How vegetated foreshores can contribute to limiting dike dimensions of sea dikes

A case study into the assessment and design procedure of including the quantitative effect of the foreshore in the flood defence system

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Marit Lambers
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Cover picture: Photo of the WGD pilot study in the Ems-Dollard estuary via:
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Master Thesis
University of Twente
Department of Water Engineering and Management

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Preface

In front of you is the final piece missing before I was able to finish my master of Civil Engineering and Management. For the last six years, I spend studying civil engineering subjects at the University of Twente where the last two years I got specialised in River and Coastal Engineering subjects while executing my master's program. Here, I discovered that I like to work with models while keeping practice in mind. Accordingly, I got very enthusiastic when the graduating assignment of Sweco came in sight and got shaped such that I was able to combine my interest in modelling while also making a link with practice. Moreover, my time at Sweco gave me the opportunity to get a feeling of how work was being done in a consultancy company and got to know all different kinds of projects which I could be working on with my profession. I had a great time at Sweco and enjoyed getting to know the company as well as the people.

While conducting my master thesis assignment for the last six months, I spend working from the office of Sweco in de Bilt with colleagues, from home, all alone or with my closest friends, and at the university on the famous 'Cornelis Lely square'. Working on those different locations with different people helped me to stay motivated throughout the whole process. Doing so, I want to take the opportunity to thank all those colleagues and friends for sharing their experiences and helping me by discussing all different kinds of topics.

Besides expressing my appreciation to my colleagues and friends, I want to thank my daily supervisors Pim and Jos. Pim was my daily supervisor from the University of Twente and always took the time to discuss certain outcomes of my study and gave me a good direction and stimulation for my thesis. Jos was my daily supervisor of Sweco who always was interested in the progress I had made. Moreover, Jos shared a lot of great ideas such that the work I was doing fitted well within the work Sweco was doing as well as explained where the possible bottlenecks would occur. Finally, I want to thank Bas, the head of the committee, for being there at the key moments in my gradating process. During those meetings, Bas would be very sharp and discreet such that my work was adding something to the academic world, while also his interest, which is almost more like a passion for building with nature solutions, was incorporated.

Besides all the people stated above, I want to close off by thanking my boyfriend and parents for having the patience to listen to me while I was trying to share about the progress I had made or the struggles I had to face. I don't know how much you understood but you were always interested.

I hope you will enjoy reading this thesis.

Marit Lambers

Enschede, July 2022

Summary

Due to the inevitable effects of climate change, more frequent and more extreme storms will take place. In addition, Sea Level Rise (SLR) will be present such that the pressure on coastal flood protection increases. Moreover, since the magnitude of the effects of climate change is still uncertain, there is a great need for primary flood defences to be adaptive to effectively respond to the changing boundary conditions. Due to the self-organising behaviour of ecosystems, there is more often looked for solutions that work together with nature resulting in Nature-based flood defences.

In this research, the Wide Green Dike (WGD) pilot study in the Ems-Dollard estuary in the Netherlands is used as an example of a Nature-based flood defence. For the WGD pilot study, 1 km of dike is being reinforced with a thick clay layer on the seaward side while also decreasing the outer dike slope to fulfil the safety standards against erosion. Besides the adaptive capacity of this reinforcement, compared to the traditional way of reinforcing with concrete, a wide vegetated foreshore is present in front of the dike. Due to the self-sustaining behaviour of this foreshore, caused by the enhancement of sedimentation by the presence of vegetation, the foreshore should be able to grow along with SLR. Moreover, due to the capacity of the vegetation to mitigate wave conditions by increased bottom friction, the vegetated foreshore lowers the hydraulic boundary conditions (= wave impact as well as wave conditions) such as the significant wave height. The above-described processes are visualised in Figure 1.

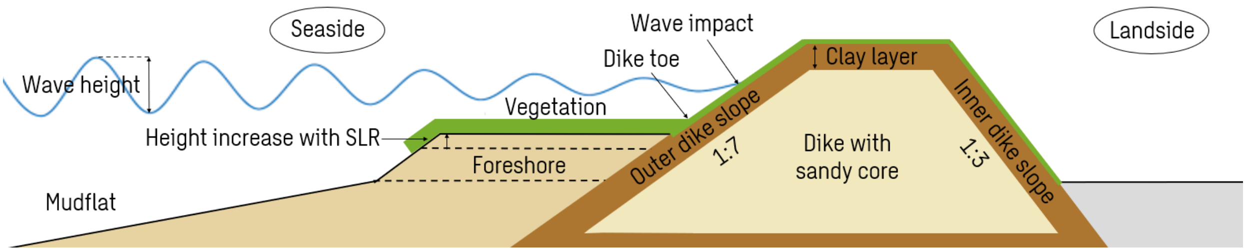


Figure 1. Visualisation WGD pilot study

By mitigating the hydraulic boundary conditions at the dike toe, the vegetated foreshore could limit the required dike dimension such that a more sustainable dike reinforcement arises. However, a qualitative analysis of the potential influence of the vegetated foreshore (of the WGD pilot study), wherein a link is made to the influence on the required dike dimensions, is still missing. Doing so, the aim of this study can be formulated as:

"*To quantify the potential influence of the (changing) foreshore of the Wide Green Dike on hydraulic boundary conditions and dike design and investigate the implementation of the foreshore potential in the assessment and design process of primary flood defences*"

After a Simulating Waves Nearshore (SWAN) model was set up for the study area, a sensitivity analysis was conducted to determine the sensitivity of the significant wave height at the dike toe when input conditions were changed. These conditions also concerned the foreshore dimensions and the method of the model to implement the effect of vegetation. The sensitivity analysis showed that the water depth on the foreshore has a greater influence on the significant wave height than the width of the foreshore (for the WGD transect). If the foreshore height increases, the water depth on the foreshore decreases due to which the effect of depth-induced wave breaking on mitigating waves increases. Moreover, by including vegetation in the method of the model indirectly, by increased bottom friction or explicitly, by including the vegetation properties, the significant wave height showed a significant decrease. After the sensitivity analysis, a scenario analysis was conducted wherein changes to foreshore dimensions were combined with an indirect method to include vegetation. Doing so, the potential influence of the foreshore could be visualised. Although different discussion points could be stated concerning the limitations of the model, the implementation of the vegetation and the sensitivity and the scenario analysis, it could be concluded that the foreshore could be of significant importance to lower the hydraulic boundary conditions at the dike toe. Hereby, the foreshore potential depends on the ability of the foreshore to pace with SLR and the presence of vegetation which causes increased bottom friction.

Once the influence of the foreshore dimensions and the vegetation implementation on the hydraulic boundary conditions were known, it still needed to be linked to their influence on the dike design. To do so, firstly the normative storm conditions had to be determined for the failure mechanism of wave overtopping (causing erosion of the dike crest and inner dike slope) and outer dike slope erosion (caused by wave impact). These conditions were determined with Hydra-NL whereafter implemented in the developed SWAN model. Due to the limitation of the SWAN model to only make a deterministic calculation, it was decided that the output of the SWAN model was scaled to the probabilistic Hydra-NL calculations. With the use of the scaled wave conditions, the significant wave height at the dike toe for wave overtopping and erosion could be determined for different scenarios. The significant wave height, other wave conditions (e.g. water level, wave length and wave period) and dike characteristics (e.g. slope and clay strength) were used in the calculation of the determination of the required dike height to fulfil the maximum overtopping discharge and required clay layer thickness to prevent erosion of the outer dike slope. Moreover, different dike slopes were used in the calculations to determine the effect of the outer dike slope on the required dike dimensions. When comparing the required dike dimensions for the standard scenario, while not taking into account foreshore height increase or the influence of vegetation (scenario 1), with the required dimensions when taking those processes into account (scenario 4), a required height decrease of 22 cm and clay layer thickness decrease of 31 cm arises for an outer dike slope of 1:5. As seen in Figure 2, the required dike dimensions significantly decrease when the outer dike slope is decreased.

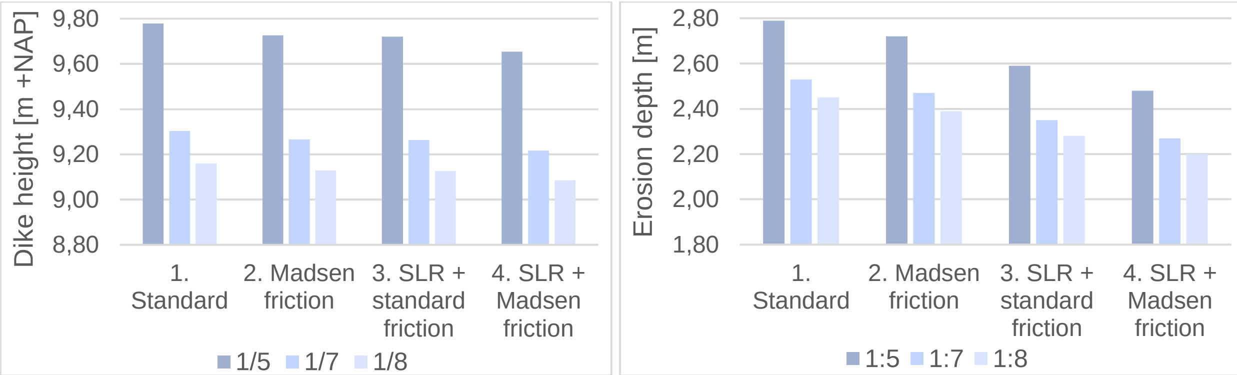


Figure 2. Required dike dimensions for different scenarios

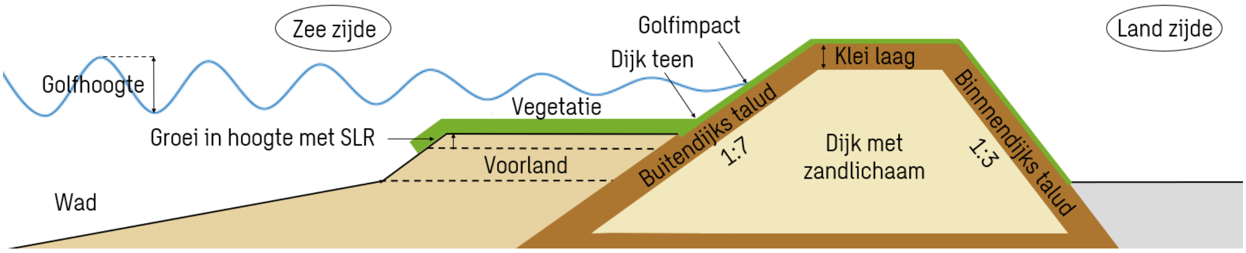
The above-given results demonstrate the importance to include the potential influence of the future state of the foreshore in the assessment and design procedure of a primary flood defence. To do so, different approaches have been proposed which could be implemented easily or with some effort in the current assessment and design procedure of a primary flood defence in the Netherlands. Within the current procedure, a Dam and Foreshore module (DaF) can be used in Hydra-NL which is designed to determine changes in wave conditions between the output location in Hydra-NL and the dike toe. However, since the DaF module is designed for the transformation of wave conditions for a maximum of 200 m, it has a limited ability to implement the foreshore potential in the current procedure. In other approaches, where the foreshore width is fictively increased in the DaF module or the output location in Hydra-NL has been moved towards the foreshore fringe, the disadvantage of the 1D calculation of the DaF module comes in sight. The final proposed approach is to conduct a separate SWAN model for a specific foreshore-dike system to calculate the transformation of the wave conditions over the foreshore with 2D calculations. However, the implementation of this approach is very time-consuming.

To conclude, the study was able to quantify the potential influence of the foreshore to mitigate wave conditions at the dike toe such that over-dimensioning of the flood defence could be prevented. However, the resilience of the foreshore to be able to pace with SLR and the potential of the vegetation to mitigate wave heights under extreme storms remain uncertain. If the foreshore shows trouble pacing with SLR, sedimentation can be enhanced with the implementation of manmade structures such as brushwood dams or a summer dike. Moreover, the preservation of good quality vegetation could play a big part in this. If this alone does not result in the designed sedimentation, Thin Layer Placement (TLP) could be used on the foreshore to increase the foreshore height. Said so, if an effort is being made regarding the enhancement of foreshore growth and the quality of vegetation, an effort of including the foreshore potential in the assessment and design procedure of primary flood defences in the Netherlands must not be left behind.

Samenvatting

Als gevolg van de onvermijdelijke effecten van de klimaatverandering zullen er vaker en extremere stormen voorkomen. Bovendien zal er sprake zijn van een stijging van de zeespiegel (SLR) waardoor de druk op de bescherming tegen overstromingen aan de kust zal toenemen. Aangezien de omvang van de effecten van klimaatverandering nog onzeker is, is er een grote behoefte aan adaptieve primaire waterkeringen om effectief te kunnen reageren op de veranderende randvoorwaarden. Vanwege het zelforganiserende gedrag van ecosystemen wordt vaker gezocht naar oplossingen die samenwerken met de natuur, resulterend in op de natuur gebaseerde waterkeringen.

In dit onderzoek wordt de brede Groene Dijk (BGD) pilotstudie in het Eems-Dollard estuarium in Nederland gebruikt als voorbeeld van een op de natuur gebaseerde waterkering. Voor de BGD-pilotstudie wordt 1 km dijk versterkt met een dikke kleilaag aan de zeezijde van de dijk terwijl ook het buitentalud van de dijk wordt verlaagd om te voldoen aan de veiligheidsnormen tegen erosie. Naast het aanpassingsvermogen van deze versterking, in vergelijking met de traditionele manier van versterken met beton, is voor de dijk een breed begroeide vooroever aanwezig. Door het zelfvoorzienende gedrag van deze vooroever, veroorzaakt door sedimentatie door de aanwezigheid van vegetatie, zou het voorland mee moeten kunnen groeien met de zeespiegel. Bovendien verlaagt het begroeide voorland de hydraulische randvoorwaarden zoals de significante golfhoogte doordat de vegetatie in staat is de golfcondities te temperen door een grotere bodemwrijving. De hierboven beschreven processen zijn gevisualiseerd in Figuur 1.



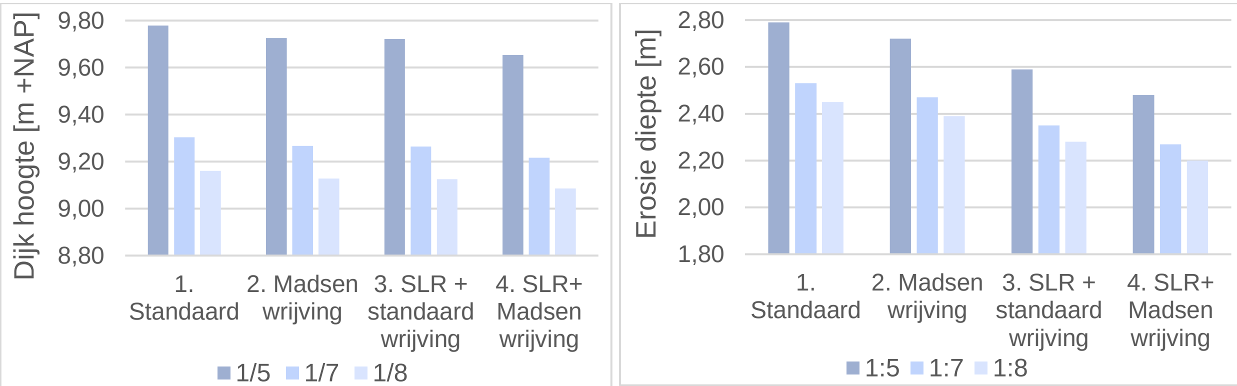
Figuur 1. Visualisatie BGD-pilotstudie

Door de hydraulische randvoorwaarden aan de dijkteen te mitigeren, zou de begroeide vooroever de benodigde dijkdimensie kunnen beperken, zodat een duurzamere dijkversterking ontstaat. Een kwalitatieve analyse van de potentiële invloed van de begroeide vooroever (van de BGD-pilotstudie), waarin een relatie wordt gelegd met de invloed op de vereiste dijkdimensies, ontbreekt echter nog. Het doel van deze studie kan dan ook geformuleerd worden als:

"*Het kwantificeren van de potentiële invloed van het (veranderende) voorland van de Brede Groene Dijk op de hydraulische randvoorwaarden en het dijkontwerp en het onderzoeken van de implementatie van het voorlandpotentieel in het beoordelings- en ontwerpproces van primaire waterkeringen*"

Nadat een 'Simulating Waves Nearshore' (SWAN) model was opgezet voor het studiegebied, is een gevoeligheidsanalyse uitgevoerd om de gevoeligheid van de significante golfhoogte aan de dijkteen te bepalen wanneer de invoervoorwaarden worden gewijzigd. Deze voorwaarden hadden ook betrekking op de afmetingen van het voorland en de gekozen methode van het model om het effect van vegetatie mee te nemen. Uit de gevoeligheidsanalyse bleek dat de waterdiepte op het voorland een grotere invloed heeft op de significante golfhoogte dan de breedte van het voorland (voor het BGD transect). Als de hoogte van het voorland toeneemt neemt de waterdiepte op het voorland af waardoor het effect van door de diepte geïnduceerde golfbreking op het temperen van de golven toeneemt. Bovendien vertoonde de significante golfhoogte een significante daling door het indirect meenemen van vegetatie, met een verhoogde bodemwrijving, of met het expliciet meenemen, door het meenemen van de vegetatie-eigenschappen. Na de gevoeligheidsanalyse werd een scenario-analyse uitgevoerd waarin veranderingen in de afmetingen van het voorland werden gecombineerd met de indirecte methode om vegetatie op te nemen. Op deze manier kon de potentiële invloed van het intergetijdengebied worden gevisualiseerd. Hoewel er verschillende discussiepunten konden worden geformuleerd met betrekking tot de beperkingen van het model, de implementatie van de vegetatie en de gevoeligheids- en de scenario-analyse, kon worden geconcludeerd dat het voorland van significant belang kan zijn voor het verlagen van de hydraulische randvoorwaarden aan de dijkteen. Het potentieel van de vooroever hangt hierbij af van het vermogen van de vooroever om mee te groeien met zeespiegelstijging en de aanwezigheid van vegetatie die de bodemwrijving doet toenemen.

Nadat de invloed van de voorlanddimensies en de het meenemen van vegetatie bekend waren, moest dit nog in verband worden gebracht met hun invloed op het dijkontwerp. Daartoe moesten eerst de maatgevende stormcondities worden bepaald voor het faalmechanisme van golfoverslag (waardoor erosie van de kruin en het binnentalud van de dijk optreedt) en buitentalud-erosie (veroorzaakt door golfslag). Deze condities werden bepaald met Hydra-NL en vervolgens geïmplementeerd in het ontwikkelde SWAN-model. Vanwege de beperking van het SWAN-model om alleen een deterministische berekening te maken, werd besloten om de uitvoer van het SWAN-model te schalen naar de probabilistische Hydra-NL berekeningen. Met behulp van de geschaalde golfcondities kon de significante golfhoogte aan de teen van de dijk bepaald voor verschillende scenario's en faalmechanismen. De significante golfhoogte, andere golfcondities (e.g. waterstand, golflengte en golfperiode) en dijkkenmerken (e.g. helling en kleisterkte) werden gebruikt bij de berekening van de bepaling van de vereiste dijkhoogte om te voldoen aan de maximale overslagdebiet en de vereiste kleilaagdikte om erosie van het buitentalud van de dijk te voorkomen. Bovendien werden verschillende dijkhellingen toegepast om het effect van de buitendijkse helling op de dijkdimensies te bepalen. Wanneer de vereiste dijkafmetingen voor het standaardscenario, waarbij geen rekening is gehouden met voorland verghoging of de invloed van vegetatie (scenario 1), wordt vergeleken met de vereiste afmetingen wanneer wel rekening wordt gehouden met deze processen (scenario 4), ontstaat een vereiste hoogteafname van 22 cm en een kleilaagdikte afname van 31 cm voor een buitendijks talud van 1:5. Zoals te zien in Figuur 2 nemen de vereiste dijkafmetingen aanzienlijk af wanneer het buitentalud van de dijk wordt verlaagd.



