Category	Meaning categories	Category range [1/years] P_f Actual failure probability levee		
Iv	Certainly meets the alert value		$P_f <$	1/1,800,000
IIv	Meets the alert value	$1/1{,}800{,}000$	$< P_f <$	1/60,000
IIIv	Meets the lower threshold and possibly the alert value at cross-section level	$1/60{,}000$	$< P_f <$	1/20,000
IVv	Possibly meets the lower threshold at cross-section level or lower threshold	$1/20{,}000$	$< P_f <$	1/1,000
Vv	Does not meet the lower threshold	$1/1{,}000$	$< P_f <$	1/33
VIv	Certainly does not meet the lower threshold	$1/33$	$< P_f$	

4.1.2. Wave characteristics at Kloosterbos

At Kloosterbos, several output locations were available to determine the wave characteristics. The wave characteristics are used from output location 8 in Figure 18 (previous page). This is the output location closely located near the weakest link at Kloosterbos, for which the levee is tested.

The wave characteristics are dependent on the water level. This means that for hydraulic boundary conditions of 1/20,000 years, combinations of wave and water levels will lead to the 1/20,000 years situation. For example, extreme water levels with common waves could lead to a 1/20,000 year situation, just like common water levels with extreme waves(RWS, 2017a).

For the Kloosterbos hydraulic boundary conditions, all categories (I_v, II_v, etc.) have been given a combination of water levels with corresponding wave characteristics. This is done for the 1/10 years water level adding 0.5m to the water levels up to the maximum water level (for the lower threshold this is the 1/20,000 years water level). The water level for the 1/10 years situation is tested as lowest water level, since lower water levels might not even reach the high foreland at Kloosterbos. In Figure 19, the water level and wave combinations are shown more clearly for category III_v at output location 8, where all combinations lead to a 1/20,000 year situation. If the levee withstands all wave and water level combinations for the safety category, then the levee corresponds to that safety category or higher.



1: $H = 5.49m + NAP;\ H_s = 0.08m;\ T_p = 1.05s$
2: $H = 5.00m + NAP;\ H_s = 0.25m;\ T_p = 2.13s$
3: $H = 4.50m + NAP;\ H_s = 0.39m;\ T_p = 2.42s$
4: $H = 4.00m + NAP;\ H_s = 0.44m;\ T_p = 2.66s$

Figure 19: Wave and Water level combinations for III_v (1/20,000 years) at output location 8, Kloosterbos

For GEBU, the combination is used with the highest wave to test the levee for (number 4 in Figure 19). This is since it is about eroding of the grass and clay upper layer, which mainly happens due high wave impacts ('t Hart, De Bruijn, & de Vries, 2016). For the Kloosterbos situation, this is different, since it is about how far the levee erodes inwards. In this case, the water level also plays a larger role due to the avalanching process (section 3.2.3). Thus, the waves and water levels both play an important role in erosion of the fully sandy levee, without one which dominates the process. Therefore, the levee at Kloosterbos is tested for all wave and water level combinations (with steps of 0.5m in water level).

Further, the main wave angle is calculated. In Figure 20, different fetches are shown for the Kloosterbos situation. The waves were calculated to enter from the northeast, due to the relatively long fetch length (~3.5km). The detailed wave and water level combinations can be found in Appendix D.

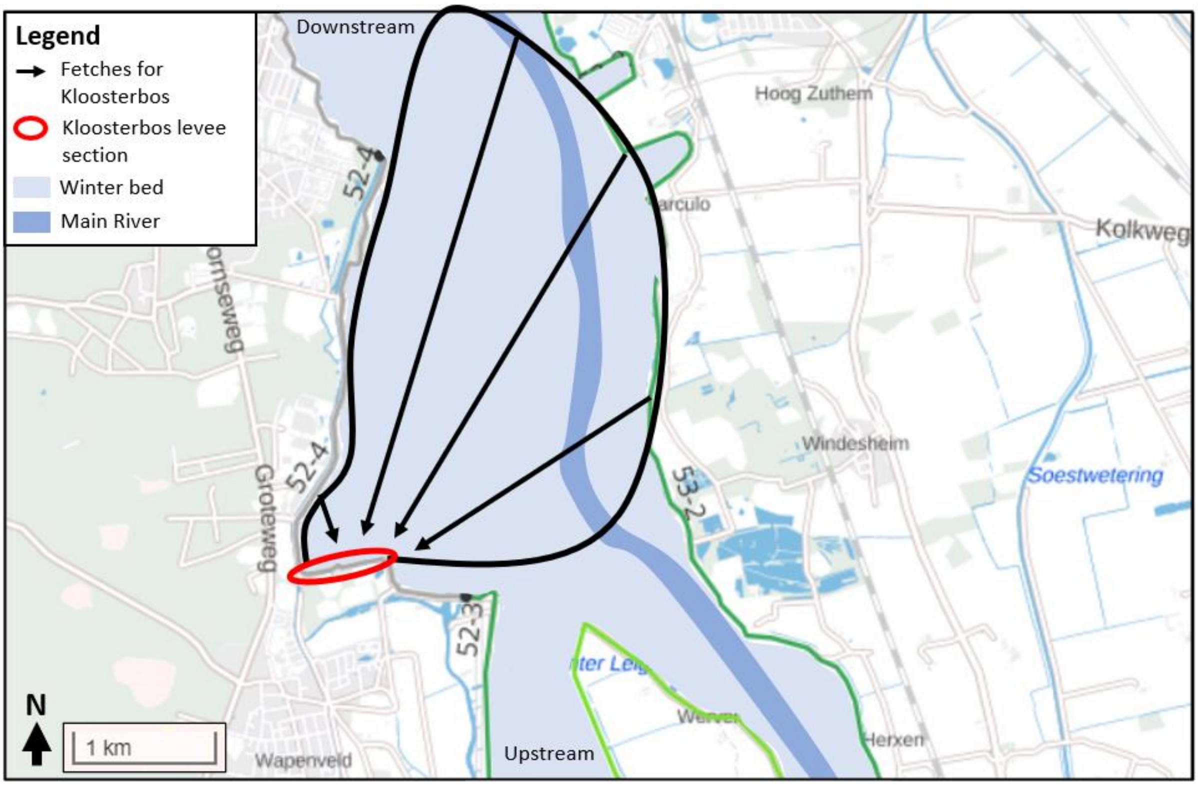


Figure 20: Bretschneider fetch directions at Kloosterbos

4.2. Erosion safety levee at Kloosterbos

With the wave characteristics known, the erosion safety of the levee at Kloosterbos is determined. A semi-probabilistic safety assessment has been carried out. The probabilities are only included in the determination of the water level and wave characteristics. To get to the eventual safety assessment, the following is needed:

- The weakest link (most critical levee profile for waterside slope erosion)
- Moment of failure (the levee reaches failure once the ultimate limit state is exceeded)
- Sensitivity analysis XBeach1D parameters

The outcomes of these will be combined to get to the overall waterside erosion safety assessment of the levee at Kloosterbos.

4.2.1. Weakest link at Kloosterbos

The weakest link is the most critical profile for which the waterside slope erosion safety at the Kloosterbos levee is calculated. The other levee profiles at Kloosterbos are equally safe or even safer than the weakest link. To determine the location of the weakest link, four critical locations were considered. The four locations are based on the levee width and fetch length in combination with the depth of the foreland. The thinner the levee, the faster it will fail due to waterside slope erosion. The larger the fetch length with deep foreland, the larger and more impactful the waves will be by the Bretschneiders equations. The possible alongshore sediment transport is not included in determining the weakest link, since the alongshore sediment transport is expected to be negligible (see section 3.2.2).

In Figure 21, the location of the four critical levee profiles at Kloosterbos are shown. Further, the inundated areas are shown for below NAP, the lower threshold water level (3.79m+NAP, 1/1,000 years), alert water level (4.7m+NAP, 1/3,000 years) and extreme water level (5m+NAP, 1/60,000 years).

Critical profile 4 is thinnest of all and thus weakest in terms of levee width. Further, the wave direction at the Kloosterbos levee is from the northeast. With this, critical profile 4 has the longest fetch with a deep foreland and therefore receives more damaging waves than the other profiles. Thus, critical profile 4 is identified as the weakest link for Kloosterbos.