Figuur 2. Benodigde dijkafmetingen voor verschillende scenario's

Bovenstaande resultaten tonen aan hoe belangrijk het is om bij de beoordeling en het ontwerp van een primaire waterkering rekening te houden met de mogelijke invloed van de toekomstige status van het voorland. Om dit mogelijk te maken zijn er verschillende benaderingen voorgesteld die eenvoudig of met enige inspanning kunnen worden geïmplementeerd in de huidige beoordelings- en ontwerpprocedure van een primaire waterkering in Nederland. Binnen de huidige procedure kan in Hydra-NL gebruik worden gemaakt van een Dam- en Vooroevermodule (DaV) die is ontworpen om veranderingen in golfcondities tussen de uitvoerlocatie in Hydra-NL en de dijkteen te bepalen. Aangezien de DaV-module echter is ontworpen voor de transformatie van golfcondities voor een maximum van 200 m, is deze module slechts in beperkte mate in staat is om het voorlandpotentieel in de huidige procedure te implementeren. Bij andere benaderingen, waarbij de vooroeverbreedte in de DaF-module fictief is vergroot of de uitvoerlocatie in Hydra-NL naar de voorlandrand is verplaatst, komt het nadeel van de 1D-berekening van de DaV-module aan bod. De laatste voorgestelde aanpak is om een afzonderlijk SWAN-berekening uit te voeren voor een specifiek vooroever-dijksysteem om de transformatie van de golfcondities over de vooroever met 2D-berekeningen te berekenen. De uitvoering van deze aanpak is echter zeer tijdrovend.

Concluderend kan worden gesteld dat de studie in staat was om de potentiële invloed van het voorland op de golfcondities aan de dijkteen zodanig te kwantificeren dat over dimensionering van de waterkering kan worden voorkomen. De veerkracht van het voorland om mee te groeien met zeespiegelstijging en het potentieel van de vegetatie om golfhoogtes bij extreme stormen te mitigeren, blijven echter onzeker. Als het voorland moeite heeft om in gelijke mate mee te groeien met de zeespiegel, kan sedimentatie op het voorland worden verbeterd door kunstmatige structuren aan te brengen, zoals rijshouten dammen of een zomerdijk. Bovendien kan het behoud van vegetatie met een goede kwaliteit hierbij een grote rol spelen. Indien dit alleen niet tot de beoogde sedimentatie leidt, zou gebruik kunnen worden gemaakt van 'Thin Layer Placement' (TLP) op de voorland om de hoogte van het voorland te verhogen. Kortom, als er een inspanning wordt geleverd om de aangroei van het voorland en de kwaliteit van de vegetatie te verbeteren, mag een inspanning om het voorlandpotentieel mee te nemen in de beoordelings- en ontwerpprocedure van primaire waterkeringen in Nederland niet achterwege blijven.

Contents

List of Abbreviations

WGD	Wide Green Dike
NbS	Nature-based Solutions
WBI2017	'Wettelijk Beoordelingsinstrumentarium' 2017
OI2014	'Ontwerpinstrumentarium' 2014
HWBP	'Hoogwaterbeschermingsprogramma'
SWAN	Simulation Waves Nearshore (model)
NAP	Normaal Amsterdams Peil
MLWL	Mean Low Water Level
MHWL	Mean High Water Level
SLR	Sea Level Rise
TLP	Thin Layer Placement
OAT	One-at-a-time

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1. Introduction

Dikes or other measures are needed along the coastline to protect the land behind it against flooding. Due to the dynamic behaviour of oceans in combination with several global and regional changes in climate conditions, which might lead to extreme weather and sea-level rise, land subsidence and extreme sediment supply, more and more is being asked of the coastal flood protection (Temmerman et al., 2013). Since the magnitude of the effects of climate change is still uncertain, there is a great need for primary flood defences to be adaptive to effectively respond to the changing boundary conditions (Vrinds, 2021). To provide adaptive measures there can be worked together with nature to help reduce the risk of flooding. Such measures are known as Nature-based Solutions (NbS). NbS are defined by IUCN as "*actions to protect, sustainably manage, and restore natural or modified ecosystems, that address societal challenges effectively and adaptively, simultaneously providing human well-being and biodiversity benefits*". Since NbS are focused on adaptive and sustainable concepts they are seen as preferable solutions/additions to traditional measures which are based on reinforcing flood defences with concrete or asphalt blocks to prevent erosion (van Loon-Steensma and Schelfhout 2017).

In practice, there are multiple NbS for which their overarching function is to provide adaptive flood defences where natural elements are included in the defence system to lower the hydraulic loads and decrease the need for hard elements/structures in the flood defence. An example of a NbS is the implementation of a Wide Green Dike (WGD) where a sea dike is being reinforced with clay instead of strengthened and heightened with concrete. To cope with the increased chance of failure due to wave impact and wave overtopping, which is resulting from a lower strength of a clay layer compared with concrete blocks, the outer slope of the dike must be made less steep. Doing so, a reduction in wave impact and wave overtopping occurs such that the clay layer and dike height do not need to have unrealistically great dimensions to reach the desired safety norms. Another implementation of NbS is the use of (innovations in the) vegetated foreshores to limit the hydraulic boundary conditions at the flood defence.

In the Ems-Dollard estuary in The Netherlands, a WGD is being implemented as a pilot study to gain information about the possibility to replace the traditional/hard reinforcement methods with nature-based methods to provide an adaptive solution. Besides the inclusion of clay as reinforcement material and the change of the outer dike slope, the vegetated foreshore of the dike in the Ems-Dollard estuary could be of great importance to limit the dike dimensions. However, a qualitative analysis of the potential influence of the vegetated foreshore of the WGD project in the Ems-Dollard estuary is still missing.

Still, since NbS are largely depending on ecosystems, their influence on reducing the hydraulic boundary conditions is uncertain leading to uncertainty in the determination of the failure probabilities of the flood defences. Hereby, the importance of adaptivity of the flood defence comes in again. To cope with the uncertainty in the magnitude of the influence of climate change on the flood defences, they are needed to be assessed on their failure probability to the most up-to-date climate predictions at least once every 12 years. After such an assessment procedure, flood defences that did not pass the assessment are needed to be strengthened to be able to keep up with the increasing threat of the (Dutch) waters.

1.1 Theoretical background

To give some insight into the theoretical background of the study, there will be elaborated on the different key aspects which are leading to the problem definition of the study. Hereby, the failure, as well as assessment and design of (sea) dikes in the Netherlands, will be addressed. Moreover, the principle of nature-based flood defences will be addressed to further elaborate on design principles as used for the case study of the WGD.

1.1.1 Failure mechanisms

Dikes can be divided into river, lake, or sea dikes. For these dikes, the failure mechanisms as shown in Figure 3 could arise. However, not all failure mechanisms have the same probability to occur for the different types of dikes. When focussing on sea dikes with a sandy core and clay with grass revetment, which is the type of dike investigated in the study, the most probable failure mechanisms are; wave overtopping (which can lead to erosion of the inner dike slope), wave impact or wave run-up (which can lead to erosion of the outer dike slope) and macro-instability of the outer or inner slope (which can lead to sliding of the inner or outer dike slope).

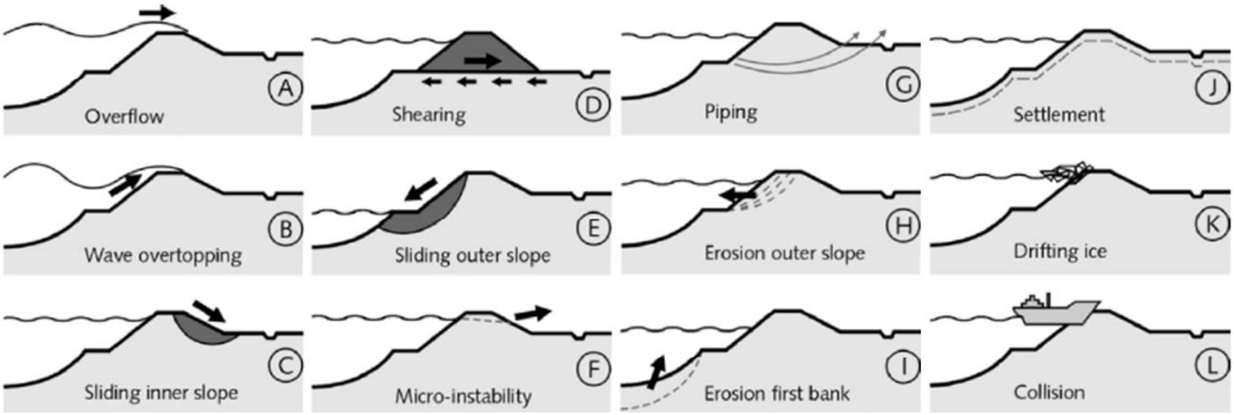


Figure 3. Failure mechanisms of soil structures (TAW, 1998)

Due to the limited strength of the clay layer protecting the sandy core of a WGD and the presence of wave run-up, the highest failure probabilities arise for wave impact and wave overtopping. Due to the wave impact or the waves overtopping the dike, the grass cover will be washed away and the clay layer is left to protect the dike from failing. The presence of a gentler outer dike slope of the WGD, compared to sea dikes in general, results in lower failure probabilities for failure due to macro-instability of the outer or inner dike slope. Due to the gentler slope, enough counterweight is present to balance out the weight of the slip plane when the shear strength in the dike decreases due to the increase in water level in the dike.

1.1.2 Assessment and design approach in the Netherlands

As of 2017, the norms for the assessment and design of dikes in the Netherlands changed from norms based on the exceedance probability of a flood event, to the failure probability of a dike. Since there is a constant exchange between assessing and designing of flood defences there is made sure that the defined allowed failure probabilities are not exceeded. In Figure 4 the relation between the assessment and the design is visualised. The different aspects of the assessment and design circle will be explained in more detail in the next sections. Finally, the software used for the assessment and designing of flood defences will be highlighted.

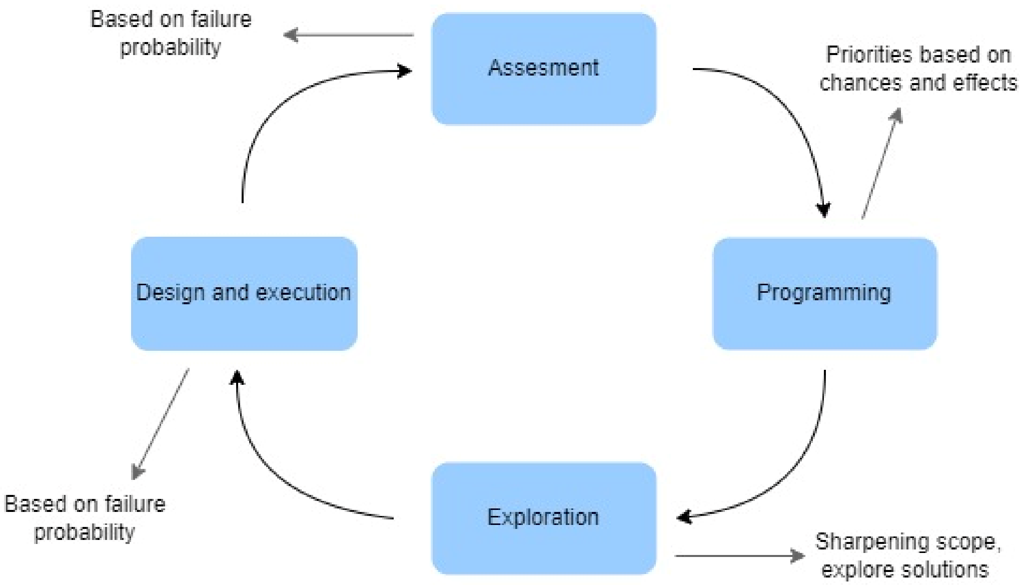


Figure 4. Assessment vs design procedure (adapted from Bree (2016))

1.1.2.1 Assessment

In the Netherlands, the safety of dikes needs to be assessed at least once every 12 years. The current assessment period ends in 2023. Within the assessment, the primary flood defences are assessed against two values, the signal and the limit value. When the failure probability reaches the signal value, a reinforcement plan should be constructed such that the reinforcement is succeeded before 2050 (step 2 in Figure 5). If the assessment results in a failure probability between the signal value and the limit value, reinforcement should start directly (number 3 in Figure 5).

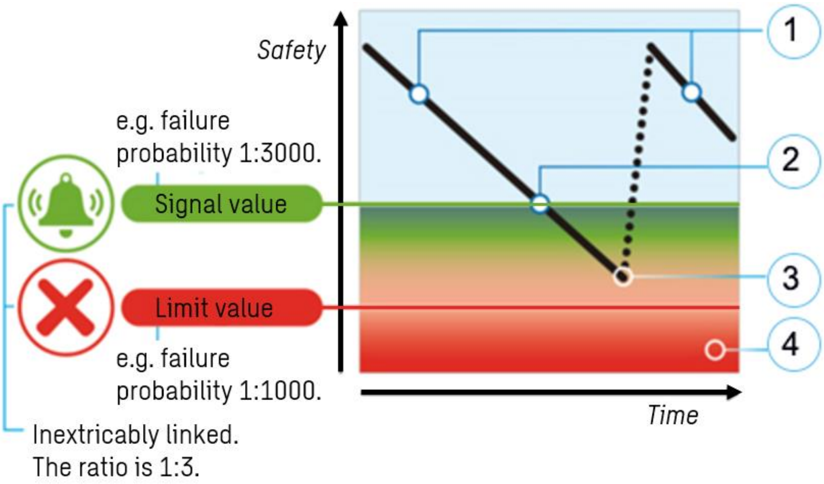


Figure 5. Signal value vs limit value (adapted from Lanz (2020))

To determine the failure probability accompanying a certain primary flood defence, the assessment approach proposed by the WBI2017 (in Dutch 'Wettelijk Beoordelingsinstrumentarium') needs to be followed. In the WBI, the rules for the determination of hydraulic boundary conditions and dike strength together with the rules for the assessment procedure of the safety of primary flood defences are defined. The assessment procedure described by the WBI guideline is tackled as a layered approach divided into simple (semi-probabilistic with partial safety factors), detailed (probabilistic with approximations) and advanced (full-probabilistic) assessment methods depending on the complexity of the problem and the quality of the available data (Simanjuntak et al., 2019). For the assessment following the layered approach, a balance needs to be found between what can be determined and what is needed to be determined to remain accuracy in the assessment procedure.

When following the WBI2017 approach, the execution of the assessment, regardless of the level of detail, is based on four activities as schematised in Figure 6. First, the required data needs to be collected. Thereafter, this data is used to schematize the situation which will lead to the model input needed for the next step, the calculation. Finally, the calculation/model results need to be interpreted to make a final decision on the dike safety. A schematisation manual, which is present for each failure mechanism, holds the different procedures of the activities included in the assessment procedure.

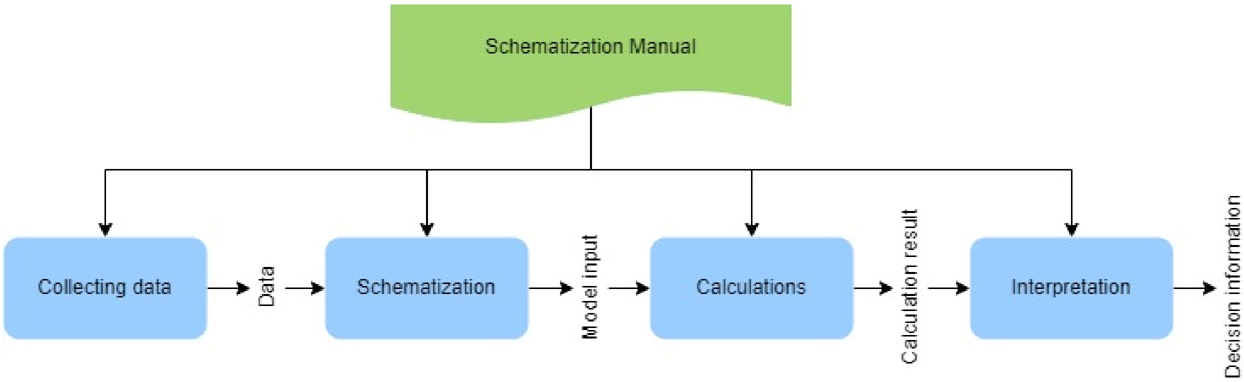


Figure 6. Schematization steps of the safety assessment WBI2017 (adapted from Rijkswaterstaat (2021b))

1.1.2.2 Programming/exploration

To determine the prioritisation of reinforcement projects in the Netherlands, the HWBP (in Dutch ‘Hoogwaterbeschermingsprogramma’) is set up. In this program, primary flood defences which resulted in a failure probability equal to the signal value are summed up based on their priority. Doing so, the HWBP is the Dutch organ that holds the bookkeeping of all the reinforcement projects for primary flood defences in The Netherlands. Within the HWBP, different solution methods are investigated to increase reinforcement speed and limit the costs when the design and construction phase starts. Moreover, different innovation projects are executed within the HWBP to experiment with more sustainable solutions such as NbS. Finally, from the HWBP various contracts are awarded to engineering companies to design the reinforcement of the different dike trajectories which are needed to be reinforced before 2050.

1.1.2.3 Design

The engineering companies aiming to reinforce primary flood defences in the Netherlands are needed to follow the design procedure as described in the OI2014 (in Dutch ‘Ontwerpinstrumentarium’). Based on data such as the length of the section of the considered flood defence and the accepted norm of failure probability, the hydraulic boundary conditions (design water level and wave conditions) are determined (Rijkswaterstaat, 2017). The hydraulic boundary conditions are thereafter implemented in the different software programs to determine the needed dike dimensions for the dike to fulfil the stated safety norms for the various failure mechanisms.

1.1.2.4 Software for flood defence assessing and designing in the Netherlands

For the assessment of flood defences, Riskeer is used to determine the failure probability for different failure mechanisms based on the current situation. For the design of flood defences, the hydraulic boundary conditions corresponding to the proposed dike designs are calculated with Hydra-NL. Within these software programs, underlying models are present for the calculation of the failure probability and hydraulic boundary conditions either based on the current state or future conditions. In Hydra-NL, statistics are used for the calculation of the wave conditions between 50-100 meter in front of the dike toe under different future scenarios and different return periods of a storm.

Since different failure probabilities are needed to be considered for the failure mechanisms as well as the fact that varying normative wave conditions are present for the different failure mechanisms, multiple calculations are needed to be conducted with Riskeer and Hydra-NL to determine the hydraulic boundary conditions corresponding to the various failure mechanisms and failure probabilities.