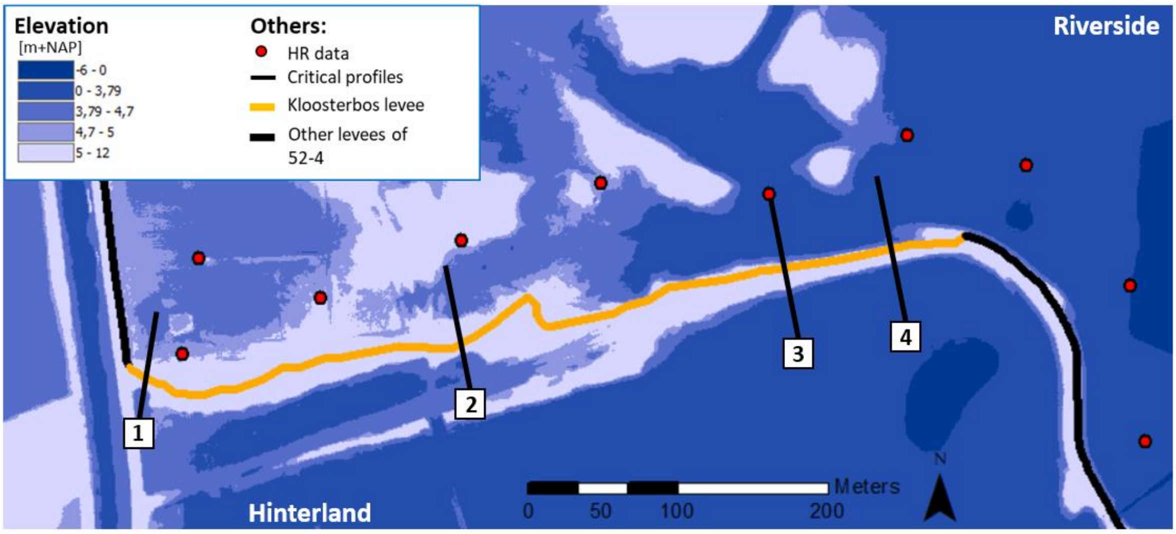


Figure 21: Four critical levee profiles at Kloosterbos

4.2.2. Moment of failure - ultimate limit state

It is important to define the moment when the levee is considered to have failed. The flood defence reaches failure at the moment on which the ultimate limit state is exceeded. The ultimate limit state is exceeded once there is a “loss of flood defence capacity in a levee segment causing the area protected by the levee segment to flood in such a way that fatalities or substantial economic damage occur” (Overheid, 2021). For GEBU, the ultimate limit state is exceeded (failure) when there is a point on the levee where the grass and clay layers are completely eroded and the sand core is reached (RWS, 2017b). The levee may still hold up for some more time, but is considered to have failed.

Several ultimate limit states have been considered for the Kloosterbos situation, such as the ultimate limit state used for micro- and macro-stability. For micro-stability (STMI), failure is reached when the crest has lowered (RWS, 2017b). For macro-stability of the landside slope (STBI), the levee reaches failure when the sliding plane cuts through the crest 1.5m or further seawards from the top of the landside slope (RWS, 2021). For the Kloosterbos situation, these two methods are not considered to be applicable. The fully sandy levee at Kloosterbos does not have a clear crest definition (not a trapezoidal form) and fails at waterside slope instead of landside slope. Further, the levee at Kloosterbos consists of sand without a clay/grass layer, whilst micro- and macro-stability assume a levee with both present. This means that the remaining levee profile at Kloosterbos has different mechanics and may fail earlier or later.

For the Kloosterbos situation, a boundary profile has been used as ultimate limit state. This is comparable with the minimum required profile as discussed in section 3.2.1.4, but the boundary profile is the minimum profile which should be left after erosion of the waterside slope has happened. If the boundary profile is exceeded by erosion, the levee is considered to have reached failure. In this study, the general geometry of the boundary profile in dune assessment is used: the waterside slope should be 1:1, the crest should be 3m wide and the landside slope should be 1:2 (see Figure 22, in red) (ENW, 2007).

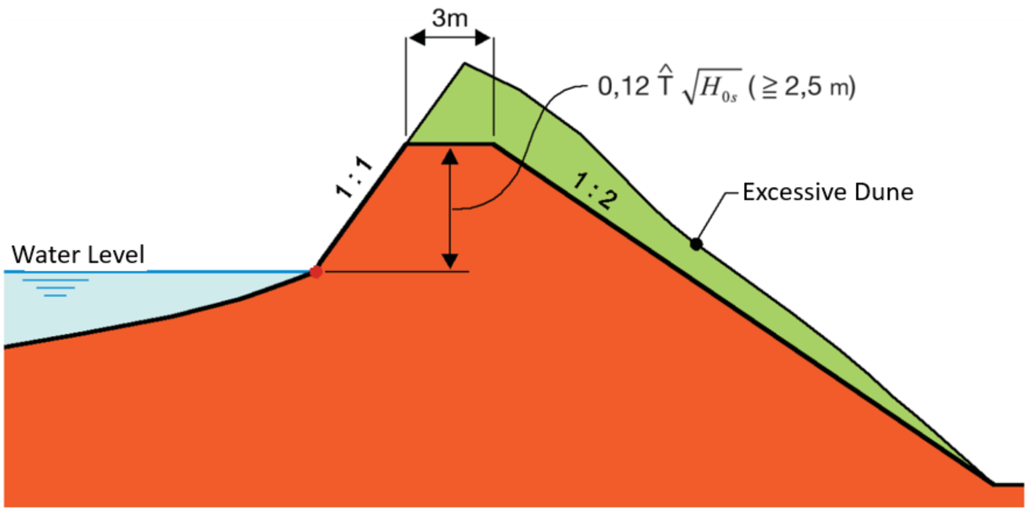


Figure 22: Boundary profile used in the current dune safety assessment (ENW, 2007)

For the Kloosterbos levee, this general geometry is kept. However, the minimum height of the boundary profile is taken differently, since the minimum crest height used in dune assessment (2.5m above water level) does not fit in the Kloosterbos levee. The minimum height of 2.5m was based for coastal dunes coping with substantially larger waves. Therefore, the height of this boundary profile may be lowered for Kloosterbos. The height of the boundary profile was determined by looking at the GEKB failure mechanism, see the approach of Waterschap Limburg (section 3.2.1.1). This focuses on wave overtopping and determines the minimum height for the levee needed to be safe for overtopping. This height is referred to as the HBN (RWS, 2017b). In this case, the HBN is calculated in Hydra NL for a waterside slope of 1:1 and a 1/1,000 year storm with a maximum allowable overtopping discharge of 0.1l/s/m, giving a minimum boundary profile height of HBN=5.5m+NAP.

The reason to take the HBN for a 1/1,000 year storm is as follows: the storm for the lower threshold at cross-section level (1/20,000 years) caused an initial erosion profile. After the storm, it should have a profile which could hold up for some more time. If this profile is made for another 1/20,000 years storm, the boundary profile is overdesigned (too conservative). However, a profile for a 1/10 year storm may fail instantly. In this study, a boundary profile for the lower threshold (1/1,000 years) is considered to be sufficient.

Further, a limited overtopping discharge is taken of 0.1l/s/m, which is also used for GEKB for levees with grass revetments (RWS, 2019). There is no limited discharge [l/s/m] set for sand revetments. However, in TAW 2002, an overtopping discharge of 0.1l/s/m is used as limited discharge for designing levees having a sand layer with poor grass revetment. In this case, the 0.1l/s/m is taken as valid assumption to use for the boundary profile. If the levee is in reality more susceptible for overtopping erosion the boundary profile might still be able to resist for a slightly weaker storm (i.e. 1/750 years), which is considered to be sufficient.

The waterside slope of the boundary profile (1:1) may be considered steep for a sandy levee, since the angle of repose of sand is generally smaller than 45 degrees (Shoji, Imamura, Nakamura, & Noguchi, 2019). However, dune erosion profiles often result with a waterside slope of 1:1 or steeper (ENW, 2007; Berard, Mulligan, Da Silva, & Dibajnia, 2017). Further, a 1:1 waterside slope is also generally used in dune erosion models as critical avalanching slope (van Rijn, 2013; Deltares, 2020b). Therefore, a boundary profile with a remaining 1:1 slope is also considered to be a valid assumption.

The resulting boundary profile is shown in Figure 23, within the weakest link at Kloosterbos.

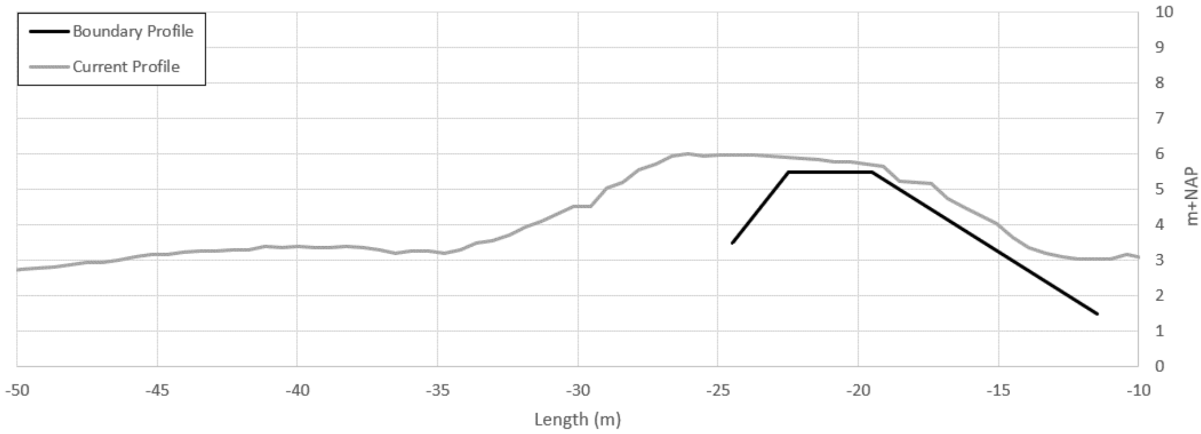


Figure 23: Boundary profile embedded in the weakest link (current profile) at Kloosterbos

4.2.3. Sensitivity analysis

For the XBeach1D model, a sensitivity analysis was carried out to see the effect of the median grain size (D_{50}), maximum grid size, peak enhancement factor (γ), particle density (ρ_s) and wave angle. It is most interesting to see any surplus erosion happening, since the preliminary results implied that the levee was safe for erosion and did not need a measure. The wave angle parameter caused substantially more erosion when it changed value. The other parameters caused minimum surplus erosion as further elaborated below.

4.2.3.1. Sensitivity wave angle

The only parameter causing substantial surplus erosion is the wave angle. The perpendicular waves (0^o) caused substantially less erosion than angled waves (at 45^o, Figure 24). The angled waves actually caused the levee to completely erode. An explanation is that angled waves change the hydrodynamical processes and add alongshore current, causing potential suspension and therefore extra erosion (B.W. Borsje, personal communication, 2015). Important to mention is that M. de Ridder (personal communication, June 2, 2021) noted that the function to incorporate wave angle is not calibrated nor validated. Therefore, the results may be more off regarding reality.

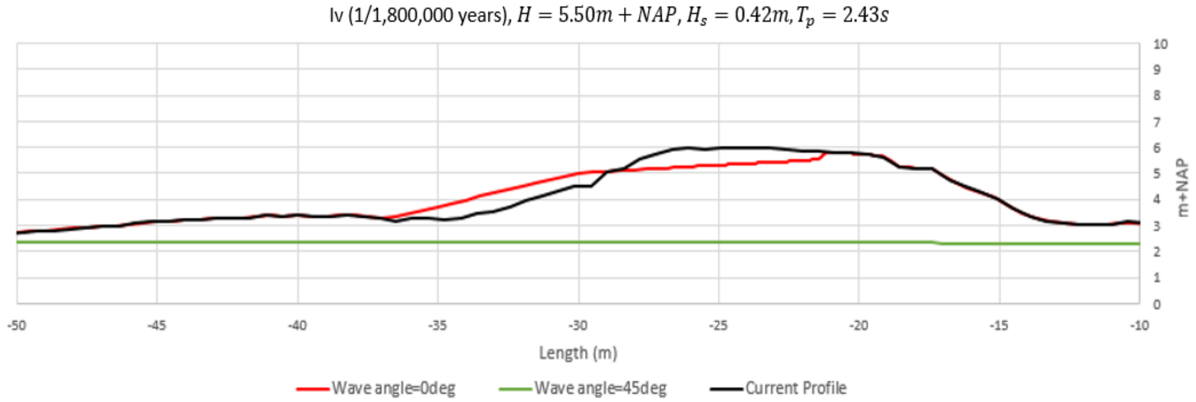


Figure 24: Sensitivity wave angle 0-45 degrees with respect to the levee normal

For standard levees, perpendicular waves were said to cause more erosion. Reasoning for this, is that a grass revetment is present at a standard levee, improving the cohesive strength of the upper levee layer (Wegman, 2020; Hinostroza Garcia , 2007). In this case, alongshore erosion plays a smaller role, than the erosion caused by wave impact ('t Hart, De Bruijn, & de Vries, 2016). The wave impact is larger for perpendicular waves (van der Meer, 2002) and therefore causes more erosion for standard levees.

At Kloosterbos, the wave angle is 42^o with respect to the levee normal (from the northeast). It is open for discussion whether the model estimates erosion well for these angled waves. However, the wave angle is still used for the waterside slope erosion safety assessment (section 4.2.4).

4.2.3.2. Summary of sensitivity parameters

In Figure 25, the sensitivities of all parameters are shown. Detailed explanation on the sensitivities of each parameter can be found in Appendix E. The wave angle is not shown in this figure, since the whole levee was eroded. It can be seen that the grid size is most sensitive for the tested parameters and substantially reduces the erosion rate, due to a rougher representation of the Kloosterbos levee. Further, differences in all parameters mostly lead to less erosion, meaning that the current erosion profile estimated by XBeach1D is already rather conservative. As mentioned, the parameters causing surplus erosion were most interesting. Thus, the wave angle will be included to assess the levee, since substantially more erosion is computed to happen with angled waves.

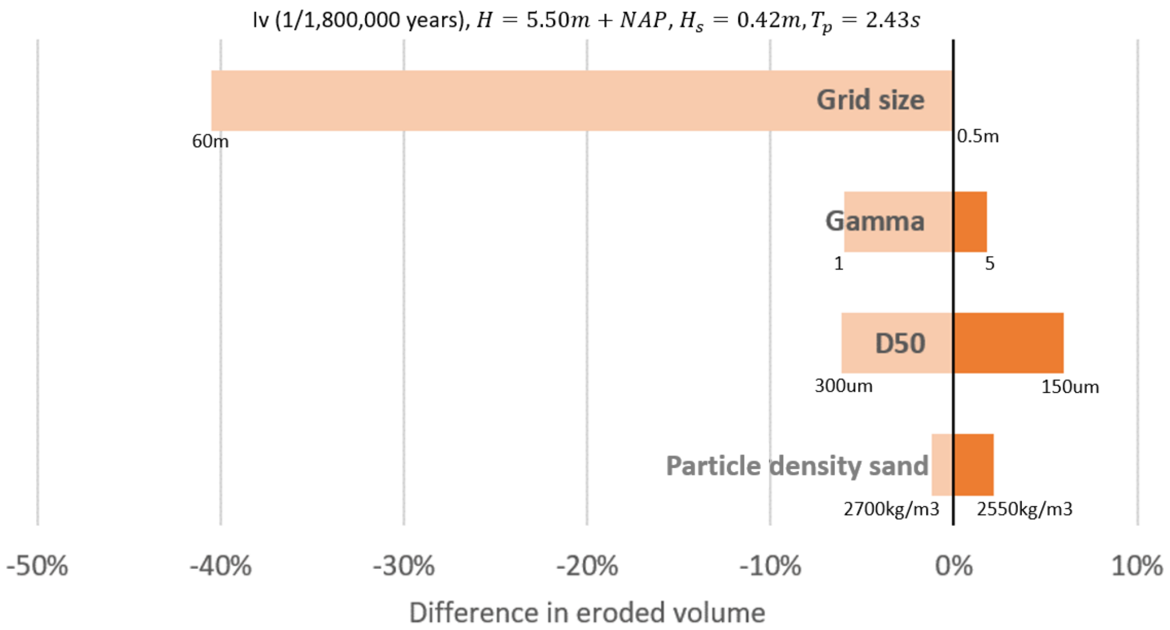


Figure 25: Sensitivity of the main parameters in XBeach1D

4.2.4. Erosion safety of the levee at Kloosterbos

With the weakest link, boundary profile and input parameters known, the waterside slope erosion safety at Kloosterbos can be determined. From the sensitivity analysis it became clear that XBeach1D is sensitive for surplus erosion to happen due to the wave angle. At first, the XBeach1D models were ran for the situation with perpendicular waves, after which the critical erosion profiles were tested by including the wave angle at Kloosterbos (42^o). With this, a safety category is assigned to the Kloosterbos section. However, there appeared to be some errors occurring in XBeach1D (see section 3.2.5). These could be resolved in the underlying XBeach.exe model, but not within XBeach1D in MorphAn. Therefore, half of the calculations had to be carried out in XBeach.exe.