The inclusion of the effect of foreshores on the hydraulic boundary conditions is very limited and conservative in the dike assessment and design. In general, only the current bathymetry is included and the effect of vegetation on the foreshore is mostly ignored.

1.1.3 Nature-based flood defences

Due to changing boundary conditions at the flood defences, mostly caused by the effects of climate change, a more adaptive approach by introducing NbS might offer a great addition to the present flood defenses. Where in general flood defences are reinforced in a conventional way where often a lot of (hard/’grey’) structures such as seawalls and dikes are included and designed conservatively to comply with the changing/uncertain future boundary conditions, adaptive solutions can provide a more sustainable and hybrid defence.

NbS in front of the dike can contribute to limit the hydraulic boundary conditions (such as wave height) at the dike toe due to the adaptive capacity of ecosystems. The inclusion of NbS such as a vegetated foreshore increases the sustainability of the flood defence due to the ability of the vegetated foreshore to be self-sustained by acting as a sediment trap when exposed to sea-level rise in the long term (Temmerman et al., 2013; Willemsen, 2020; Zhu et al., 2020). On top of that, due to the presence of the root system on the vegetated foreshores, stabilization of the bed is reached which results in a decrease in the ability to erode due to flow over the bed (Willemsen, 2020). Moreover, one highly valued characteristic of the vegetated foreshore is its ability to dissipate wave energy (Möller et al., 2014; van Wesenbeeck et al., 2017). The stated processes increase the sustainability and cost-effectiveness of the flood defence due to the limited increase in hydraulic boundary conditions (van Loon-Steensma, 2015).

Waves travelling onshore are increased by wind effect and decreased in height due to depth-induced wave breaking and bottom friction. When reaching the (vegetation on the vegetated) foreshores, the water depth quickly reduces which increases the effect of depth-induced wave breaking. Moreover, due to the increased bottom friction (caused by the vegetation) and a loss of wave energy due to waves propagating through vegetation stems, branches and leaves, the wave height reduces (Vuik et al., 2016). By doing so, the wave height between the mudflat, the foreshore cliff and the dike toe shows a great reduction indicating the potential of a vegetated foreshore to lower the hydraulic boundary conditions such as wave impact. The described processes can all be found in Figure 7 where an example of a vegetated foreshore is shown. By making optimal use of the vegetated foreshore, the dike dimensions could be limited such that less material will be needed in the dike design which increases the sustainability of the dike.

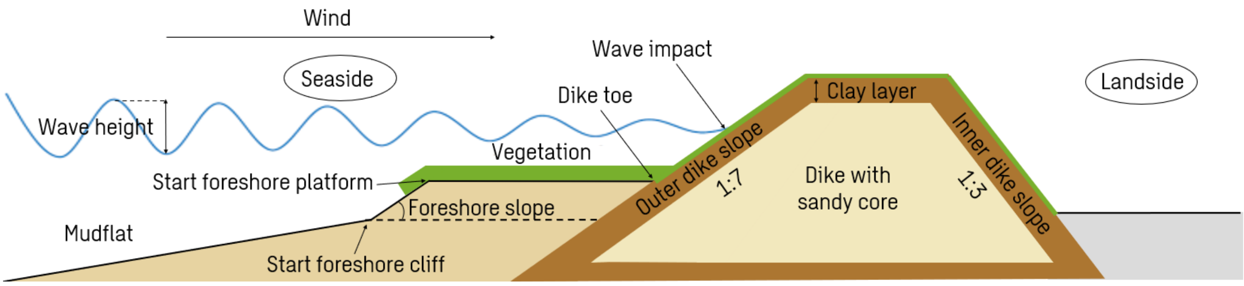


Figure 7. Dike geometry with vegetated foreshore under storm conditions

However, during storm conditions, where the flood defences are designed for, different wave conditions are present than during "normal" conditions. This implies that the capacity of vegetated foreshore under day-to-day conditions cannot be applied directly to the capacity when storm conditions apply (Vuik et al., 2016). Wave energy dissipation due to a vegetated foreshore may be limited during storm conditions where the water levels are too high for the vegetation to influence the wave conditions. Due to a lack of data that quantifies wave attenuation due to vegetated foreshores under severe storm conditions, it is hard to validate the empirical formulates and process-based descriptions used to extrapolate data from measured conditions to extreme conditions. This results in a high level of uncertainty in the ability of vegetated foreshores to contribute to a flood defence (Zhu et al., 2020).

To cope with this uncertainty, the power of the NbS must be combined with conventional engineering structures such that nature-based flood defences arise as shown in the right panel in Figure 8. By including the adaptiveness of the NbS into the flood defence, it is expected that nature-based flood defences can limit the constant threat of coastal flooding (van Zelst et al., 2021; Zhu et al., 2020). Where conventional engineering is focused on increasing the dimensions of the current flood defences to keep the water out (left panel Figure 8), nature-based flood defences are focussed on using the benefits of the ecosystem to help mitigate the hydraulic boundary conditions at the dike such that the height of the dikes could be limited to still be able to keep the water out.

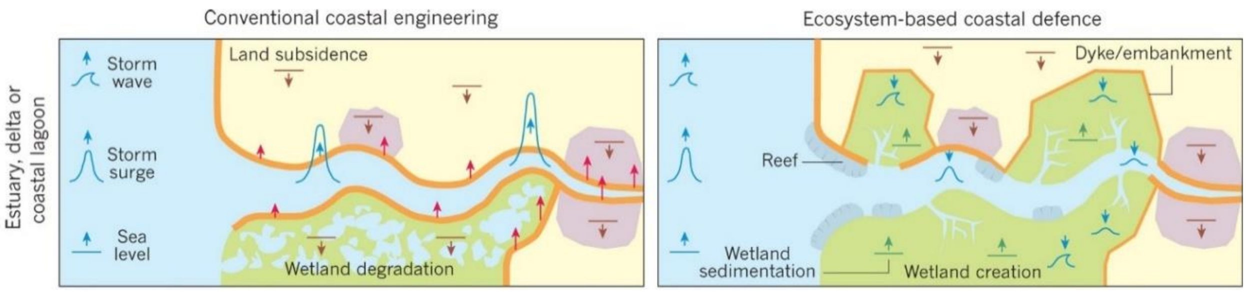


Figure 8. Conventional versus nature-based coastal defence (Temmerman et al., 2013)

1.1.4 Significant wave height

The significant wave height is a very important parameter to know at the dike toe during storm conditions since it influences failure mechanisms such as wave overtopping and erosion of the outer dike slope. When the significant wave height is larger, more waves can overtop the dike and the wave impact on the outer dike slope will have more energy. If more waves can overtop the dike and/or the wave energy of the wave impact is larger, failure of the dike is more prone to occur. Since the significant wave height is, amongst others, influenced by the water depth as well as bottom friction, the significant wave height changes when the foreshore dimensions or the way the effect of vegetation is taken into account changes. Doing so, for the rest of the study the significant wave height is seen as the most important hydraulic boundary condition.

1.2 Problem definition

Although NbS, for the measure where a dike is supported by a vegetated foreshore, sound very promising, it remains difficult to (1) quantify the effect of the vegetated foreshore to limit the hydraulic boundary conditions, which leads to (2) difficulty in the design of the reinforcement of the dike and to (3) include the presence of the vegetated foreshore in the complete assessment and design process the flood defence.

According to Vuik et al. (2016), the capacity of vegetated foreshores to attenuate waves depends both on the vegetation properties as on the hydraulic characteristics such as wave height and water depth. Moreover, the width and the height of the (vegetated) foreshore are stated as important characteristics of the efficiency of foreshores to attenuate wave energy (Vuik et al., 2016; Willemsen, 2020; Zhu et al., 2020). Since the state of the vegetation under different climate conditions is hard to determine for the vegetated foreshore area and only little data is available about the ability of salt marshes to reduce wave impact under extreme conditions, the contribution of the coastal ecosystems is uncertain (Zhu et al., 2020). For the dike section at the Dollard, the quantification of the effect of different foreshore settings on the hydraulic boundary conditions was limited to a height increase of the foreshore of 0.045 meter due to ecological constraints (Natura2000 area). Since a reduction of the hydraulic boundary conditions at the dike toe may have a significant influence on the failure probability of the different failure mechanisms, a correct determination of the influence of the foreshore is essential.

However, since the hydraulic boundary conditions present at the dike toe may be variable due to the uncertainty in the effect of the vegetated foreshore and climate conditions, the required dike dimensions might be variable as well. When various boundary conditions are defined, different dike designs can be determined.

To determine the varying hydraulic load at the dike toe, the hydraulic boundary conditions need to be determined with the use of hydrodynamic models. These models determine the wave conditions at around 50 meter in front of sea dikes on the (vegetated) foreshore under future scenarios. However, the changes in the bathymetry of, and vegetation on, the foreshore over the design period, are not considered. Moreover, vegetation or other structures may be present between the dike toe and the point where the hydraulic boundary conditions are determined. Doing so, discrepancies arise when considering (the changing) foreshore characteristics which can significantly influence wave height attenuation. Therefore, it is important to quantify this influence to correctly design the flood defence under future foreshore characteristics.

1.3 Research objective and questions

<u>Research objective</u>

To quantify the potential influence of the (changing) foreshore of the Wide Green Dike on hydraulic boundary conditions and dike design and investigate the implementation of the foreshore potential in the assessment and design process of primary flood defences.

Main research question

The main research question is focused on quantifying the influence of different foreshore dimensions as well as implementations on the foreshore and model settings on the hydraulic boundary conditions and the design of the WGD. Moreover, the goal is to propose different approaches which could be used to include the foreshore potential of already present foreshores in the assessment and design procedure of a primary flood defence. Concluding, this will lead to the following main research question:

"How do foreshore dimensions, implementations and model settings of the Wide Green Dike lead to variable hydraulic boundary conditions and dike design and how can this foreshore potential be applied in the assessment and design of a primary flood defence?"

Sub-questions

1. How do different foreshore dimensions, implementations on the foreshore and model settings influence the hydraulic boundary conditions at the WGD?
2. How do the varying hydraulic boundary conditions influence the dike design?
3. How can the foreshore potential of already present foreshores be included in the assessment and design procedure of a primary flood?

1.4 Thesis outline

Before the different sub-questions will be introduced and worked out, chapter 2 includes a case description of the Ems-Dollard region and the WGD pilot study. In chapter 3 the description and set-up of the model used for the first sub-question can be found first. Besides the introduction of the model used, a sensitivity as well as scenario analysis are part of the method of the first sub-question and are subsequently explained. In the last three sections of chapter 3, the results will be presented which are being criticised in the discussion and conclusions are drawn in the conclusion section. In chapter 4, first an introduction of the second sub-question can be found. Thereafter, the method of the second sub-question can be read. For the method, first the way the results of the first sub-question have been scaled to serve as the input of the second sub-question can be found. Moreover, there will be elaborated on the way the different failure mechanisms are tackled in the second sub-question. After the results are shown, which are resulting in different dike heights and clay layer thickness for the different scenarios and dike slopes, an discussion and conclusion will follow. In chapter 5 the last sub-question can be found. Herein, the method describes the different approaches implemented to include the effect of vegetated foreshores in the assessment and design process. Thereafter, the result of the analysis of these different approaches can be found. To close off chapter 5 and the last sub-question, a discussion and conclusion can be found about the results/implementation of the last sub-question. Closing off, an overall discussion about the research will follow in chapter 6 whereafter in chapter 7 the conclusion and recommendations about the conducted study can be found.

2. Case description

According to the work of van Loon-Steensma et al. (2012), the potential of a salt marsh (vegetated foreshore) to be able to be developed highly depends on the present abiotic conditions (i.e., elevation in relation to tidal range, concentrations of fine-grained sediment and velocity of currents along the coast). If the concentration of fine-grained sediment is more than 5% and the maximum flow velocity is smaller than 1.2 m/s, depending on the bathymetry, a salt marsh is already present, natural development of a salt marsh will arise, or small efforts are required for salt marsh formation to develop. If the maximum flow velocity is higher than 1.2 m/s, and the bathymetry of the foreshore is between -5 m +NAP and Mean Low Water Level (MLWL), larger efforts are needed for a salt marsh formation to occur. Finally, if the elevation is more than 5 m below NAP and less than 5% of fine-grained sediment is present, it is not likely that a salt marsh can be developed at the location. In Figure 9 the already present salt marshes are indicated in green and for the other areas, the abiotic conditions are translated to the different levels of salt marsh development.

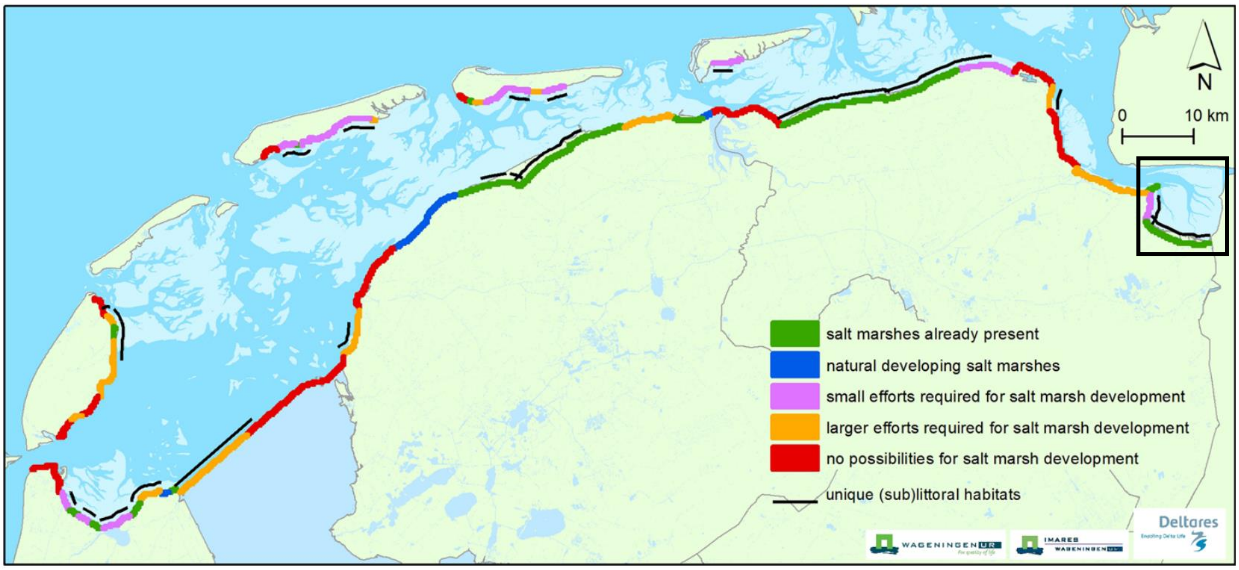


Figure 9. 'Salt marsh potential map' along the Dutch Wadden Sea (van Loon-Steensma, 2015)

If salt marshes are present, they can decelerate tidal currents by exerting friction such that the flow velocity and wave heights dampen over the foreshore (Marijnissen et al., 2020). By doing so, the hydraulic load on the dike will be reduced. Moreover, since vegetated foreshores tend to accumulate sediment, an extra beneficial effect of vegetated foreshores to limit the pressure on the flood protection infrastructure in the longer term arises (Vuik et al., 2019). Since dikes reinforced with clay instead of concrete have a lower resistance against wave impact, vegetated foreshores are very favourable for a WGD to be successfully implemented.

When focussing on the Dollard, as visualised in the black square at the right side in Figure 9, a salt marsh is already present due to the preferable abiotic conditions. Doing so, the Dollard basin is seen as a favourable location for a WGD to be implemented. The Dollard forms a bay of about 100 km^2 at the border between Germany and The Netherlands (Marijnissen et al., 2020). The Ems is the estuary which connects the Dollard basin to the Wadden Sea. As seen in Figure 10, around 80% of the Dollard consists of tidal flats where salt marshes are present towards the land borders which stimulates the implementation of a WGD.

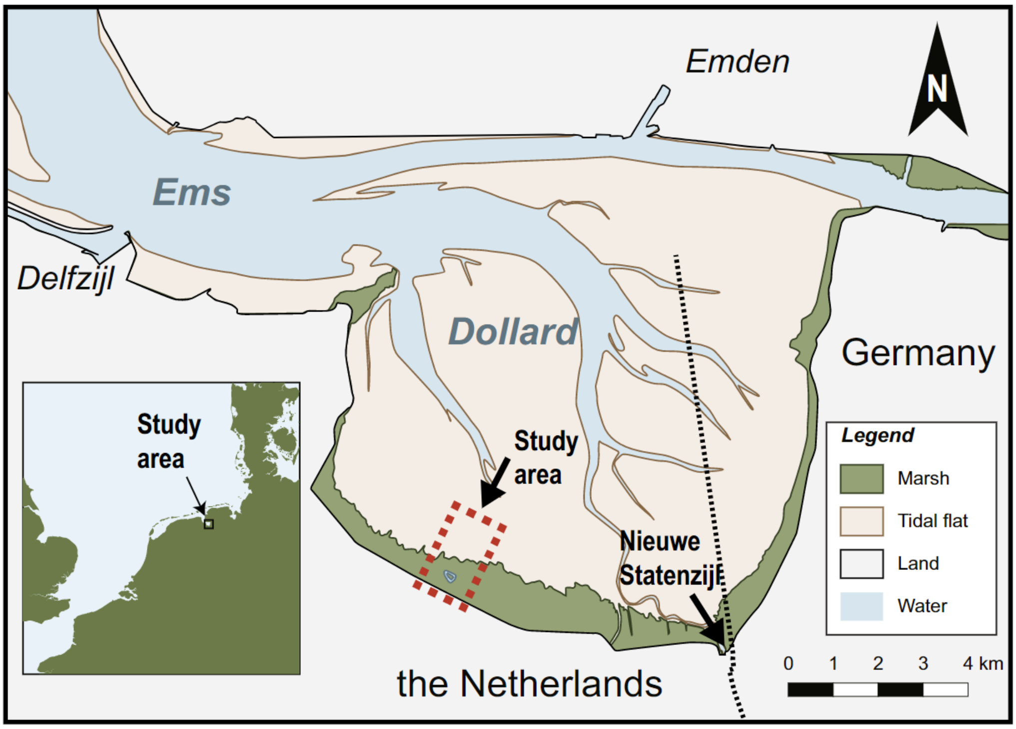


Figure 10. Study area WGD project (Marijnissen et al., 2020)

According to the dike assessment done by Sweco in 2010 based on the WBI assessment of 2017, 100% of the Dollard dikes must be reinforced. The largest contributions to the exceedance in accepted failure probability are failure due to erosion of the inner and outer slope caused by wave impact or wave overtopping or failure due to macro-instability of the inner slope (Sweco, 2022). However, as stated before, by making use of the design of the more gentle outer dike slope, macro-instability is no longer seen as a thread for dike failure.

The current outer dike slope of the case study dike was designed with an angle of 1:4 at the lower section and an angle of 1:7 at the upper section (van Loon-Steensma & Schelfhout, 2017). The lower slope section of the dike is reinforced according to conventional engineering with asphalt and stones to cope with wave impacts (Marijnissen et al., 2020) (Figure 11). However, due to settlement, the total outer dike slope can be assumed to have a slope of 1:5 from the dike toe to the dike crest.