XBeach1D and XBeach.exe calculated the erosion profiles for the different safety categories (I_v, II_v, III_v, etc.), which were compared to the current and boundary profile. If the resulting erosion profile cut through the boundary profile, failure was reached. In Figures 26, 27 and 28, the erosion profiles for the I_v (1/1,800,000 years), II_v (1/60,000 years) and III_v (1/20,000 years) situation are shown on a scale of 3:1 (x:y). From these, it came out that the levee at Kloosterbos does not reach failure for the II_v and III_v situation, but does reach failure for the I_v category. As mentioned, the XBeach1D model is sensitive for the wave angle. Therefore, the safety categories II_v and III_v were also tested with the wave angle present at Kloosterbos.

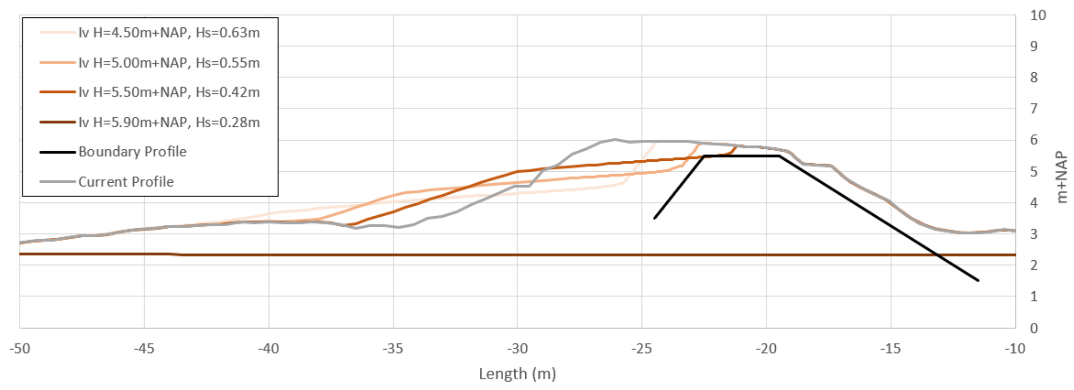


Figure 26: Erosion profiles weakest link for I_v (1/1,800,000 years) at Kloosterbos

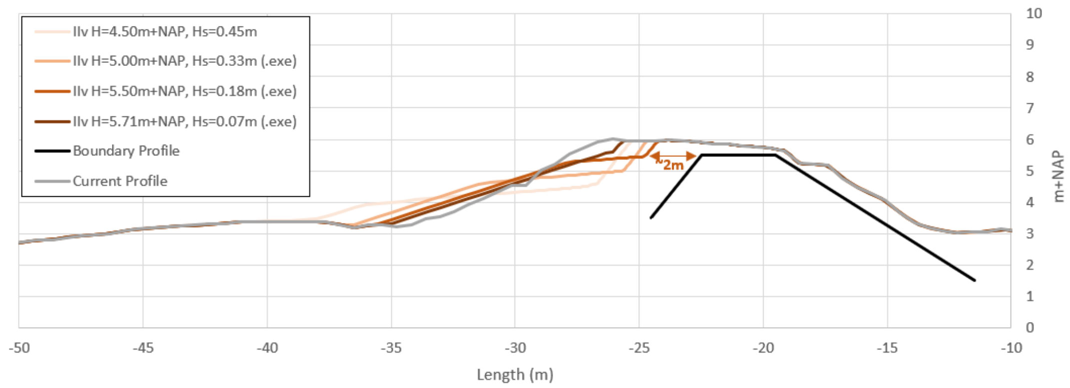


Figure 27: Erosion profiles weakest link for II_v (1/60,000 years) at Kloosterbos

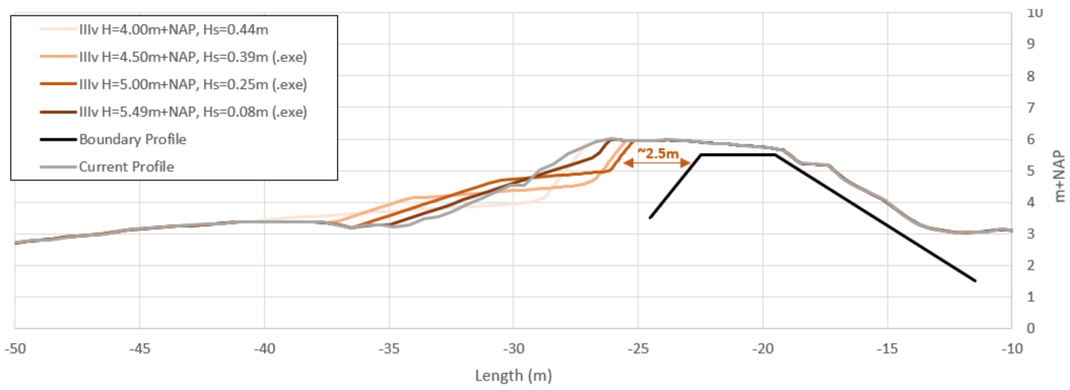


Figure 28: Erosion profiles weakest link for III_v (1/20,000 years) at Kloosterbos

The figures above are for waves perpendicular to the levee (0^o from the levee normal). The wave characteristics at the Bretschneider output location 8 also include a wave angle of 42^o with respect to the levee normal (northeast). The most inland erosion profiles are taken from the above figures for II_v and III_v to test the wave angle for. The impact of wave angles is visualized in Figures 29 and 30 (next page).

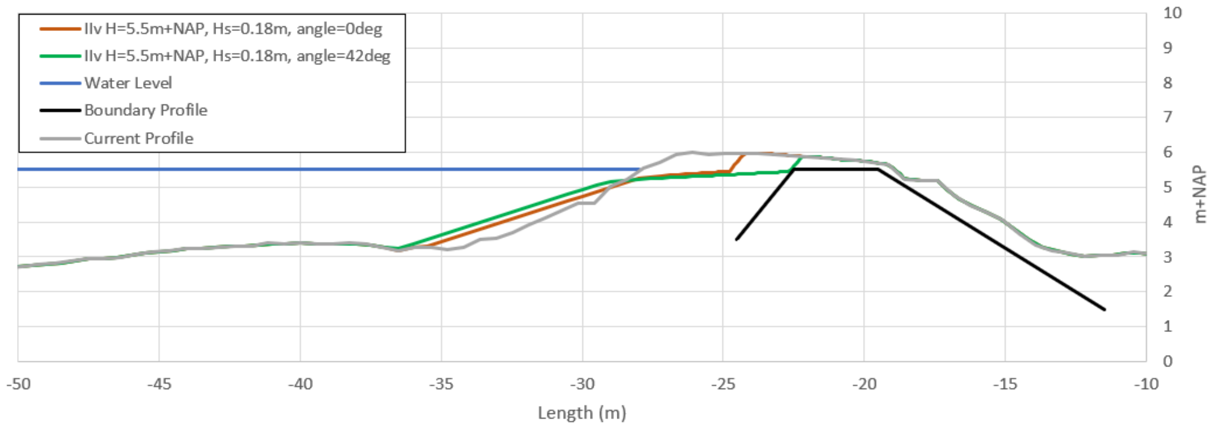


Figure 29: Critical erosion profile weakest link with angled waves for II_v (1/60,000 years) at Kloosterbos