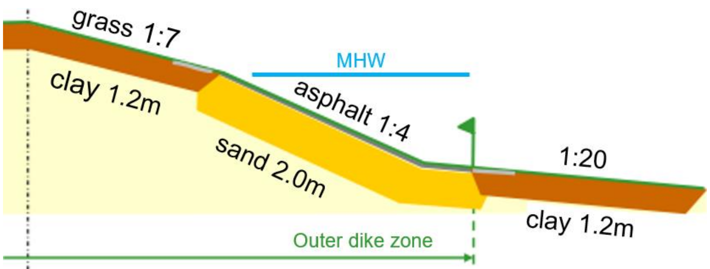


Figure 11. Current (traditional) dike design (adapted from van Loon-Steensma & Schelfhout (2013))

In the pilot study, a dike section of 750 m is aimed to be reinforced with sediment from the estuary itself. By doing so, the outer dike slope will be reinforced with clay from the area and covered with grass. The outer slope will feature a gentle slope with an angle of 1:7 from toe to crest (Figure 12). Due to the less steep slope, the wave impact will reduce so that the use of asphalt or stones will no longer be needed to protect the dike against failure due to erosion. Reinforcing an outer dike slope with clay became an option since only recently changed in the WBI assessment that failure of the grass is no longer equivalent to dike failure. The dike is only recognised as failing when also the clay revetment underneath the grass cover erodes such that the dike strength is beneath the threshold of dike strength.

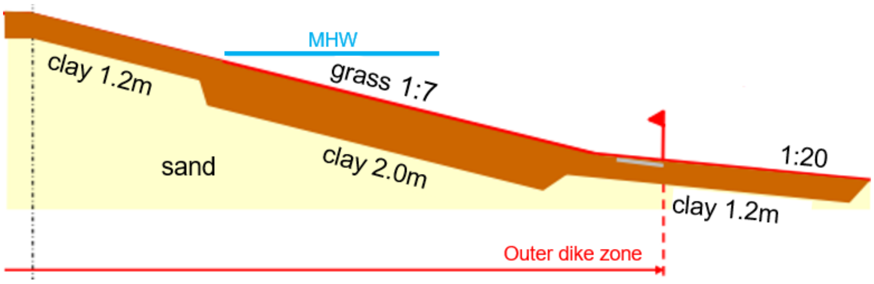


Figure 12. Wide green die design (adapted from van Loon-Steensma & Schelfhout (2013))

By reinforcing the dike with clay instead of the use of conventional engineering material, reinforcement that may be needed for future reinforcement projects will be simplified (i.e., adaptive measure) (van Loon-Steensma and Schelfhout 2013, 2017). However, this also shows a disadvantage of reinforcing with clay since even after milder storms the grass cover or even clay revetment may erode. This erosion needs to be fixed to be able to cope with the next (more extreme) storm. This requires more material than the traditional reinforcement which is not designed to erode during storms less extreme than designed for. Moreover, a disadvantage of the WGD concept is due to the less steep slope, the width of the dike increases such that the dike needs more space which affects the (protected) foreshore or the current infrastructure in the hinterland. Another advantage on the other hand however is that for the WGD project in the Ems-Dollard estuary, there is aimed to locally collect and mature the clay needed for the dike reinforcement from sludge from the area. By doing so, low transportation and manufacturing costs are present. This is an extra element to the more sustainable solution the WDG potentially offers.

For the significant wave height, which is seen as the most prominent hydraulic boundary conditions for the sake of this study, different factors are of importance. Besides the influence of bottom friction and water depth, which influence the wave energy, the wave height is influenced by wind speed, wind direction, wave direction and fetch. Hereby, the fetch is the length over which the wind blows over the water in a single direction. The significant wave height is the average wave height, from through to crest, of the highest third of the waves as visualised with H_s in Figure 13 (Bureau of Meteorology, 2015).

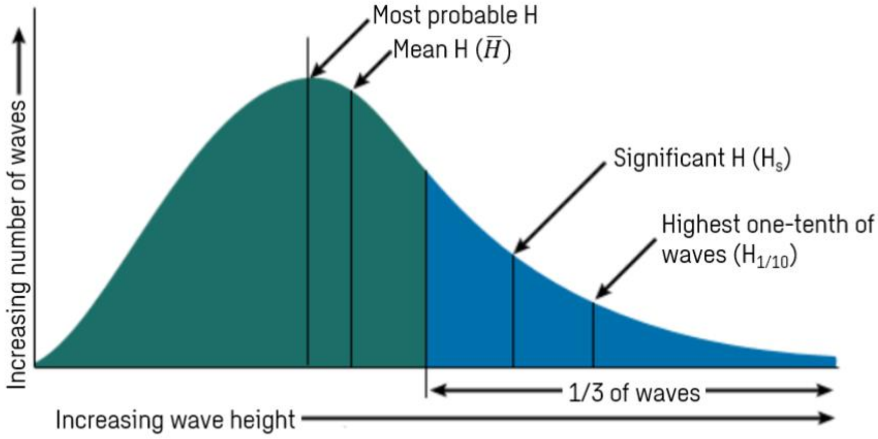


Figure 13. Significant wave height distribution (Collins, 2014)

3. Quantifying the influence of the foreshore on the significant wave height

A SWAN (Simulation Waves Nearshore) model is often selected to obtain realistic estimates of wave parameters in coastal areas, lakes and estuaries from given wind, bottom and current directions (SWAN User Manual, 2021). A SWAN model is based on the wave action balance equation with sources and sink terms for wave growth by the wind, nonlinear transfer of wave energy through three-wave and four-wave interactions and wave decay due to whitecapping, bottom friction, depth-induced wave breaking and vegetation (SWAN User Manual, 2021).

Although SWAN is widely used in various studies (van Zelst et al., 2021; Vuik et al., 2016, 2018; Willemsen et al., 2020), SWAN also has its limitations. One of its limitations is based on a poor approximation of quadruplet wave-wave interactions for long-crested waves (in deep water). However, this fundamental problem is shared with other third-generation wave models such as WAM and WAVEWATHCH III (SWAN User Manual, 2021). Also, for the triad wave-wave interactions poor approximations are made in the SWAN model for long-crested waves (in shallow water) (SWAN User Manual, 2021). These nonlinear wave-wave interactions are responsible for transferring wave energy from the spectral peak to lower frequencies and higher frequencies (SWAN User Manual, 2021). However, as stated, the limitations mostly reduce the accuracy of the SWAN model output when long-crested waves are present or when translation from deep to shallow water waves occurs. Since in the study area mostly short crested waves are present in shallow water, the limitations are not expected to largely influence the model output. The same counts for the limitations of the SWAN model regarding the computation of wave-induced set-up and wave-induced currents which processes are not relevant for the study area.

For the first part of the study, a SWAN (version 41.31) model is designed to simulate the wave conditions in the Dollard. Doing so, the hydraulic boundary conditions while different foreshore characteristics and model settings have been implemented are determined for the WGD location. However, to get an image of the sensitivity and the range of the significant wave height if changes are made in input conditions or model settings, a sensitivity analysis was conducted at first. Hereby, also another location in the Dollard basin is considered wherein different foreshore characteristics are present. Thereafter, a scenario analysis is conducted wherein different foreshore dimensions, implementations on the foreshore and different methods for the implementation of vegetation are stated. After the description of the method of the first sub-question, a results section can be found wherein the results of the sensitivity and scenario analysis are stated. Finally, a discussion and conclusion will follow wherein important discussion points will be pointed out and conclusions will be drawn which are partly used as input for the second sub-question.

3.1 Methods

In the next sections first a description of the SWAN model will be given whereafter the model set-up for the study will be considered. Moreover, the set-up for the sensitivity analysis and the scenario analysis will be described.

3.1.1 Model description

A SWAN model consists of model input, such as the definition of the grid cells (the computational grid) and bottom level, boundary conditions such as the significant wave height and the direction of the current, physics, such as wave dissipation due to bottom friction or depth-induced wave breaking, numerics, such as the minimum level of accuracy of the iterations and output commands that define which output parameters should be generated. In the next sections, these model domains will be described.

3.1.1.1 *Model input*

As a starting point, the SWAN model as used as a background model for Hydra-NL calculations was received from Deltares. However, due to computational restrictions, the area for which the SWAN model was calculating the wave conditions needed to be reduced. To do so, the Dollard was cut out of the total grid of the Wadden Sea model that was received. This area can be seen in the left map in Figure 14 with the purple rectangle. The red, green and blue areas visualise the different grids used in the Wadden Sea model. Herein, the red grid is used to generate boundary conditions for the green and blue grids. To reduce the computational capacity the blue and green gird were defined just like the purple grid as used for this study. In the right map in Figure 14, the bathymetry of the cut-out section of the Dollard basin is shown. Hereby, also the location of the boundary conditions as well as the two different study areas as later explained are shown.

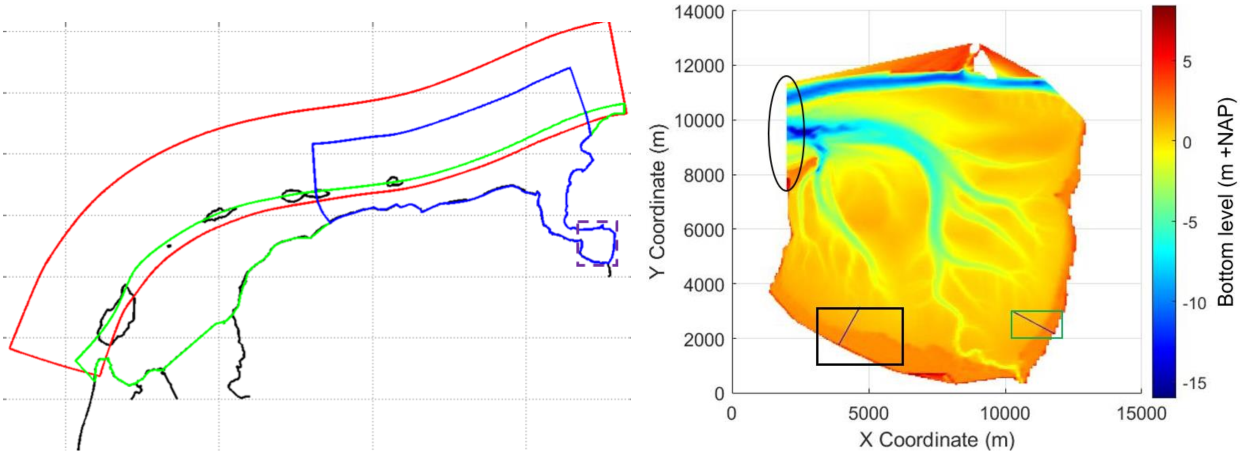


Figure 14. Map of different girds of the background SWAN model of the Wadden Sea as used for the input of Hydra-NL (left map). Bathymetry Dollard basin with study area and segment of WGD in the black rectangle, boundary segment in the oval and study area and segment of location in Germany in the green rectangle (right map).

After interpolating the depth over a grid of 50X50 meter, the bottom level for the area as shown in the right map in Figure 14 arises. The 50X50 m grid cells form the computational grid of the study and can be considered as a rectangular and uniform grid. The bottom level is added to the model as an input grid with the same coordinates as the computational grid.

There is chosen to further zoom into the study area and make use of a nested grid of 10X10 meter to be able to visualise the effect of the foreshore in more detail. This area is indicated with the black rectangle in Figure 14. Moreover, in the black rectangle, a blue line can be seen. This line visualises the cross-section of the WGD which is drawn to visualise wave and bathymetry characteristics over the transect. The same is done for a transect in Germany which is used in the sensitivity analysis of the study. This area and cross-section are shown with/in the green rectangle.

Finally, besides the definition of the computational grid and the bottom level, a (constant) water level and wind characteristics (speed and direction) are defined in the model input section.

3.1.1.2 *Boundary conditions*

Since the Dollard is cut out of the Ems-Dollard basin estuary, a boundary condition was needed to be assigned to the west part of the Dollard. The boundary conditions were chosen to be defined by a Jonswap-shaped spectrum at the boundary of the computational grid (SWAN User Manual, 2021). Along the segment, as shown with the black rectangle in Figure 14, a significant wave height, mean wave period, mean wave direction and directional spreading needed to be defined. These boundary conditions were taken from a Hydra-NL calculation normative for a storm with a return period of 1/3.000 year since this is the maximum allowed failure probability of the dike section of the WGD (Sweco, 2022).

3.1.1.3 *Physics*

In the physics section, first the generation mode of the SWAN model is defined. For the model used in the study, the third-generation mode for wind input (due to Snyder et al., 1981)) and quadruplet interactions (due to Komen et al. (1994)) are defined. Moreover, depth-induced wave breaking is included in the model with the formula of Battjes & Janssen (1978) with a constant breaker index determined by Battjes & Stive (1985). Wave dissipation by bottom friction is included in the model by a constant friction coefficient defined by Hasselmann et al. (1973) by using the value used for typical sandy bottoms. Finally, the triad wave-wave interactions are activated in the model using the LTA (Lumped Triad Approximation) method of (Eldeberky, 1996). For supplementary information about the technical background of the used (physical) formulations, the scientific and technical documentation of Swan version 41.31 can be consulted (SWAN team, 2018).

3.1.1.4 *Numerical parameters*

To set certain constraints to the model runs, numerical parameters are defined. The model will stop running when either a certain absolute change in significant wave height from one iteration to the other is reached or when a certain relative change in significant wave height from one iteration to the other is reached and the curvature of the iteration curve of the significant wave height normalized with the significant wave height is less than a certain value. One of these conditions has to be fulfilled in a minimum amount of wet grid cells. Moreover, something about the use of parameters in a stationary computation is included in the description of the numerical parameters. To include diffraction, a proportionally constant is defined in the numerical parameters sections. Finally, the SWAN model will stop running when after 50 iterations no solution is been found. However, this had never happened for the model used in the study.

3.1.1.5 *Output*

In the SWAN model, output parameters are determined for all of the grid points in the computational grid. By knowing these parameters over the grid points, MATLAB is used to plot the output over the whole study area. In Figure 15 an example of the output of the significant wave height over the whole Dollard and the cross-section of the WGD with boundary conditions corresponding to the 1/3.000 year normative storm is shown. Hereby, the location of the boundary condition can be seen along the x-coordinate of 2000 and from 7800 till 11300 on the y-coordinate axis.

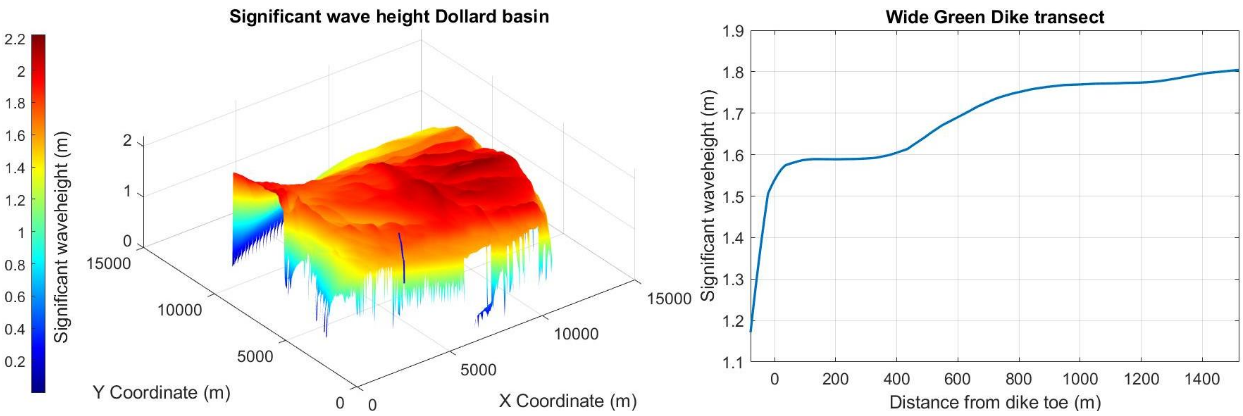


Figure 15. Significant wave height over Dollard basin (left) and cross-section WGD (right) for a 1/3.000 year storm

3.1.2 Model set-up

Besides the default options for the physics and numerical parameters, the input parameters and boundary conditions of the SWAN model that is constructed are taken from calculations of Hydra-NL. To do so, the significant wave height calculation is conducted for the climate G scenario without taking into account any model or statistical uncertainty (KNMI, 2014). This calculation is done since this calculation is most close to the calculation method of the SWAN model. In Table 1 the model set-up parameters of the input parameters are shown. The model set-up of the physics and the numerics can be seen in Table 10 in Appendix A. The values taken from the Hydra-NL calculation correspond to the output values of the significant wave height calculation for a 1/3.000 storm frequency and are extracted from an output location at the location of the WGD which is located about 90 m in front of the dike toe.

Table 1. Model set-up parameters

	Model parameter	Value	Source
Model input	Bathymetry	Bathymetry 2010	Background SWAN model Hydra-NL
	Constant water level	6.32 m +NAP	Hydra-NL 1/3.000 storm
	Constant wind speed	32.6 m/s	Hydra-NL 1/3.000 storm
	Constant wind direction	330˚	Hydra-NL 1/3.000 storm
Boundary conditions	Significant wave height	1.98 m	Hydra-NL 1/3.000 storm
	Mean wave period	3.75 s	Hydra-NL 1/3.000 storm
	Mean wave direction	335.3˚	Hydra-NL 1/3.000 storm
	Directional spreading	20˚	Default value SWAN user manual

3.1.3 Sensitivity analysis

To determine the importance of the choice of model input and model settings, the beforementioned model inputs and boundary conditions as shown in Table 1 were exposed to a sensitivity analysis. Moreover, the way the physics of the model are implemented regarding bottom friction over the vegetated foreshore will be changed to visualise the effect of the choice of these physics to model output.

The goal of the sensitivity analysis was twofold. First of all, by conducting the sensitivity analysis, it would become clear which input parameters are important parameters to determine as accurately as possible since they have a great influence on the significant wave height at the dike toe. Secondly, efficient measures to lower the significant wave height at the dike can be detected such that these measures can be implemented in certain scenarios in the scenario analysis.

After a first inventory, it became clear that the foreshore characteristics in the Dollard basin are differing along the “coastline”. Therefore, there was chosen to look at the influence on the significant wave height at two locations in the Dollard. Besides looking at the influence of the input parameters on the significant wave height at the WGD location, there was also looked at the influence at a dike toe location in Germany. By doing so, the influence of the parameters could be compared for different foreshore characteristics.

In Figure 14 the transect considered for the German transect is shown in the green rectangle whereas the transect of the WGD is shown in the black rectangle. In Figure 16 the bathymetry of the two different foreshores are shown. Hereby, the start of the foreshore cliff and the foreshore platform are visualised. For the vegetation, it is assumed to start at 1.4 m +NAP which will be elaborated on later. The width of the vegetation is measured as the distance between the dike toe and the end of the vegetation. In Table 2 the foreshore characteristics of the two different dike locations considered in the sensitivity analysis are shown.