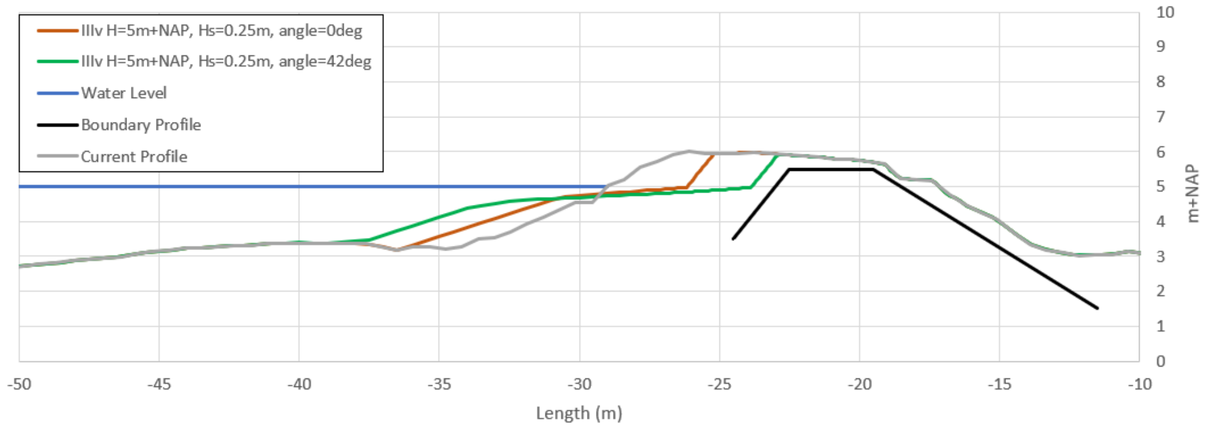


Figure 30: Critical erosion profile weakest link with angled waves for III_v (1/20,000 years) at Kloosterbos

For safety category III_v there is still ~0.5m left before failure is reached (Figure 30). However, for II_v the erosion profile nearly touches the boundary profile (Figure 29). The 0.1m margin for II_v is considered too small, due to the model (parameters) uncertainty, and failure is reached for II_v. The 0.5m margin for III_v is considered to be sufficient. Therefore, the levee at Kloosterbos does comply to the III_v safety category.

5. Discussion

5.1. Theoretical implications

A few theoretical notifications were made in this study. Firstly, it was observed that the wave angle has a substantial impact on the resulting erosion profiles. Normally, perpendicular waves are used for the safety analysis of for a levee with grass and clay revetment (RWS, 2019). However, for a fully sandy levee it came out that angled waves have more erosion impact on the levee. When using a wave angle, the hydrodynamical processes changes and alongshore current is added, causing potential suspension and therefore extra erosion (B.W. Borsje, personal communication, 2015). Important to mention is that the extension to include the wave angle in XBeach1D has not been calibrated nor validated (M. de Ridder, personal communication, June 2, 2021).

Moreover, questions arose from water authorities about the sand composition. XBeach1D works with the particle density and neglects saturation of the sand and bulk density. For the saturation of the sand, high water events in the upper river area (including Kloosterbos) can hold up from days to weeks (RWS, 2017a), meaning the levee may have become completely saturated before the storm hits. A saturated sand layer has less shear strength than unsaturated sand (Mizutani, Nakagawa, Yoden, Kawaike, & Zang, 2013), causing more erosion to happen than for unsaturated sand. This could indicate that more erosion will happen than XBeach1D calculated. For coastal modelling, the saturation of the sand is not included, possibly due to the short high water event (4-6 hours).

Further, the bulk density is not included in XBeach1D, whilst research shows that more dense sands significantly reduce erosion (Overton, Pratikto, Lu, & Fisher, 1994). Dunes continuously deform and therefore often consist of dilute sands. The Kloosterbos levee is exists for over centuries, with minimum observed changes in geometry (A. Verboom, personal communication, May 12, 2021). The sand has therefore settled over a long period, increasing the high bulk density. Therefore, the Kloosterbos levee may be less susceptible for erosion than XBeach1D currently has modelled.

5.2. Practical implications

XBeach1D is only applicable for 1D erosion processes. It is therefore important to investigate whether 2D erosion processes play an important role for the studied section. If so, XBeach1D is not applicable and other 2D erosion models should be investigated. In the Kloosterbos situation, the river current is considered to be negligible, due to the high winter bed with vegetation lying rather far from the summer bed (~1.5km). Therefore, a 1D environment was considered to be applicable. However, fully sandy levees lying rather close to the summer bed with no high and vegetated foreland probably encounter currents from the river, causing XBeach1D not to be applicable.

Within XBeach1D, errors came up during the simulations. XBeach1D stopped computing results when reading and writing wave mass flux and wave energy files. In XBeach1D, the boundary conditions are defined every 3600 seconds. The underlying XBeach.exe file (version Kingsday) works when defining static boundary conditions. The error could not be solved within XBeach1D itself. Therefore, XBeach.exe should be used to calculate the erosion profiles, for which XBeach1D gives errors.

There is a minimum required profile mentioned in Handreiking Constructief ontwerpen (den Adel, Barends, de Groot, & Heemstra, 1994), see section 3.2.1.4. If this profile fits in the current levee profile, the levee can be considered safe for erosion. This could be used together with XBeach1D as methodology for assessing fully sandy levees. The minimum required profile could function a simple test and XBeach1D could be used as detailed test. However, the minimum required profile stated in Handreiking Constructief ontwerpen should be revised, since it may be outdated (25 years old).

5.3. Future research

Within this research, some assumptions have been made. To obtain a more precise and better understanding of the levee's safety, the following should be researched further:

- Make the transition from the 1D to the 2D XBeach model
- Include uprooting of the trees on top of the levee
- Include the foreland effects on the wave characteristics
- Base the boundary profile on more failure mechanisms (instead of only GEKB)
- Quantify the uncertainty in the XBeach model more precisely

From the 1D to 2D XBeach model

The XBeach1D model is made for systems being mostly uniform alongshore and where no alongshore currents are present. In this study, the simplified 1D model was taken for Kloosterbos and estimates erosion rather well. However, a 2D model would be able to simulate the possible alongshore currents together with the alongshore erosion happening at the levee more precisely. Further, the levee section as a whole can be evaluated instead of only the weakest link. This could help indicating where measures are needed. Moreover, a 2D model could be applied for other fully sandy levees where 2D processes play a larger role.

Include uprooting of the trees

One of the effects not taken into account, is the uprooting of the trees on top of the levee at Kloosterbos. Trees mainly decrease the stability of the levee. Especially in presence of a storm for which a levee is tested, they might uproot and fall over. This will create a large hole in the levee (Waterschap Rivierenland, sd), possibly leading to more critical erosion profiles than currently stated.

Include the effect of the foreland on the wave characteristics

In this study, the foreland effects are not fully included in the wave characteristics. The wave impact is therefore overestimated and causes more erosion. Including the elevation of the foreland more precisely would give a better representation of the actual waves at Kloosterbos. The vegetation could also be included, but this does give a dependence on the quality of vegetation. Hence, it is recommended to only include elevation of the foreland more precisely for the wave characteristics.

Test the boundary profile for more failure mechanisms

Currently, the boundary profile is mainly based on the general geometry used for dune assessment. The height is based on GEKB. However, other failure mechanisms are not included in determining this minimum profile. The boundary profile might suffice for GEKB, but could fail for other failure mechanisms. It could also be that the boundary profile is currently too conservative and that it can be smaller and still suffice. This could prevent unnecessary levee measurements or an overestimation of the levee's erosion safety.

Quantify the uncertainty in XBeach

The uncertainty in the model is not thoroughly investigated in this study. This is important to know, in order to assess whether the boundary profile is cut or not. The levee at Kloosterbos was considered to fail for the II_v (1/60,000 years) safety category due to the surplus erosion caused by the wave angle. A 10cm layer of sand layer was left before the boundary profile would be cut (Figure 29, page 29). The uncertainty in the model parameters and model structure can overcome this 10cm margin quickly. Therefore, the levee was considered to have reached failure. However, the uncertainty of the erosion profiles by wave angle are rather large due to the lack of calibration and validation of the wave angle in XBeach1D. Investigating the model (parameters) uncertainties more thoroughly would lead to a better estimation of the erosion profiles and therefore, a better safety assessment.

6. Conclusion

In this research, it was investigated how the waterside slope erosion safety at the Kloosterbos levee could be quantitatively assessed. This was done by answering three research questions. Below, the answer to each question is further examined.

6.1. Which wave model can be used to determine the wave characteristics at Kloosterbos without including the vegetated foreland?