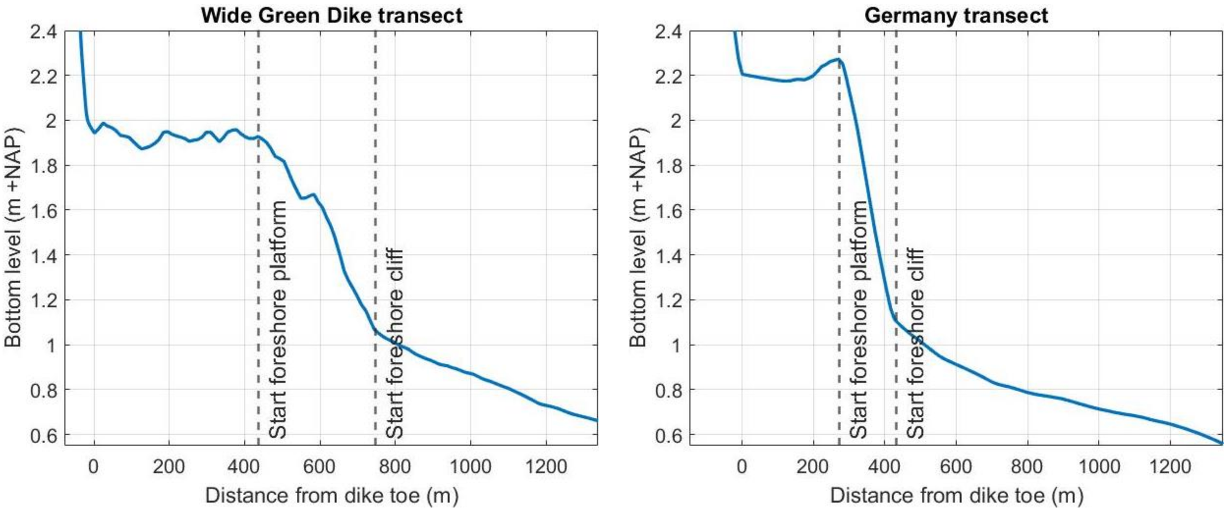


Figure 16. Bottom level different locations

Table 2. Foreshore characteristics

	Location WGD	Location Germany
Foreshore width [m]	650	390
Foreshore (platform) height [m +NAP]	1.9	2.2
Foreshore slope cliff [-]	$2.6*10^{-3}$	$5.4*10^{-3}$

As seen in both Figure 16 and Table 2, the foreshore of the WGD is lower but longer and has a less steep slope than the foreshore of the transect in Germany. As stated in multiple studies, as well the height as the length of the foreshore are important influences to reduce the significant wave height (e.g. Vuik et al., 2016; Willemsen, 2020; Zhu et al., 2020). Therefore, it was expected that different patterns would arise for the significant wave height over the two investigated foreshores.

The best way to investigate the influence of the foreshore characteristics from this point onwards is to separately increase/decrease the foreshore height and width of the different dike transects. Besides the foreshore characteristics, also the influence of input parameters of the SWAN model will be investigated in the sensitivity analysis. Moreover, it was determined to also investigate the influence of the incorporation of bottom friction due to the vegetation on the foreshore. In the basic model run, a constant Jonswap bottom friction is used over the whole study area. Doing so, the influence of a more rough bottom on locations where vegetation is present may not be taken into account. To do so, the influence of implementing bottom friction based on Madsen et al. (1988) as well as two theories which include vegetation properties are investigated within the sensitivity analysis.

As a starting point of the sensitivity analysis, the model parameters extracted from the Hydra-NL calculation as already shown in Table 1 were used. For the bottom friction, a constant Jonswap bottom friction of 0.038 is used which is a typical value for sandy bottoms. An overview of the model runs that were conducted in the sensitivity analysis is shown in Table 3. Hereby, there is chosen to visualise the results of the sensitivity analysis ranging from -20% to +20% and thus the parameters needed to be ranged between the values accompanying those percentages. However, for some parameters/foreshore characteristics, this range was made a bit larger since it was not possible to implement the specific range.

Table 3. Sensitivity analysis

	Parameter	%-range	Absolute value range
Model/Input parameters	Water level [m +NAP]	-20%/+20%	5.06 - 6.32 - 7.58 (Storms of: 1/14 - 1/3.000 - 1/99.000+ year)
	Mean wind speed [m/s]	-20%/+20%	26.08 – 32.6 – 39.12
	Mean wind direction [˚]	-20%/+20%	264 – 330 – 36
	Significant wave height [m]	-20%/+20%	1.58 - 1.98 - 2.38 (Storms of: 1/200 - 1/30.000 - 1/45.000 year)
	Mean wave period [s]	-20%/+20%	3 – 3.75 – 4.5
	Mean wave direction [˚]	-20%/+20%	268.24 – 335.3 – 42.36
	Directional spreading [˚]	-50%/+50%	10 – 20 – 30
Foreshore characteristics	Foreshore height [m]	-20%/+20%	WGD transect: 1.52 – 1.9 – 2.28 Germany transect: 1.76 – 2.2 – 2.64
	Width foreshore [m]	WGD transect: -13.8%/+27.7% Germany: -20.5%/+38.5%	WGD transect: 560 – 650 – 740 – 830 Germany: 310 – 390 – 470 – 540

	Bottom friction [-]	-50%/+176%	Madsen: 0.041 m – 0.059 m (for locations where vegetation is present) (Wamsley et al., 2010)
	Vegetation [-]	- - Vegetation properties	Jacobsen et al. (2019) Suzuki et al. (2012) height=0.30 m, diameter=0.003 m, stem density=600 stems/m^2, drag coefficient =0.4 [-] (Vuik et al., 2016)

3.1.4 Foreshore characteristics

In the next sections, some background information will be given about the way the vegetation on the foreshore is characterised within the sensitivity (and scenario) analysis of the study. In the rest of the thesis, the foreshore will be assumed as the part where vegetation is present. More information about the changes in foreshore height and width are shown in Appendix B and C.

3.1.4.1 *Vegetation boundary*

There is assumed that from the Mean High Water Level (MHWL), vegetation is present. In literature, this tidal benchmark is often used as an approximation of the seaward marsh edge (Bakker et al., 2002; Balke et al., 2016; D'alpaos et al., 2007; McKee & Patrick, 1988). For the Dollard basin, the MHWL is stated at 1.4 m +NAP (Dillingh, 2013). In Figure 17 a map of the vegetation present in the model is combined with a google maps image showing that the assumption of the 1.4 m +NAP vegetation boundary is valid.

Figure 17. Vegetation validation Dollard basin

3.1.4.2 *Vegetation properties*

As stated before, in the basic model run in SWAN, constant bottom friction is applied to account for wave reduction due to bottom friction. However, in the sensitivity as well as later on shown in the scenario analysis, the bottom friction due to vegetation is adjusted in multiple ways. In doing so, the effect of the presence of vegetation is implicitly and explicitly accounted for in the sensitivity analysis.

The implicit way assumes that when a storm occurs, the stems of the vegetation will break and thus only bottom friction should be assumed. This approach is conducted by Madsen et al. (1988) and from now on will be stated as "bottom friction Madsen". The approach is substantiated by Vuik et al. (2018) and also used in the study of Willemsen et al. (2020). The value of the bottom friction which should be applied if this assumption is made is based on a Nikuradse roughness length scale, which depends on both the type of vegetation on the foreshore (e.g. dense or high), as on the water depth. In the study of Steetzel et al. (2020), the Nikuradse roughness lengths are formulated according to the Manning coefficients conducted in the study of Wamsley et al. (2010). For the sensitivity analysis conducted, the Nikuradse roughness length representative for “lage kwelder”, which is the vegetation type most suitable for the vegetation on the foreshore of the WGD according to the most recent data of GeoWeb. GeoWeb is a database maintained by the Dutch government wherein data about the vegetation in rivers and on foreshores is collected. Besides “lage kwelder”, also the roughness value for “midden kwelder” is used to determine the influence of denser and higher vegetation on the significant wave height. The roughness length of the bare mudflat (in front of the vegetation) is assumed to be equal to 0.001 as also used by Vuik et al. (2019).

The explicit way allows for the vegetation properties (i.e. stem height [m], stem diameter [m], stem density [stems/m^2] and bulk drag coefficient [-]) to be incorporated into the SWAN model. The model developed by Mendez & Losada (2004) is used to account for the wave attenuation in the explicit way of including vegetation. In SWAN, two different approaches are available for the explicit implementation of vegetation. One is that of Jacobsen et al. (2019) and the other method is that of Suzuki et al. (2012). Both of the methods have been evaluated in the sensitivity analysis where for the vegetation properties the values as abstained by Vuik et al. (2016) have been used as also stated in Table 3 already. For the areas in the study area where no vegetation is assumed to be present, the implicit bottom friction of Madsen is assumed with a value of 0.001 as representative for the roughness length of a bare mudflat.

3.1.5 Scenario analysis

Within the scenario analysis, it is possible to combine certain measures, (expected) future foreshore conditions and/or vegetation on the foreshore to quantify the potential influence of the foreshore. The results of the sensitivity analysis could hereby be used as a base to determine which parameters should be included in (one of) the scenarios since it had shown to influence the significant wave height a lot. However, most of the influential parameters in the sensitivity analysis were not suitable to apply changes to in the scenario analysis since this does not comply with the normative storm conditions calculated with Hydra-NL or did not show a significant influence on the significant wave height. Still, changes in foreshore dimension (width and height) and the inclusion of Madsen bottom friction were concluded to be interesting to include in the scenario analysis.

As already stated before, it is expected that the foreshore will be able to grow with sea level rise (SLR). For SLR, a value of 25 cm is taken into account since this is expected for the climate G scenario. Besides growing along with SLR, an extra height increase was added for one of the scenarios since this could be implemented by human interventions by thin layer placement (TLP) (Raposa et al., 2020). This intervention could be interesting if needed for WGDs to be an accepted solution for dike reinforcement strategies. Although not included in the sensitivity analysis, since it was not possible to differ the parameter, it remains interesting to further investigate the potential influence of human interventions by the inclusion of brushwood dams or a summer dike. To sum up, the different scenarios can be read in Table 4 and with corresponding numbers visualised in Figure 18.

Table 4. Scenario’s scenario analysis

	Scenario number	Model characteristics
	1.	Standard scenario (Basic bottom friction)
Natural process	2.	Bottom friction Madsen
	3.	Sedimentation with SLR + Basic bottom friction
	4.	Sedimentation with SLR + Bottom friction Madsen
Human intervention	5.	Sedimentation with SLR + Bottom friction Madsen denser vegetation
	6.	Sedimentation with SLR + 20cm (TLP) + Bottom friction Madsen
	7.	Sedimentation with SLR + 90m width increase + Bottom friction Madsen
	8.	Brushwood dams at the end of the foreshore platform + Bottom friction Madsen
	9.	Summer dike at the end of the foreshore platform + Bottom friction Madsen

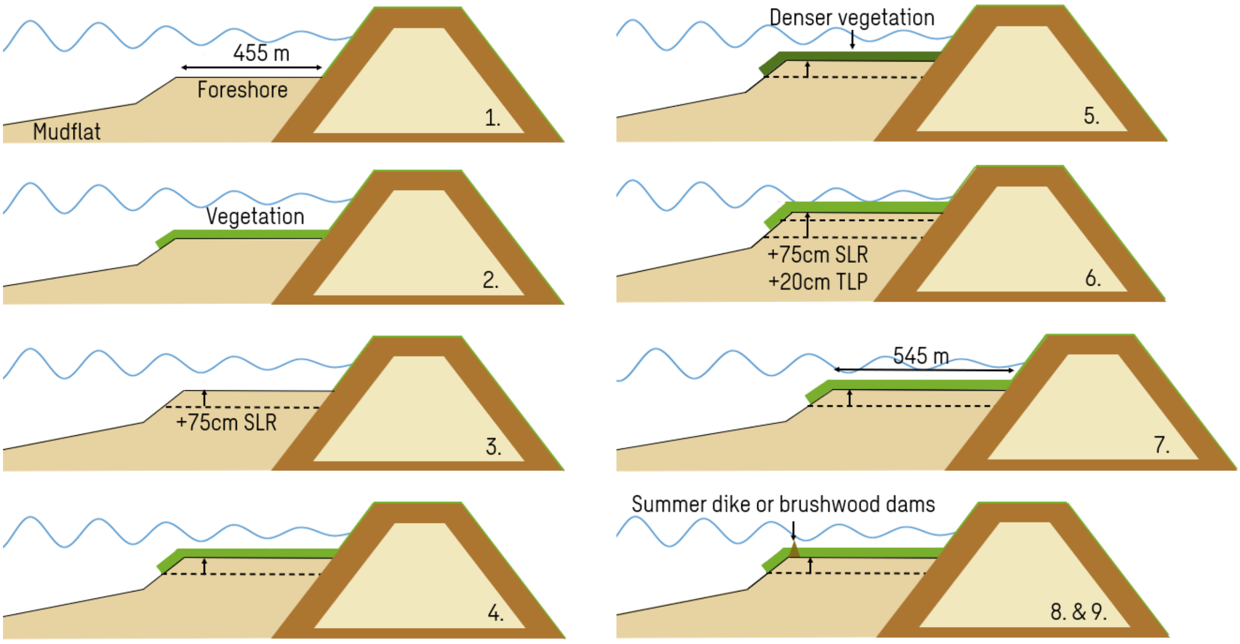


Figure 18. Visualisation scenarios

3.1.5.1 *Brushwood dams and summer dike*

To determine the effect of brushwood dams and a summer dike, the "obstacle" command in SWAN is used. For this command, it is chosen to use the formula of Goda et al. (1967). In the formula, the transmission of the waves over a dam depends on the (negative) freeboard of the dam, the incident (significant) wave height at the upward side of the dam and the two coefficients depending on the shape of the dam. In the study, the brushwood dams are implemented as a vertical thin wall of 0.5 m above bed level and the summer dike is implemented as a dam with a slope of 1:3/2 with a height of 1 m above bed level. For the implementation of the obstacles in the SWAN model, it was not possible to include the permeability of brushwood dams. However, as stated by the study of Schmitt et al. (2013), the transmission coefficient as used in the physical modelling of the brushwood dams would be equal to 1 when the water level increases the height of the brushwood dams as visualised in Figure 19.

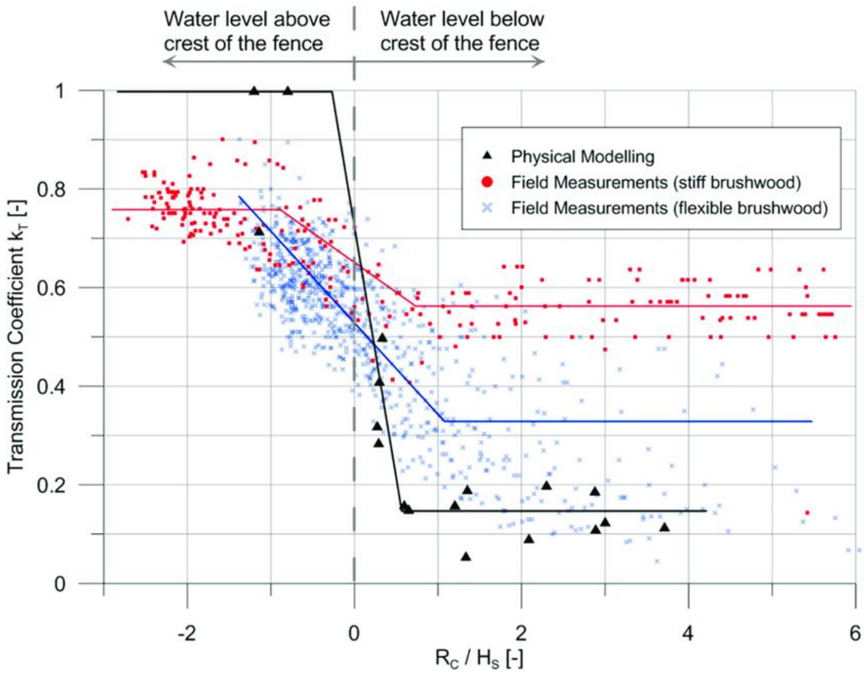


Figure 19. Wave transmission coefficients of bamboo fences under various hydrological conditions (Winterwerp et al. (2020) modified after Schmitt et al. (2013))

3.2 Results

In the following sections, the results of the sensitivity and scenario analysis are shown. For the sensitivity analysis first, an analysis is done for the influence of the different locations and foreshore characteristics on the significant wave height over the transect. Thereafter the results of the sensitivity are divided into the sensitivity of the model/input parameters and changes in foreshore height and width and the sensitivity of the way the vegetation is implemented in the model. This distinction is made since changes in the implementation of the effect of vegetation on the significant wave height cannot be expressed in percentages since a different implementation is considered from the basic implementation instead of only changing the values. The results of the second part of the sensitivity analysis are therefore shown in a cross-section of the different transects wherein the course of the significant wave height towards the dike toe is shown. The results of the scenario analysis are presented in a similar way to visualise the effect of the different scenarios over the foreshore transect.

3.2.1 Sensitivity analysis

A comparison of the course of the significant wave height over the foreshores of the WGD and the location in Germany can help with showing the importance of the foreshore when the location and characteristics of the foreshore differ. To do so, the course of the significant wave heights over the two transects is shown in In Figure 20. Hereby, the input parameters as shown in Table 1 with the implementation of the basic bottom friction are used. As seen, the wave height is about 18 cm higher when reaching the start of the foreshore cliff of the transect in Germany than for the transect of the WGD. Still, at both locations, the wave height reduces to a height of about 1.6 m at the dike toe. This indicates that the foreshore of the transect in Germany is more effective than the foreshore of the WGD even though the foreshore is a lot smaller and the foreshore slope is steeper. In extension, it can be concluded that (for this situation) the height of the foreshore plays a more important role in reducing wave height than the foreshore width.

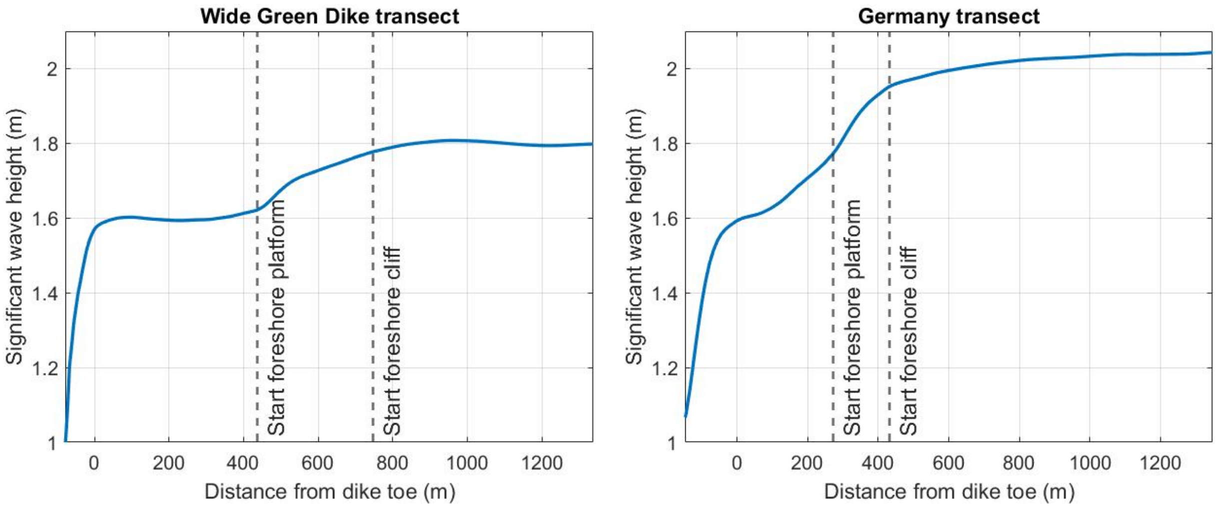


Figure 20. Significant wave height different locations

Said so, it could be concluded that during normative storm conditions, the higher the foreshore, the more effective the foreshore could be for reducing waves approaching a WGD. When more regular storms are present, the water depth on the foreshore reduces resulting in even a greater effect of bottom friction and depth-induced wave breaking. Doing so, the effect of the higher foreshore on the reduction in significant wave height at the dike toe would be even more pronounced.

3.2.1.1 *Model/Input parameters and width/height of the foreshore*

As the different model inputs and foreshore characteristics have been changed for the parameter ranges as shown in Table 3, the sensitivity of the significant wave height could be shown. Doing so, the different sensitivities for as well for the WGD as the transect in Germany are shown in Figure 21. Hereby, the significant wave height is shown on the y-axis and the change in parameters is shown on the x-axis.

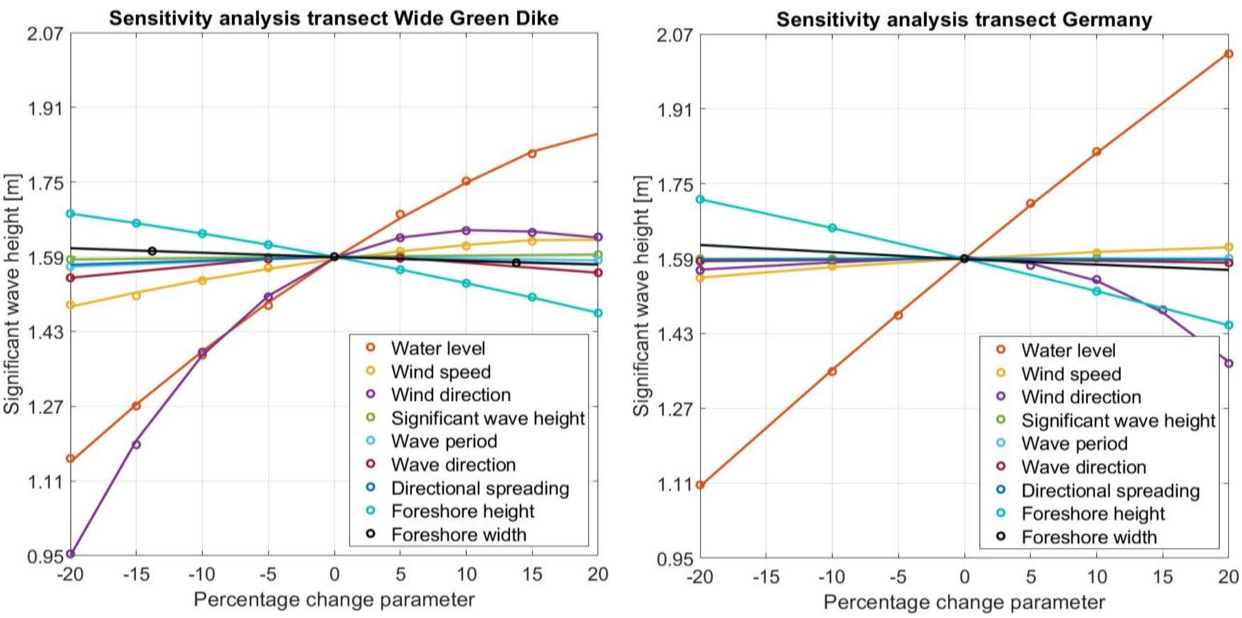


Figure 21. Results sensitivity analysis

In general, it can be seen that changes in the input parameters have more influence on the significant wave height at the WGD location than for the location in Germany. This can be concluded due to the bigger spread in the significant wave height for the investigated parameters. For the location in Germany, only significant changes can be seen for the water level, wind direction and the foreshore height whereas for the location of the WGD also changes in the wind speed show a mentionable effect on the significant wave height at the dike toe.

The biggest influence on the significant wave height seems to be water level. When the water level increases, the significant wave height at the dike toe increases due to a decrease in the reduction forces created by bottom friction and depth-induced wave breaking caused by the increase in water depth on the foreshore. Doing so, you would expect that the foreshore of the transect in Germany can lower the significant wave height to a greater extent when the water level increases than that of the WGD due to the higher bottom level of the foreshore of the transect in Germany. However, the opposite seems to happen where the significant wave height when increasing the water level of the WGD transects seems to reach a maximum value (Figure 21). This is caused by the fact that the wave height is not linearly growing with water depth but growing to a maximum value as seen in Figure 22. For the WGD location the value where the decrease in the increase in significant wave height is reached earlier due to the lower laying foreshore.

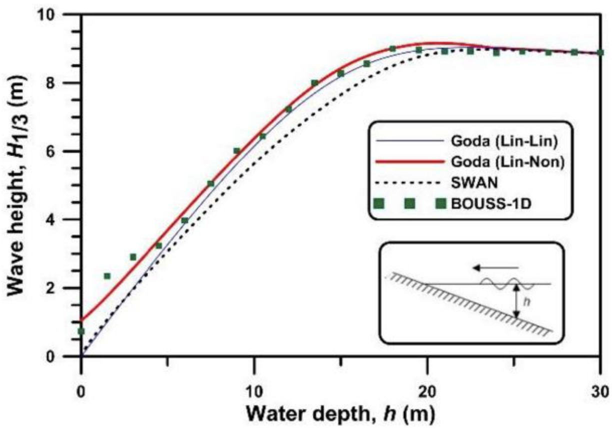


Figure 22. Water depth vs significant wave height (Kang et al., 2020)

An increase in foreshore height implicitly is a decrease in water level since it will lower the water depth on the foreshore. If the water depth decreases due to an increase in foreshore height it is thus expected that the wave height decreases due to the increasing effect of bottom friction and depth-induced wave breaking. This effect can clearly be seen in Figure 21 where the effect is a bit stronger for the location in Germany which was expected due to the higher bottom level of the foreshore. The same counts for wind speed where higher wind speeds result in higher water levels and higher wave heights as a result of wind set-up as both seen in Figure 23. These effects result in higher significant wave heights at the dike toe.

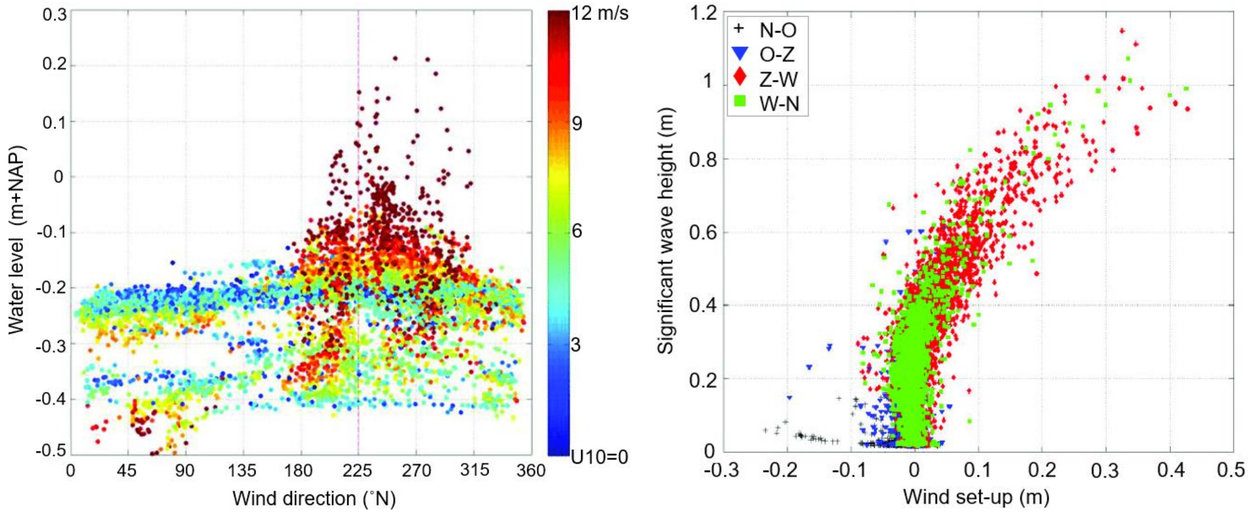


Figure 23. Relationship between wind speed and water level (left) and wind set-up and significant wave height (right) (Penning et al., 2016)

Finally, for the other parameters (significant wave height boundary condition, wave period, directional spreading and foreshore width), no significant changes in significant wave height could be detected for any of the investigated locations when changes in those input parameters are implemented.

3.2.1.2 *Vegetation implementation*

In the standard scenario, the method of the model does not allow for vegetation to be included in the model implicitly or explicitly. By changing the method of the model concerning including increased bottom friction or vegetation properties, it is possible to include the potential effect of vegetation. This effect is visualised in Figure 24. When vegetation is included implicitly, by increased bottom friction, or explicitly, by the inclusion of vegetation properties, a steeper decline in significant wave height occurs compared to the original run with equal bottom friction over the whole study area. This shows that neglecting the presence of vegetation may lead to an underestimation of the additional effect of vegetation on a foreshore.

Due to the greater width of the foreshore of the WGD, it was expected that when the method of the model allowed for vegetation to be included in the model, a greater influence on the significant wave height would be detected for the WGD than for the dike location in Germany. This is expected since the effect that vegetation has on lowering the (significant) wave height is present for a longer distance resulting in greater wave reduction by the vegetation. However, since the foreshore of the WGD is located lower with respect to NAP, it was questionable if the implementation of vegetation/increased bottom friction could counter this disadvantage. With the results of the sensitivity analysis, it can however be stated that vegetation implemented implicitly (Madsen) or explicitly (Jacobsen or Suzuki), overcomes the importance of a higher located foreshore. This can be seen by the greater potential of decreasing the significant wave height at the dike toe for the WGD than for the dike location in Germany (Figure 24).

Whether the implicit or explicit vegetation method should be applied in the model to include the influence of the present vegetation depends on the storm conditions. When applying the implicit way, it is assumed that the vegetation is no longer standing up straight and a mean value is taken for the roughness height of the vegetation based on an estimation of the vegetation present. When applying the explicit way, the vegetation properties and bulk drag coefficient of the specific vegetation in the area must be known. Since the bulk drag coefficient depends on the water depth and other wave conditions, it is not yet possible to make a genuine estimation of the bulk drag coefficient conditions (Möller et al., 1999, 2014). This limits the use of the explicit method to include the effect of vegetation in the model to wave conditions for which the bulk drag coefficient is measured in experiments. Doing so, for the purpose of this study where extreme storm conditions are assumed to be present, the implicit way may be more accurate to implement.

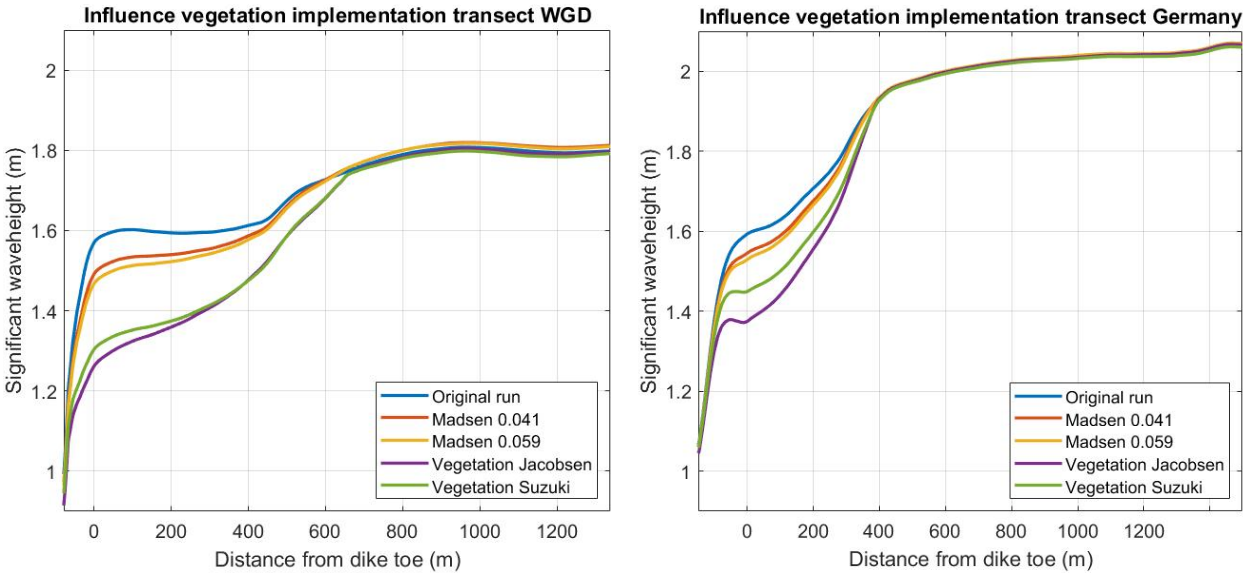


Figure 24. Results sensitivity analysis with implementation influence vegetation

3.2.2 Scenario analysis

For all the scenarios implemented within the scenario analysis, the largest decrease in significant wave height occurs when the waves are “climbing” the foreshore cliff from about 800 m to 400 m in front of the dike toe. This is due to the fast increase in bottom level due to which the maximum wave height rapidly decreases as a result of the decrease in water depth. Since only for the scenario where the foreshore width is increased the location of this fast decrease in water depth changes, the differences in the significant wave height for the other scenarios are mostly shown when the foreshore platform is reached. This could clearly be seen in Figure 25 where the results of the scenario analysis for a return period of 1/3.000 year are plotted. In the figure, the location from where the vegetation starts the linewidth is increased. When reaching the vegetation on the foreshore, different patterns occur for the course of the significant wave height under different scenarios.

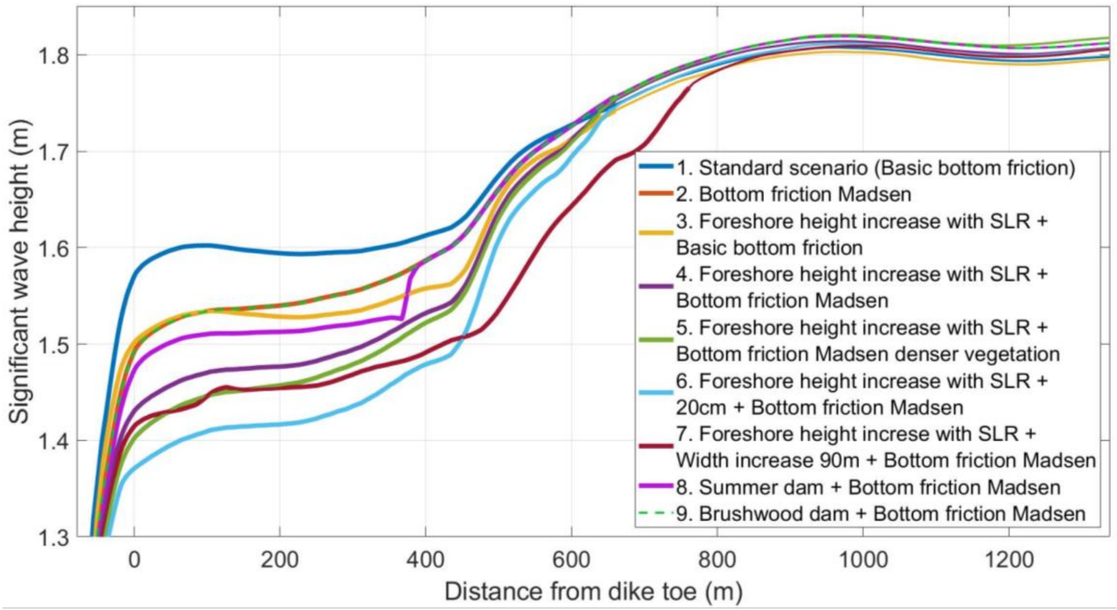


Figure 25. Results scenario analysis WGD transect for a normative storm of 1/3.000 year

When looking at the results of the scenario analysis as shown in Figure 25, it can be seen that as well including foreshore height increase as including increased bottom friction for the presence of vegetation show a promising decrease in significant wave height at the dike toe compared to the standard scenario.

As seen in Figure 25, both including the increased bottom friction in the bathymetry (yellow scenario) as changing the method of the model to account for increased bottom friction (orange scenario) show the same effect on the significant wave height at the dike toe compared to the standard scenario (-7 cm). When these two scenarios are combined, the significant wave height at the dike toe decreases even 6 cm more. For the scenarios where the foreshore width is increased in the bathymetry (purple), manmade structures are implemented in the model (magenta and striped green) or where more dense vegetation is included in the method of the model, only small effects on the significant wave height at the dike toe can be detected. The biggest influence on the significant wave height is shown when an extra 20 cm of foreshore height is added with respect to the inclusion of increase with SLR in combination with increased bottom friction (light blue scenario). Hereby, the importance of the inclusion of the natural or manmade foreshore height increase can be stressed.

3.3 Discussion

Although some first conclusions can be drawn quickly from the results of the first sub-question, some interesting points are needed to be addressed before diving into the conclusion and moving on to the second sub-question. The discussion points of the first sub-question are dived into the limitations of the study due to the set-up of the used SWAN model, and limitations present due to the implementation of vegetation in the SWAN model. Moreover, some statements are made about the way the sensitivity analysis is tackled as well as statements about what should, and should not have been implemented in the scenario analysis and why.

3.3.1 Limitations SWAN model

To limit the computational time of the model, some simplifications had to be applied to the developed SWAN model with respect to the SWAN model as used as the background model of Hydra-NL. By doing so, some limitations arose which will be elaborated on in the next paragraphs.

To start, there is made use of a rectangular computational and bottom grid instead of curvilinear ones and no grid is included that takes the currents into account. Moreover, where in the background SWAN model a curvilinear water level grid is included, the water level in the model designed for this study is assigned with a constant level. However, it is not assumed that those simplifications have a great influence on changes in wave conditions since the study area is located in a semi-enclosed, rather small basin, where no great changes could arise when currents or small changes in water levels are included.

Another simplification is that no changes in boundary conditions are included between the start- and the endpoint of the boundary segment. Doing so, the implemented boundary conditions are equal over the whole line segment specified as the boundary.

The greatest difference between the background SWAN model of Hydra-NL and the SWAN model set-up for this study is that the developed SWAN model only conducts one calculation based on the set model inputs and boundary conditions making it a deterministic calculation. Within Hydra-NL, numerous runs are conducted wherein the model inputs and boundary conditions are changed in such a way that different combinations are investigated resulting in the normative wave conditions for different failure mechanisms with various return periods and climate scenarios. Doing so, the wave conditions are probabilistically determined such that the most extreme situations that could occur arise.

The combination of the above-stated limitations of the developed SWAN model results in the difference in output between Hydra-NL and SWAN calculations. In Table 5 the difference in wave conditions are visualised wherein the water level, wind speed and wind direction are equal since there are constant values in the SWAN model. The differences thus arise for the mean wave period, the wave direction and, the largest difference can be detected for the significant wave height. With the conduction of the sensitivity analysis however, it became clear that those parameters were not very influenceable when changed as input parameters. Therefore, there is chosen, for the next sub-question, to not change the input parameters to calibrate the output of the deterministic SWAN model with the more accurate probabilistic Hydra-NL calculations, but to calibrate the output parameters of the SWAN model runs with the output parameters of the accompanying Hydra-NL calculation. In addition, this makes it also possible to include uncertainty calculated within Hydra-NL since it is not the goal anymore to generate results as close to the Hydra-NL calculations as possible. By doing so, the percentages of which the SWAN output is needed to be corrected arise which are also shown in Table 5.