In this study, two wave models were compared, namely the SWAN model and Bretschneider equations. Both are used in the current safety assessment methodology 'WBI2017'. From the two models, it was found that the Bretschneider equations was most applicable for Kloosterbos. The Bretschneider equations already provided a database with wave characteristics, which were sufficient to use for the erosion model (XBeach1D). The SWAN does provide more detailed results, but there was no database available for Kloosterbos. Setting up such a SWAN model would cost a large amount of time (too much for this study).

Thus, the Bretschneider equations can be used to determine the wave characteristics at Kloosterbos.

6.2. Which erosion model can be used to provide a safety assessment for erosion of the waterside slope at the levee at Kloosterbos?

The erosion model found to be applicable for Kloosterbos is XBeach1D (version: Kingsday) in the supporting module MorphAn (version: 1.9.0). This model was compared to four other erosion models (DUROS+, D++, equations of Klein Breteler et al. and DUROSTA). These four erosion models mainly contained limitations regarding the wave characteristics, making it not applicable for the Kloosterbos levee.

Further, three external water authorities (Waterschap Limburg, Waterschap Rijn en IJssel and WDOD) shared their methodology on assessing a fully sandy levee. These were mainly qualitative safety assessments and not further used. Yet, the methodology of Waterschap Limburg was partly used in determining the boundary profile at Kloosterbos.

Concluding, the erosion model XBeach1D can be used to provide a safety assessment for erosion of the waterside slope at the levee at Kloosterbos.

6.3. What is the waterside slope erosion safety of the levee at Kloosterbos?

With the wave characteristics known, the erosion safety of the levee at Kloosterbos was determined. A semi-probabilistic safety assessment was carried out, where the probabilities were located in the water level and wave characteristics.

The sensitivity analysis showed that most parameters are not sensitive and that angled waves caused substantial surplus erosion, compared to perpendicular waves. Important to mention is that the extension of XBeach1D coping with angled waves is not calibrated nor validated. However, the surplus erosion caused by angled waves is used in the eventual erosion safety assessment.

The flood defence reaches failure at the moment on which the ultimate limit state is exceeded. For the Kloosterbos situation, a boundary profile is defined as ultimate limit state. The boundary profile is the minimum profile left after erosion of the waterside slope has happened. If the boundary profile is cut through by erosion, the levee is considered to have failed. The general geometry used in dune assessment is also used for the boundary profile at Kloosterbos: a waterside slope 1:1, a dune crest of 3m wide and a landside slope of 1:2. The height of the boundary profile was determined in Hydra NL for the failure mechanism GEKB with a 1:1 waterside slope and 1/1,000 years storm with a maximum allowable overtopping discharge of 0.1l/s/m, giving a minimum boundary profile height of 5.5m+NAP.

It was found that the Kloosterbos levee would reach failure for I_v (1/1,800,000 years), but not for II_v (1/60,000 years) with perpendicular waves. The most inward erosion profiles were tested for the wave angle present at Kloosterbos (42^o) for II_v (1/60,000 years) and III_v (1/20,000 years). This resulted in failure for II_v, but not III_v. It is open for discussion whether the erosion profiles by wave angle are representative due to the lack of calibration and validation of the wave angle in XBeach1D. In this study, the error in erosion profiles by wave angle is not further investigated and it is assumed that they are valid.

Thus, the levee section at Kloosterbos complies to safety category III_v (1/20,000 years) and therefore meets the lower threshold. No levee measures have to be done by Vallei en Veluwe at Kloosterbos, when looking at the waterside slope erosion failure mechanism.

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Appendix A – Example of the concealed levee at Kloosterbos

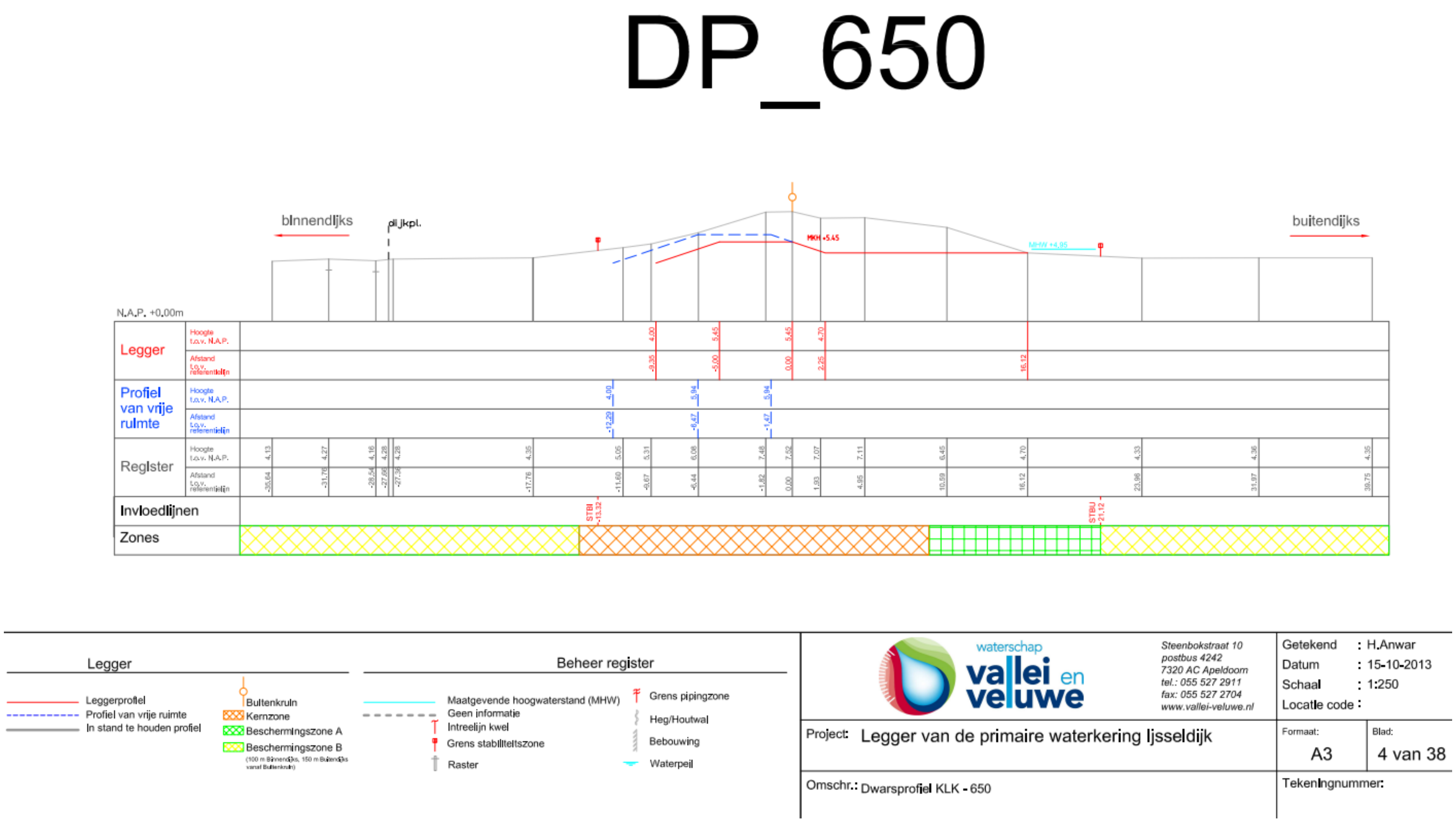

Figure 31: One of the cross sections of the levee at Kloosterbos

Appendix B – Considered erosion models

DUROS+ and D++

DUROS+ and D++ are the two models currently used for the safety assessment of dunes (Boers, Achtergronddocument toetsschema duinafslag, 2015). A detailed description of the models can be found in the TRDA2006 (ENW, 2007). They both describe the erosion processes of the coastal dunes and have an erosion safety assessment embedded in the model. The structure of both models is the same, but DUROS+ focuses on coastal regions with deep seawaters and D++ on shallow waters (i.e. Wadden Islands).

Applicability DUROS+ to Kloosterbos

It seems that the DUROS+ model may be applicable, since the Kloosterbos situation is fully consisting out of sand and there are levee profiles available at Kloosterbos to use for this model. Further, the model has been validated for erosion of dunes (den Heijer, Baart, & van Koningsveld, 2012).