Table 5. Difference output Hydra-NL vs SWAN

	Water level [m +NAP]	Wind speed [m/s]	Wind direction [˚]	Mean wave period [s]	Differecen mean wave period [%]	Wave direction [˚]	Differecen wave direction [%]	Significant wave height [m]	Differecen Hs [%]
1/3.000 - Hydra-NL	6,32	32,6	300	3,75	-2,09	335,3	6,56	1,98	26,11
1/3.000 - SWAN	6,32	32,6	330	3,83		357,3		1,57	

3.3.2 Vegetation implementation

For the inclusion of the effect of vegetation on the wave conditions first of all a statement has to be made on how the location of the vegetation is implemented in the SWAN model. To do so, there was chosen to implement vegetation at grid cells which are located higher than 1.4 m +NAP since this was stated as MHWL. This vegetation boundary was validated with the help of Figure 17. However, when SLR occurs, MHWL will also rise such that the vegetation boundary would move more towards the land/dike since the vegetation will drown. This could result in less vegetation in real life than present in the model such that the effect of vegetation could be overestimated.

For the inclusion of vegetation into the SWAN calculation, different assumptions have been made that influence the model output. First of all, by implicitly include vegetation as increased bottom friction, a Madsen bottom friction value is assumed which depends on both the water depth and the type of vegetation. Hereby, the type of vegetation is estimated with the help of the vegetation data presented in GeoWeb. To do so, it is assumed that the vegetation is of equal quality over the whole area where vegetation is assumed to be present. This assumption may lead to discontinuities since in practice it is not the case that the type/quality of vegetation is the same over the whole stretch of the foreshore. This is visualised in Figure 26 where a part of the vegetation of the Dollard basin is visualised with different colours. Herein, it can be seen that both in longitudinal as in perpendicular direction, the type of the vegetation differs. Moreover, since the vegetation data presented in GeoWeb is determined from aerial pictures combined with field measurements from July 2018, it is a snapshot of the type/quality of the vegetation at a certain point in time (Ministerie van Infrastructuur & Milieu Rijkswaterstaat, 2020). When the type of vegetation is determined closer to the winter period, the type/quality of the vegetation may differ and therefore have a different, less favourable, effect on the wave conditions.

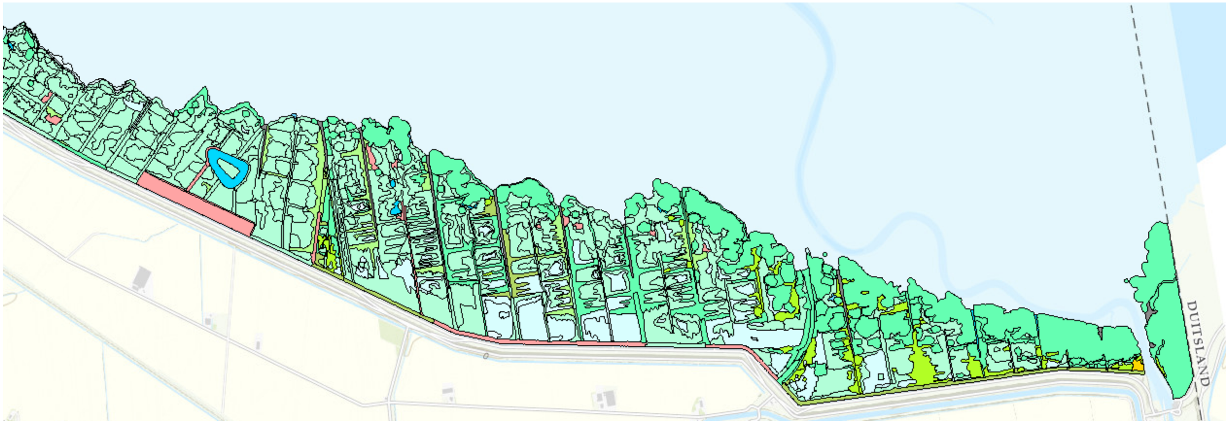


Figure 26. Snapshot GeoWeb vegetation Dollard area (based on data from Ministerie van Infrastructuur & Milieu Rijkswaterstaat (2020))

Besides implicitly including the vegetation on the foreshore by increased bottom friction based on the water depth and the type of vegetation, one option was to explicitly include vegetation such that the vegetation properties are directly determining the influence of vegetation on wave conditions. However, when explicitly including vegetation in the model, the vegetation properties, as well as de bulk drag coefficient of the vegetation, are needed to be known. Since no experiments were conducted on the study case area, the vegetation properties were unknown and vegetation properties from another study were used. However, since the bulk drag coefficient largely depends on water depth and wave height, the coefficient differs under different storm/wave conditions (Möller et al., 1999, 2014). Since the available experiments which give a value for the bulk drag coefficient were conducted under moderate storm conditions, the drag coefficient implemented in the SWAN model is most likely highly overestimated for the extreme storm condition (Vuik et al., 2016). As a result, as seen in Figure 24, a great influence on the significant wave height occurs in the sensitivity analysis when vegetation is explicitly taken into account in the SWAN model which effect is most likely highly overestimated and therefore not taken into account in the rest of the study.

3.3.3 Sensitivity analysis

The sensitivity analysis was done to investigate the sensitivity of the significant wave height to a change in input/model parameters and foreshore characteristics by chancing the parameters one-at-a-time (OAT). Doing so, the individual effect on the significant wave height while changing one specific parameter can be detected while keeping the other parameters fixed. Hereby, the sensitivity of the significant wave height when multiple parameters will be changed, which could be the case if for example the water level or significant wave height would increase when the wind speed is increased in the analysis, could not be detected within the sensitivity analysis.

Another statement has to be made about the way the sensitivity analysis was conducted for the WGD compared with the analysis conducted for the transect in Germany. As stated before, the starting point of the input parameters of the sensitivity analysis as shown in Table 1 are based on the storm normative corresponding to a storm occurring once every 3.000 year at the WGD. By changing the location of the determination of the normative storm conditions to the location of the dike in Germany, which is located less sheltered with respect to the connection to the open sea, different normative input and boundary conditions will be present. This could result in different patterns/significance in changes to the significant wave height. By doing so, different conclusions could be drawn when comparing the sensitivity of the significant wave height for the two different transects. However, since the hydraulic boundary conditions for the location in Germany could not be determined with Hydra-NL, since they lay outside the study area Hydra-NL, this effect could not be quantitatively determined.

When changes in wind speed are applied within the sensitivity analysis, an interesting result can be seen. Although in general it can be stated that the greatest influence on wave height should occur when the wind speed increases for a wind directed perpendicular to the dike, this is not reflected in the sensitivity analysis. For the sensitivity analysis, the wind direction normative for the location of the WGD was used which is 330˚. This direction is a result of the sheltered location of the WGD where a longer fetch has more influence on creating high waves at the dike than a wind directed perpendicular to the dike which generally causes the highest waves. As seen in Figure 14, this results in about 60˚ difference from perpendicular for the WGD and 30˚ difference from the dike located in Germany. Thus, you would expect that the increase in wind speed would have more influence on the dike transect in Germany than on the WGD. However, the opposite seems true since the significant wave height at the dike toe shows to be more sensitive to changes in the wind speed for the WGD location than for the location in Germany (Figure 24). The explanation for this could be found in the importance of the height of the foreshore. Since the foreshore of the transect in Germany is located higher, a small increase in significant wave height, due to the increase in wind speed, can better be embedded by the higher located foreshore of the location in Germany than the lower located foreshore of the WGD.

Finally, an interesting result was shown within the sensitivity analysis concerning the sensitivity of the significant wave height to changes in foreshore width. The change in foreshore width only shows a small increase/decrease in significant wave height for both of the transects. Doing so, the significant wave height only decreases a couple of centimetres when the foreshore width is increased in the order of tens of meters. According to studies such as conducted by Zhu et al. (2020) and Vuik et al. (2016), the effect of a wider foreshore is stated to have a far greater influence on the significant wave height. Concerning this, it has to be mentioned that within the sensitivity analysis, no vegetation was present in the method of the SWAN model as the bottom friction on the foreshore had the same value as on the mudflat. In Figure 27, a comparison can be seen between the reduction of significant wave height for changing water depths and foreshore width for a foreshore without (left) and with vegetation (right). It can be seen that when including vegetation, the reduction in significant wave height for increasing foreshore widths is much steeper for the situation where vegetation is included than when vegetation on the foreshore is not included.

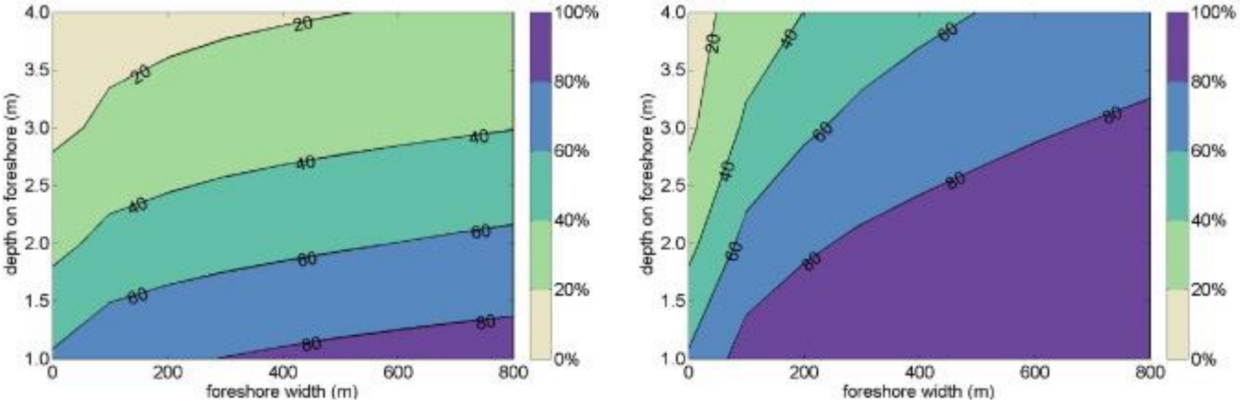


Figure 27. Reduction of significant wave height in case of water depth and width on/off the foreshore. Left panel; foreshore without vegetation, right panel; foreshore with vegetation (Vuik et al., 2016)

Another explanation can be found when looking at the course of the significant wave height over the different foreshores as shown in Figure 28. Hereby, for the foreshore of the WGD transect, it can be seen that when waves are moving towards the dike toe, the wave height does not decrease drastically anymore. Doing so, an increase or decrease in foreshore width does not have a great effect on increasing or decreasing the significant wave height at the dike toe. For the foreshore of the transect in Germany it can be seen that when increasing the foreshore width, the significant wave height reduces with a less steep slope than with the original slope. Doing so, almost the same significant wave height is detected at the dike toe. This shows that an increase in width does not influence the decrease in wave height significantly. A reduction in foreshore width however will at a certain point drastically influence the significant wave height since the slope of reduction in significant wave height will become steeper such that the foreshore could not mitigate the wave height to the value occurring with the original slope.

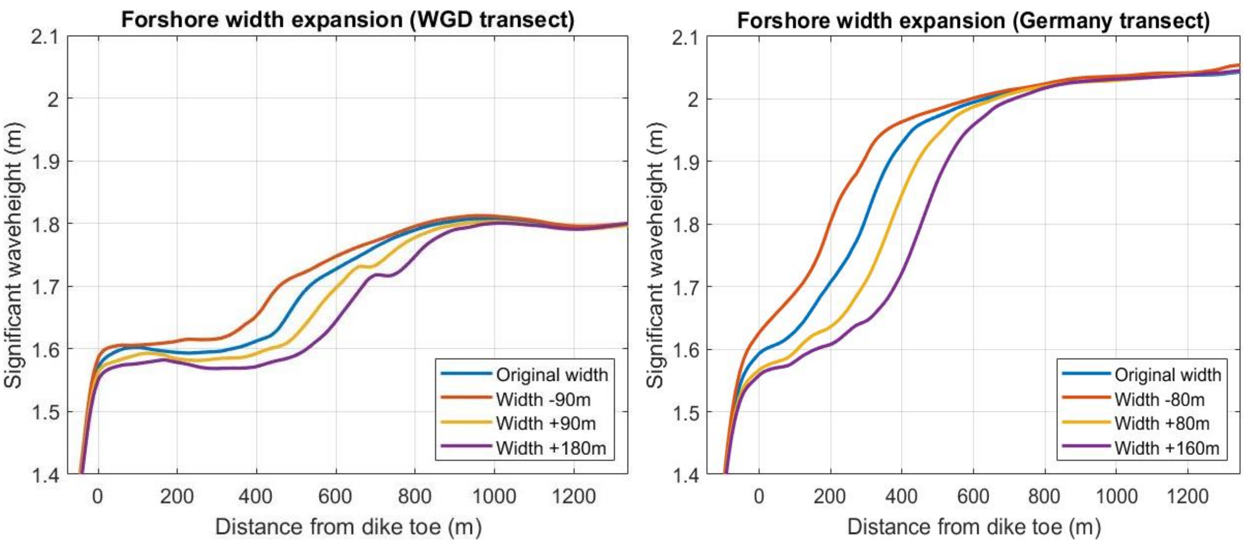


Figure 28. Significant wave height course for the two transects when including foreshore width expansion

3.3.4 Scenario analysis

Although the width of the foreshore (of the WGD) did not seem to have a great influence on the significant wave height in the sensitivity analysis, it might become influential when a different vegetation implementation is chosen as seen in Figure 24 where a more steep decline arises when the vegetation is implemented in the SWAN model implicitly or explicitly. However, based on the outcomes of the sensitivity analysis where the vegetation was included implicitly and explicitly, there was chosen to not further explicit take into account since, as stated before, it is very probable that an overestimation of the capacity of the vegetation is accounted for due to the lack of measurements of the vegetation capacity to lower the significant wave height under extreme conditions.

Finally, also the inclusion of brushwood dams and a summer dike were taken into account in the scenario analysis. However the brushwood dams and a summer dike did not show a significant influence on the reduction in significant wave height under extreme storm conditions, they could be of benefit under more moderate storms. By implementing brushwood dams or a summer dike, the wave conditions will be reduced when less extreme water depts are present. By doing so, they could enhance the foreshore to pace with SLR due to the enhancement of lower flow velocities which makes it possible for the sediment reaching the foreshore to settle (Siemes et al., 2020).

3.4 Conclusion

Based on the sensitivity analysis it can be concluded that the water level has the biggest influence on the significant wave height at the dike toe due to the water depth (on the foreshore). Hereby, the foreshore height implicitly is equal to a change in water level since it directly influences the water depth on the foreshore. Moreover, the wind speed causes an increase in wave height and water level due to wind setup. Said so, if a correct estimation of the significant wave height at the dike toe is needed to be calculated, the parameters influencing the water level and water depth on the foreshore are needed to be determined as accurately as possible.

As the input parameters as used in the basic run of the sensitivity analysis are based on statistical properties of future wave conditions, it remains difficult to determine how accurate these parameters are for future scenarios. Since no assumptions could be made based on the correctness of the Hydra-NL calculation, there is chosen not to change any input parameters in the SWAN model to better match the output parameters of the SWAN model with the Hydra-NL calculations but to change the output of the SWAN model to Hydra-NL output in the next sub-question.

Based on the results of the sensitivity as well as the scenario analysis, it can be concluded that if no different bottom friction is assumed for the vegetation on the foreshore then for the rest of the mudflat, the capacity of the vegetation to lower the significant wave height at the dike toe is highly underestimated. Moreover, the increase in foreshore height due to SLR or an extra increase by human intervention has a significant effect on the significant wave height at the dike toe and should therefore be considered in the design procedure of a dike reinforcement project.

The addition of brushwood dams or a summer dike did not significantly influence the significant wave height under the extreme conditions applied in the scenario analysis and should therefore not have to be taken into account when considered as a measure to lower the significant wave height at the dike toe. However those measures could be implemented to increase the accretion rate of the vegetated foreshore, it has to be mentioned that by implementing one of those measures, especially a summer dike, the ecosystem will be influenced. Moreover, if a summer dike is present, the foreshore will not be flooded yearly anymore resulting in different vegetation and a lower located area.

The two factors which are seen as the greatest contributors to limit the significant wave height at the dike toe are the decrease in water depth on the foreshore which is a result of the vertical increase of the foreshore with SLR (or higher) and the inclusion of increased bottom friction on locations where vegetation is present. Doing so, if the foreshore shows difficulties growing along with SLR, it is advised to investigate the possibility to enhance the vertical growth of the foreshore by implementing brushwood dams or a summer dike. If those options are expected to not be enough, it would be advised to enhance the tidal marsh resilience in the face of sea-level rise by TLP to make there is benefited from the decrease in water depth on the foreshore (Raposa et al., 2020). Finally, the evolution of the foreshore width has to be monitored to make sure that it does not reach its critical width such that the foreshore is not able to account for the wave dampening the dike design is based on.

4. Translating the foreshore potential to dike design

The goal of the second sub-question is to determine the needed dike reinforcement under different hydraulic boundary conditions based on the scenarios of the previous sub-question. By doing this, the potential influence of the vegetated foreshore can be translated to the dike design.

To do so, first of all, the tools that are used to calculate the safety of the dike under different wave conditions needed to be investigated. Hereby, it is decided to look at the two failure mechanisms which are most prone to occur at WGDs and are to a large extent determined by the significant wave height. Due to the extreme boundary conditions during a storm, high failure probabilities arise for wave overtopping and erosion of the outer slope due to wave impact.

Before diving into the method of the research question, the way the maximum allowed failure probability of the different failure mechanisms for the WGD will be explained. These maximum allowed failure probabilities will be used in the design of the dike reinforcement for the different scenarios of the first research question. Hereby, based on the conclusion of the first sub-question it is decided to not take into account the last two scenarios of the implementation of the brushwood dams and summer dike since these scenarios are not feasible to limit the significant wave height in dike reinforcement projects.

For the settings of the Hydra-NL calculations, there is made use of the W+ climate scenario where 75 cm of SLR should be taken into account. This climate change scenario with accompanying SLR is chosen since this value is also used for the design of primary flood defences.

4.1 Failure probability

For the assessment and design according to the WBI2017 and the OI2014, the allowed failure probability over a dike segment is the same. With the use of the allowed failure probability of a dike segment the allowed failure probability of a cross-section per failure probability will be determined. The complete calculation can be seen in Appendix D of which the concluding results are shown in Table 6 below.

Table 6. Results calculation maximum allowed failure probability per failure mechanism

	Max. failure probability per cross-section $P_{eis,dsn}$ [1/year]
Wave-overtopping	1/37.500
Erosion grass outer slope (GEBU)	1/200.000

Although for the different failure mechanisms the dike has to be designed for different return periods, the failure mechanisms are still connected in a way. There can be said that if the dike fails due to a failure of the inner dike revetment, the outer dike revetment is also allowed to fail. This results in the allowed failure probability of the outer slope revetment shifts from 1/200.000 year to 1/37.500 year which results in lower hydraulic boundary conditions the dike has to withstand against erosion.