However, there are some limitations to the model, looking at the Kloosterbos situation. Firstly, the DUROS+ model is used for dunes where the hydraulic boundary conditions are determined at ~20m depth (ENW, 2007). At Kloosterbos, the water level at deeper water ranges at ~5 meter depth. This is substantially shallower, meaning no representative boundary conditions can be derived for DUROS+. Secondly, DUROS+ is designed for storms with a duration of 4 to 6 hours (ENW, 2007). In the assessment of river levees, the levees should withstand 12 hours of peak storm conditions for erosion of the waterside slope (van Rinsum, Bisschop, Delhez, Lansink, & van Ruiten, 2020). This required storm duration is substantially longer than can be simulated by DUROS+. Lastly, DUROS+ is valid for peak wave periods (T_p), which lie in between 12-20 seconds (ENW, 2007). For Kloosterbos, the peak wave period will be a maximum of 3 seconds, which is substantially below the minimum required 12s.

Concluding, DUROS+ does simulate dune erosion rather well, but is not applicable for the smaller scale levee at Kloosterbos.

Applicability D++ to Kloosterbos

The model D++ mostly works the same as DUROS+, but with D++ it is possible to define the hydraulic boundary conditions at shallow water. This option was originally added for the Wadden Islands (Boers, 2012). In case of the Kloosterbos situation, this makes D++ more applicable than DUROS+.

However, D++ still has the limitation for the wave period (T_p ranges $12 - 20s$) (Boers, 2012). This seems contradictory, since the hydraulic boundary conditions should be adapted for shallower waters where wave periods could be smaller. Further, D++ is based on the underlying DUROS model of 1984 (Boers, Technisch Rapport Duinwaterkeringen en Hybride Keringen 2011, 2012), which was only validated for the shorter 4-6 hour storms (TAW, 1984). Therefore, D++ can neither simulate 12 hour storm durations.

Thus, D++ is also not considered applicable for the Kloosterbos levee section. It has, however, less limitations than DUROS+ to use.

Het Bekledingsmodel (equations of Klein Breteler et al.)

Another erosion model considered, is Het Bekledingsmodel. This is a model made for sandy levees, with a clay layer and grass revetment. In this model, the erosion of the sand core of the levee is also considered (HKV, 2019). For the Kloosterbos situation, the most interesting part of the model lies in the simulation of the erosion of the sand core. The sand core erosion is simulated in this model using the equations of Klein Breteler et al. (2012). The equations of Klein Breteler et al. (2012) describe the sand core erosion over time for a levee when the clay layer has eroded. These equations are valid for a wave height between 0.7m to 3m (Klein Breteler, Capel, Kruse, Mourik, & Kaste, 2012).

Applicability equations of Klein Breteler et al. to Kloosterbos
The equations of Klein Breteler et al. (2012) are not considered to be valid for the Kloosterbos situation. At the Kloosterbos situation, the waves are all below 0.7m height (~0.3m). Further, the model is validated for levees with a clay layer, instead of sand layer. The erosion profile of the sand core could therefore be different for the fully sandy levee at Kloosterbos, since parts of clay were observed to fall instead of natural slumping of the sand at the seaside (Klein Breteler, Capel, Kruse, Mourik, & Kaste, 2012).

XBeach

XBeach was originally developed to assess hurricane effects on sandy beaches. It was funded by the U.S. Corps of Engineers. Following from this, the model became also funded by the Dutch Public Works Department. This resulted in a model which was more extended and validated for dunes and urbanized coasts to provide the possibility for a dune safety assessment (Deltares, 2020). Currently, XBeach is a widely internationally used model, supported by the consortium of the EU, UNSESCO-IHE, Delft University of Technology, Deltares and the University of Miami (Deltares, sd). Further, the model is validated for 60 international erosion tests (van Dongeren, sd) and planned to be used in the next assessment round as new safety assessment tool for dunes, instead of DUROS+ and D++ (van Geer & Roos, 2020).

Applicability XBeach to Kloosterbos
XBeach has, in contrast to the other erosion models, the possibility to include the wave characteristics present at Kloosterbos. No specific limitations have been set for the peak period range and location of the hydraulic boundary conditions. Further, XBeach has been validated for small- and large-scale dune erosion experiments (Deltares, 2015), making it probable that XBeach approximates the waterside slope erosion rather well for the relatively small fully sandy levee at Kloosterbos.

However, the model is designed for the coastal region. Therefore other processes may play differently in the river (i.e. currents), causing the model to be less valid. This problem is then again present for all dune erosion models.

DUROSTA

The DUROSTA model is a predecessor of DUROS+ and was previously used for the safety assessment of dunes (Steetzel H. , 1992). The model is currently replaced by DUROS+ and has therefore become outdated. However, DUROSTA works with peak periods up to 12s and has no restriction of larger wave period ($12 - 20s$).

Applicability DUROSTA to Kloosterbos
DUROSTA is a predecessor of DUROS+ and may be outdated to use for Kloosterbos. DUROSTA has been compared to other models like XBeach in previous research (van Santen, Steetzel, van Dongeren, & van Thiel de Vries, 2012), which showed that XBeach gives more accurate erosion profiles.

DUROSTA has the advantage to work with peak periods up to 12s and not the higher range of 12-20s in DUROS+ and D++. The higher peak period range in DUROS+ and D++ were required when it became clear that wave periods at sea often extend the 12s range (Baaren, 2007). From this, it can again be seen that the knowledge embedded in the DUROSTA software was more limited.

In short, the model DUROSTA has the possibility to include shallow hydraulic boundary conditions, but gives less accurate erosion profiles than XBeach and may be too outdated for Kloosterbos to use.

Appendix C – Sand types at Kloosterbos

In Table 8, the sand types at different borehole locations are shown beneath the ground level. The location of the borehole measurements can be found in Figure 32. The classifications used in Table 8 are based on NEN5104. From Table 8, it can be deduced that the soil consists of an equal mixture of rather fine (150-210um) and rather coarse sand (210-300um).

Table 8: Borehole measurements near the weakest link

Depth [m]	KLK-647TA	KLK-648TA	KLK-649C	KLK-649D
0-1	ZMF	0-0.2m ZMF 0.2-1m ZMG	ZMG	ZMG
1-2			ZMG	ZMF
2-3			ZMG	ZMF
3-4			4-4.5m ZMG, 4.5-5m ZMF	
4-5			ZMF	

* ZMF = Rather fine sand (150-210um), ZMG = Rather coarse sand (210-300um)

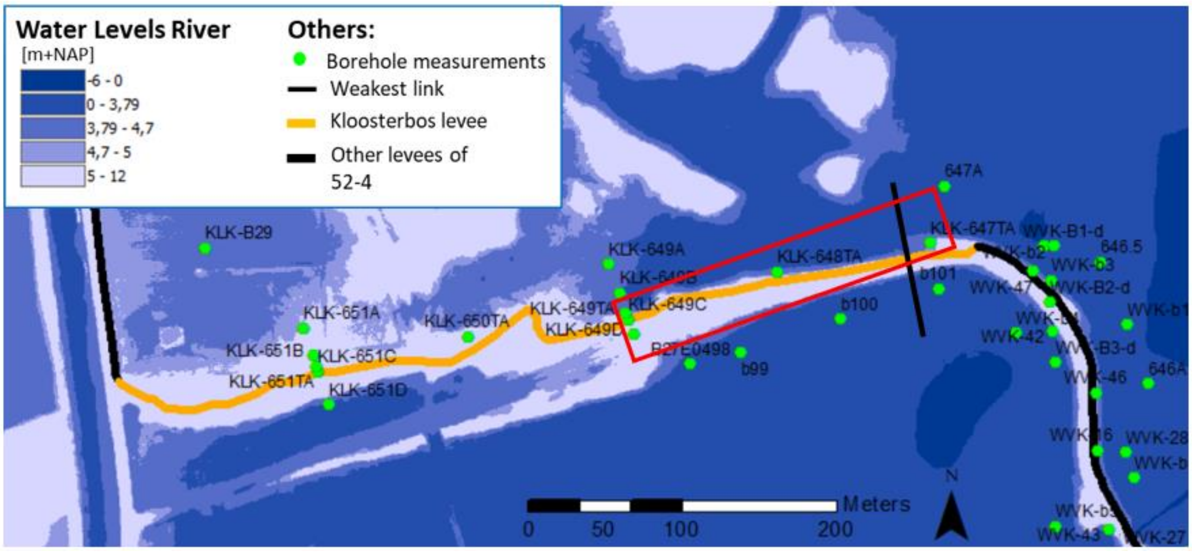

Figure 32: Borehole measurements used to determine D_{50}

Appendix D – Wave characteristics at Kloosterbos

Table 9: Wave characteristics at Kloosterbos for output location 8

Y HB location (RD) [m]	Dam used?	Foreland geometry included?	Safety category	Water level [m+NAP]	Wave height (Hs) [m]	Wave period (Tp) [s]	Wave angle w.r.t. levee normal [°]	Wave angle w.r.t. north [°]
495043.438	no	no	I_v	5.9	0.28	2.28	42	45
495043.438	no	no	I_v	5.5	0.42	2.3	42	45
495043.438	no	no	I_v	5	0.55	2.56	42	45
495043.438	no	no	I_v	4.5	0.63	2.86	42	45
495043.438	no	no	I_v	4	0.69	2.91	42	45
495043.438	no	no	I_v	3.76	0.69	2.96	42	45
495043.438	no	no	II_v	5.71	0.07	1.04	-138	225
495043.438	no	no	II_v	5.5	0.18	1.91	42	45
495043.438	no	no	II_v	5	0.33	2.11	42	45
495043.438	no	no	II_v	4.5	0.45	2.43	42	45
495043.438	no	no	II_v	4	0.48	2.74	42	45
495043.438	no	no	II_v	3.76	0.51	2.68	42	45
495043.438	no	no	III_v	5.49	0.08	1.05	-138	225
495043.438	no	no	III_v	5	0.25	2.13	64.5	67.5
495043.438	no	no	III_v	4.5	0.39	2.42	42	45
495043.438	no	no	III_v	4	0.44	2.66	64.5	67.5
495043.438	no	no	III_v	3.76	0.43	2.68	42	45
495043.438	no	no	IV_v	4.77	0.08	1.06	-138	225
495043.438	no	no	IV_v	4.5	0.22	2.02	64.5	67.5
495043.438	no	no	IV_v	4	0.28	2.12	64.5	67.5
495043.438	no	no	IV_v	3.76	0.32	2.1	42	45
495043.438	no	no	V_v	3.79	0.08	1.06	-138	225
495043.438	no	no	V_v	3.76	0.08	1.06	-138	225

Appendix E – Sensitivities in XBeach1D

Sensitivity maximum grid size

Within XBeach, it is possible to differentiate the maximum grid size to work with. XBeach1D calculates a grid size to work with from the given grid size range. The default maximum grid size is 60m, which led to XBeach1D working with grid sizes of 2-3m. This would give a coarser estimation of the erosion profile, than working with smaller grid sizes. In this case, smaller maximum grid sizes of 1m and 0.5m are used to see how much the erosion profile differs. It would be preferable to work with larger grid sizes to minimize running time, but a grid size precise enough to estimate erosion rather well. The minimum grid size is set to 0.1m for all three tests.

In Table 10, the results show that the grid size does have a substantial impact on the erosion volumes. The difference between grid size 0.5m and 1m is rather small (~3%), but going to a coarser grid size of ~2m gives a substantially different erosion volume. For the eventual erosion safety assessment, the maximum grid size was set to 0.5m. However, the maximum grid size could also have been taken as 1m.

Table 10: Sensitivity grid size

	Max. grid = 1m	Max. grid = 0.5m	Max. grid = 60m
Eroded volume [m^3]	3.67	3.77	2.25
Difference in eroded volume [%]	-2.7%	-	-40.5%

Sensitivity peak enhancement factor

The peak enhancement factor in the JONSWAP wave spectrum determines the peakedness of the spectrum. For this study, it was set to 3.3 due to comparable studies also considering this value (Liu, 1987; Homoródi, Józsa, & Krámer, 2012; Astier, Astruc, Lacaze, & Eiff, 2012). The range of γ is 1-5 in XBeach1D. The actual waves in the river may be more corresponding with γ=1, leading to a Pierson-Moskowitz spectrum. The Pierson-Moskowitz spectrum does not include for fetch, resulting in a broader wave spectrum peak (Benitz, Lackner, & Schmidt, 2015). Further, the river could be corresponding to a higher peak enhancement factor of $\gamma = 5$ (maximum in XBeach), due to the difference in wave mechanics in rivers. XBeach1D was run for $\gamma = 1$, $\gamma = 3.3$ and $\gamma = 5$.

In Table 11, the results are shown. It can be deduced that the peak enhancement factor is little sensitive. Especially, $\gamma = 1$ gives a larger deviation. However, as mentioned, the focus of the sensitivity analysis is on the potential surplus of erosion, which is in this case negligible (1.8%).

Table 11: Sensitivity peak enhancement factor

	$\boldsymbol{\gamma = 1}$	$\boldsymbol{\gamma = 3.3}$	$\boldsymbol{\gamma = 5}$
Eroded volume [m^3]	3.55	3.77	3.84
Difference in eroded volume [%]	-6.0%	-	+1.8%

Sensitivity particle density (ρ_s)

The particle density was assumed to be equal to the average soil particle density of 2650kg/m^3. However, this value may differ due to a different particle composition at Kloosterbos. The particle density for soil lies in a range of 2550-2700kg/m^3 respectively (Blake, 2008). XBeach1D was run three times, for $\rho_s = 2550kg/m^3$, $\rho_s = 2650kg/m^3$ and $\rho_s = 2700kg/m^3$.

In Table 12, the results are shown. It can be deduced that the particle density has a negligible sensitivity to the results (~2%).

Table 12: Sensitivity particle density

	$\boldsymbol{\rho_s = 2550kg/m^3}$	$\boldsymbol{\rho_s = 2650kg/m^3}$	$\boldsymbol{\rho_s = 2700kg/m^3}$
Eroded volume [m^3]	3.86	3.77	3.73
Difference in eroded volume [%]	+2.2%	-	-1.2%

Sensitivity median grain size (D_{50})

The median grain size was determined by looking at the upper sand layer (1m) at the weakest link and deeper lying sand 200m west of the weakest link. An equal mixture of rather fine (150-210um) and rather coarse sand (210-300um) was found, giving an average 210um for D_{50}. However, the actual soil composition could differ at the weakest link. In this case, XBeach1D was tested for $D_{50} = 150um$, $D_{50} = 210um$ and $D_{50} = 300um$.

From Table 13, it can be deduced that the median grain size is slightly sensitive. It is considered not to be negligible when comparing it to the two previous mentioned sensitivities regarding surplus erosion.

Table 13: Sensitivity D_{50}

	$\boldsymbol{D_{50} = 150um}$	$\boldsymbol{D_{50} = 210um}$	$\boldsymbol{D_{50} = 300um}$
Eroded volume [m^3]	4.00	3.77	3.54
Difference in eroded volume [%]	+6.0%	-	-6.1%

Appendix F – MATLAB file for XBeach.exe (Kingsday)

```
clear all, clc

%% importing netCDF data
zb=ncread('filename.nc','zb'); %import time dependent zb data

fid= fopen('x.grd','r');            %import the x grid
line=fgetl(fid);                    %get line from table
X=str2num(line);                    %convert to vector

%% plotting dune profile changes
figure(1), clf(1), hold on
axis([-60 0 1 7])

cur=plot(X(:),zb(:,:,1),'k','linewidth',1);   %plot initial and end profile
afsl=plot(X(:),zb(:,:,end),'r','linewidth',1);%plot initial and end profile

legend([cur,afsl],'Current profile','Erosion
profile','location','northwest')
xlabel('Length (m)')
ylabel('Height (m+NAP)')
title('Dune erosion profile XBeach.exe')
hold off

%% plotting bottom level changes
change=zb(:,:,1)-zb(:,:,end);
figure(2), clf(2), hold on
plot(X,change,'r')
legend('XBeach.exe')
xlabel('Length (m)')
ylabel('\Delta height (m)')
title('Dune erosion height difference XBeach.exe')
hold off

%% Writing to excel
cl=strings([500,3]);                    %used matrix to clear the excel file
Data=[X',zb(:,:,end),zb(:,:,1)];        %Matrix with results
xlswrite('Results',cl,'R_Xbeach','A2') %clear the excel file 'R_XBeach'
xlswrite('Results',Data,'R_Xbeach','A2')%write to the excel file 'R_XBeach'
```