4.2 Method

Before the reinforcement tools can be used to determine the dike dimensions under the different failure mechanisms, all (boundary) conditions as applied in the SWAN model for the different scenarios are needed to be determined. After the scaling parameters have been discussed, the different reinforcement tools used in the study will be elaborated on.

4.2.1 Hydra-NL vs SWAN calculations

For the Hydra-NL calculations, different calculations are done for wave overtopping than for erosion. To determine the conditions normative for wave overtopping, the HBN (“Hydraulisch Belasting Niveau”) calculation is done for an overtopping discharge of 10 l/s/m since this is seen as the critical overtopping discharge. For the failure mechanism of erosion of the outer slope, a water level calculation is done first.

Thereafter, the wave impact on grass revetment calculation is conducted with the found water level of the previous calculation (-10 cm due to available statistical space).

As stated before, there is chosen to adjust the output of the SWAN model so it will match the Hydra-NL output based on the standard Hydra-NL calculations. To do so, the Hydra-NL output is compared with the SWAN output as shown in Table 7 for the return periods as elaborated on in the previous section. The percentages with which the output must be scaled with are shown in the table as well.

Table 7. Hydra-NL vs SWAN output

	Water level [m +NAP]	Wind speed [m/s]	Wind direction [˚]	Mean wave period [s]	Difference mean wave period [%]	Wave direction [˚]	Difference wave direction [%]	Significant wave height [m]	Difference Hs [%]
1/37,500 Wave overtopping – Hydra NL	8,27	37,8	300	4,12	7,57	332,6	-3,81	2,42	30,11
1/37,500 Wave overtopping – SWAN	8,27	37,8	300	3,83		345,77		1,86	
1/37.500 Erosion – Hydra-NL	7,17	34	330	4,09	2,02	352,2	-1,35	2,72	49,45
1/37.500 Erosion – SWAN	7,17	34	330	4,01		357,01		1,82	

As seen in Table 7, the normative wave conditions for erosion differ a lot from the normative conditions of wave overtopping. This difference occurs due to the origin of the wave conditions which are normative for the different failure mechanisms. For wave overtopping, the wave conditions where a high water level in combination with high waves are normative whereas for the erosion of the outer slope the conditions that cause the highest wave are normative.

4.2.2 Failure mechanisms, tools and reinforcement

For wave overtopping, a direct reinforcement measure is to increase the dike height. When the dike height increases, less water can overtop the dike. For the reinforcement against erosion, the most important parameter is the thickness of the clay layer of the outer slope. By increasing, the dike is capable of withstanding higher a hydraulic load since the clay layer can withstand more wave impact until the clay layer is completely gone and the dike will fail.

Besides the thickness of the clay layer and the height of the dike, a reduction in the outer slope of the dike is an implementation that is beneficial for both failure mechanisms since it will limit the force of the wave impact and decreases the amount of wave run-up such that less erosion and overtopping will take place.

To investigate the complete picture of dike reinforcement options, different outer dike slopes are investigated ranging from 1:5, which is the current outer dike slope, to 1:7, which is the dike slope used in the final design, and 1:8, to investigate the extra decrease in dike height and thickness of the clay layer. However, when increasing the dike slope, it must be kept in mind that the dimensions in height and clay layer thickness may decrease, the needed horizontal space of the dike increases which may not be more convenient than an increase in height or clay layer thickness.

In the following sections, the formulas and tools used for the determination of the dike dimensions according to the failure mechanisms of wave overtopping and erosion of the outer slope will be discussed.

4.2.2.1 *Wave overtopping → Erosion of the inner slope*

Erosion of the crest and/or the inner slope revetement (GEKB) is determined by waves overtopping the crest or due to overflow. The overtopping volumes are statistically dependent on the overtopping discharge and associated wave conditions. According to the WBI2017, the assessment regarding GEKB is done completely probabilistic wherefore a schematization of the foreshore as well as the roughness of the outer slope is needed to be made (Rijkswaterstaat, 2021a).

The wave overtopping and accompanying formulas according to the design and assessment approach for relatively gentle slopes are shown in equation 2 to 5 below (EurOtop, 2018). Equation 6 describes the needed dike height to fulfil the overtopping requirements.

$$\frac{q}{\sqrt{g*H_{mo}^3}} = \frac{0.026}{\sqrt{\tan\alpha}}\gamma_b * \varepsilon_{m-1,0} * exp\left(-\left(2.5\frac{R_c}{\varepsilon_{m-1,0}*H_{m0}*\gamma_b*\gamma_f*\gamma_\beta*\gamma_v}\right)^{1.3}\right) \quad (2)$$

With: $\varepsilon_{m-1,0} = \frac{tan\alpha}{\sqrt{\frac{H_{m0}}{L_{m-1.0}}}}$, $L_{m-1.0} = \frac{g*T_{m-1.0}^2}{2\pi}$, $T_{m-1.0} = \frac{T_p}{1.1}$, $\gamma_\beta = 1 - 0.0033|\beta|$ (3)

Which leads to: $q = \sqrt{g * H_{mo}^3} * \frac{0.026}{\sqrt{\tan\alpha}}\gamma_b * \varepsilon_{m-1,0} * exp\left(-\left(2.5\frac{R_c}{\varepsilon_{m-1,0}*H_{m0}*\gamma_b*\gamma_f*\gamma_\beta*\gamma_v}\right)^{1.3}\right)$ (4)

and $R_c = \left(\frac{-\left(\ln\left(\frac{q*\sqrt{\tan\alpha}}{\sqrt{g*H_{mo}^3*\varepsilon_{m-1,0}*0.0026}}\right)\right)^{\frac{1}{1.3}}}{2.5}\right) * \varepsilon_{m-1,0} * H_{m0} * \gamma_b * \gamma_f * \gamma_\beta * \gamma_v$ (5)

$$h_{dike} = R_c + h_{water} \quad (6)$$

Where q is the overtopping discharge [l/s/m] R_c is the crest freeboard [m], γ_v is the influence factor for a wall at the end of a slope [-], α is the slope of the front face of the structure [-], $L_{m-1.0}$ being the deep water wave length [m], $T_{m-1.0}$ is the spectral period [s], T_p is the peak wave period [s], H_{m0} is the spectral significant wave height [m], γ_b is the influence factor for a berm [-], γ_f is the influence factor for roughness elements on a slope [-], γ_β is the influence factor for oblique wave attack [-], β is the angle of incidence [˚], α is the slope of the outer dike [˚], $\varepsilon_{m-1,0}$ is the breaker parameter [-], h_{dike} is the dike height [m +NAP] and h_{water} is the water level [m +NAP].

For the wave overtopping formulas, it can be said that the influence factor for the roughness elements (γ_f) as well as the influence factor for a wall at the end of the slope and the influence of the berm (γ_b) can be made equal to 1 since they are not present or considered as not effective for a reduction in overtopping of the WGD.

4.2.2.2 *Wave impact/attack → Erosion of the outer slope*

Erosion of the outer slope revetment (GEBU) can occur due to wave impact or wave run-up. However, if in a certain zone of the dike both wave impact as wave run-up are present, it is stated that the dike will be more prone to fail due to erosion due to wave impact than due to wave run-up (Rijkswaterstaat, 2021a). Therefore, this study will only address erosion due to wave attack.

To determine the required clay layer thickness for a green dike to fulfil the accepted failure probability for erosion of the outer dike slope, Sweco developed an Excel model. With the use of the results of the Delta Flume tests in combination with numerous simulations of wave impacts with the help of the numerical wave simulation model ComFlow, Deltares was able to develop a new formula for predicting the erosion velocity of bare clay due to wave attack (Deltares, 2020). This formula is included in the Excel model used by Sweco. The final (solved) formula predicts the erosion volume (V_e) as a function of time from the moment that the erosion hole starts developing (t), slope angle of the original profile (α), wave steepness (s_{op}), wave period (T_p) significant wave height (H_{m0}), erosion coefficient (c_e) (Deltares, 2020):

$$V_e = 16.7H_{m0}^2\left(1 - e^{-2.2*c_e(\tan(\alpha))^2*\min\left(3.6;0.0061*s_{op}^{-1.5}\right)*\left(1-\frac{0.4}{H_{mo}}\right)^2 t}\right) \quad (7)$$

With; $S_{op} = \frac{H_{m0}}{1.56*Tp^2}$ (8)

However, since the formula is based on tests conducted in the Delta Flume for different clays, there is a range in sand and organic material content for which the formula can be applied accurately. For the organic material, this percentage must be less than 5% (Deltares, 2020). Since the clay used for the reinforcement project at the Ems-Dollard has an organic content of 12%, equation 10 had to be adapted and validated with data from the Delta Flume tests with clay from the Dollard area before it was possible to make predictions about the residual strength of the clay revetment of the Dollard dike. The calibrated (solved) formula for the erosion volume is shown in equation 9 where a quadratic relationship for the slope is replaced with a linear relationship (van Steijn & Klein Breteler, 2021).

$$V_e = 16.7H_{m0}^2\left(1 - e^{-0.55*c_e*\tan(\alpha)*\min\left(3.6;0.0061*s_{op}^{-1.5}\right)*\left(1-\frac{0.4}{H_{mo}}\right)^2 t}\right) \tag{9}$$

Still, some difficulties arose when the Excel model was evaluated to use with the output generated in the first sub-question. This mainly had to do with the fact that the output of the SWAN model generates only one significant wave height which is not depending on the time of the storm (t). To use the model in the designed way, the water level course must be determined for a storm cycle. The water level course consists of the Mean High Water during the storm plus the storm surge. Based on these water levels, it is possible to calculate the corresponding wave conditions in Hydra-NL. However, before these mean wave conditions can be filled in in equation 3 to calculate the erosion volume, the wave significant wave height H_{m0} and the wave period T_p must be corrected for the angle of incidence of the waves. This correction is shown in equation 10 where β is the angle of incidence [˚] (Deltares, 2019).

$$f_b = max(0.35; (\cos\beta)^{0.67}) \quad for - 90^\circ \leq \beta \leq 90^\circ \tag{10}$$

Since it was not possible to determine the water course with corresponding wave conditions in the conducted study, it is decided that the output of the SWAN model already presents the average values for the normative storm event for 18 hours. Moreover, since the determination of the wave steepness resulted in a steepness that is physically not possible would be present, the wave steepness is adjusted to 0.05 [m/s^2] (Sweco, 2019).

The erosion volume is an important parameter to keep as small as possible since this is the volume of clay that needs to be added to the dike after the storm is past. The bigger the volume, the higher the repair costs. Besides the erosion volume, also the erosion depth d_e [m] and erosion length L_e [m] are of importance. The erosion depth is equal to the required clay layer plus 0.4 m for residual strength and 0.4 m for uncertainty due to grazing cows etcetera (Sweco, 2019). The erosion length could be of importance when the length has such a great value that it reaches the crest of the dike. In Figure 29 the erosion profile with all its parameter is shown. Hereby, d_o[m] is the starting depth of the erosion pit and is equal to 0.5 m, the slope of the terrace is equal to 1:10 and the slope of the cliff is equal to 2:1 (Sweco, 2021). The equations of the erosion depth and the erosion length as included in the Excel model are shown below Figure 29 in equations 11 and 12.

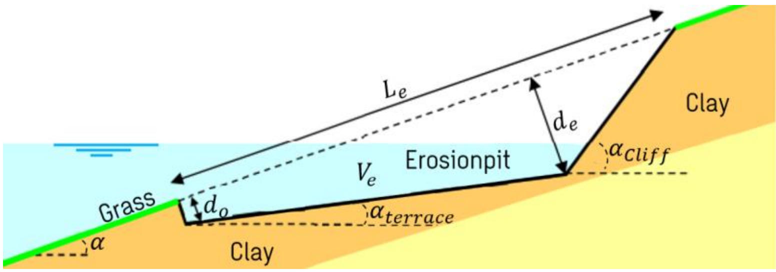


Figure 29. Erosion profile with dimensions (adapted from van Steijn & Klein Breteler (2021))

$$d_e = \sqrt{\frac{2V_e\tan(\alpha - \alpha_{terrace}) + d_0^2}{\left(1 + \frac{\tan(\alpha - \alpha_{terrace})}{\tan(\alpha_{cliff} - \alpha)}\right)}} \tag{11}$$

$$L_e = \frac{d_e - d_0}{\tan(\alpha - \alpha_{terrace})} + \frac{d_e}{\tan(\alpha_{cliff} - \alpha)} \tag{12}$$

4.3 Results

4.3.1 Scenarios failure mechanisms

The scenario's as used in the first sub-question, minus the scenarios where the brushwood dams or summer dike are included, were implemented in the SWAN model based on the output of Hydra-NL for both wave overtopping as erosion calculations. Thereafter, the output of the significant wave height was scaled according to the scaling parameters as shown in Table 7. Doing so, the significant wave height over the WGD transect as shown in Figure 30 and Figure 31 arise.

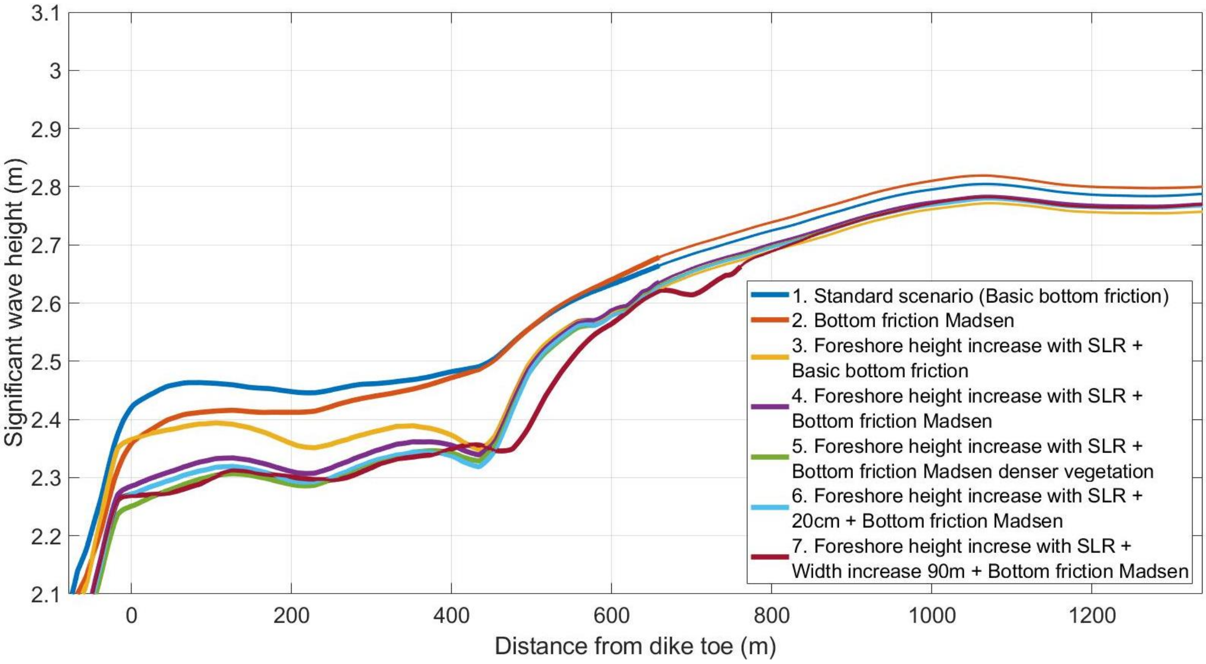


Figure 30. Results scenario analysis wave overtopping

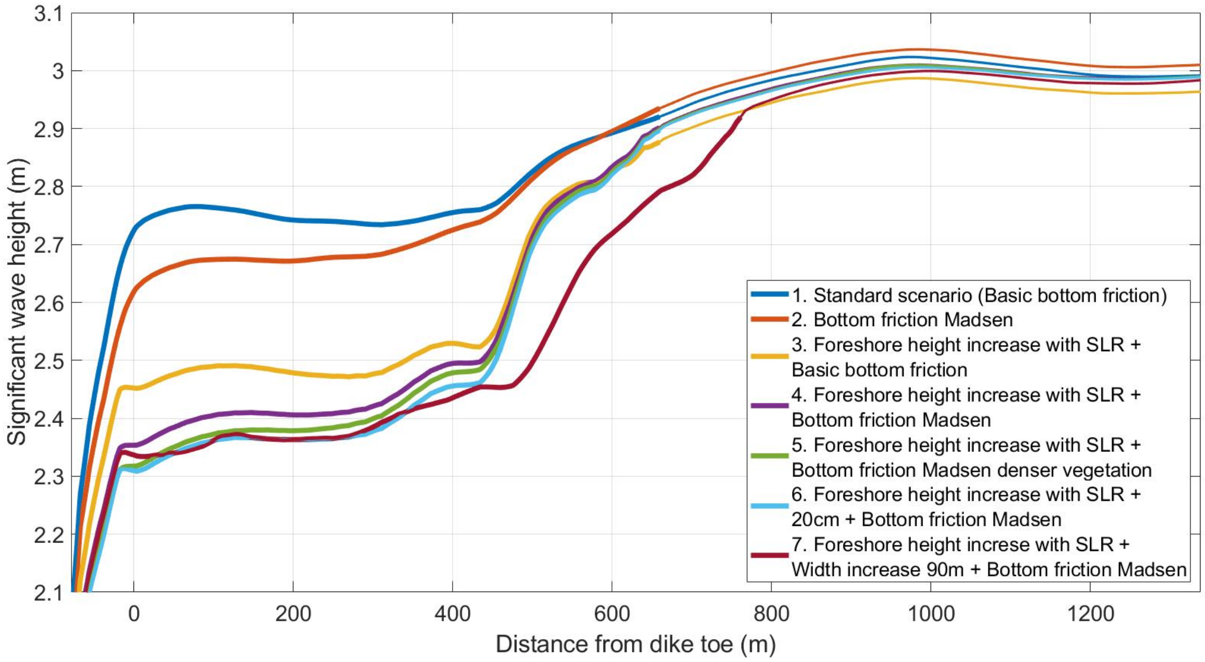


Figure 31. Results scenario analysis erosion

Firstly, it is striking to see that the different scenarios show a greater influence on the reduction in significant wave height at the dike toe for the failure mechanism of erosion than for wave overtopping. Where the total spread in significant wave height is about 20 cm for wave overtopping it is almost twice as large for the failure mechanism of erosion. The physical explanation for this observation is that for the failure mechanism of erosion, as a result of the probabilistic approach of Hydra-NL, the water level is more than 1 m lower than for the wave conditions normative for wave overtopping. This results in a proportionally larger effect of bottom friction and depth-induced wave breaking such that an increase in bottom friction due to the implementation of Madsen bottom friction and/or the increase in foreshore height would have more influence on the significant wave height. This effect is sustained in multiple studies such as the study of Vuik et al. (2016).

In addition to the previous, especially the increase in foreshore height shows a greater influence on the significant wave height during lower water levels and the increase in density of the vegetation on the foreshore shows a larger influence during greater water levels. This effect is also detected in the study of Vuik et al. (2016) where the statement is substantiated by the presence of a decrease in depth-induced wave breaking due to relatively small wave height to water depth ratios such that the wave attenuation by vegetation becomes the dominant mechanisms. This effect can be seen when comparing the effect of an increase in foreshore height with the implementation of Madsen bottom friction for wave overtopping and erosion. As seen in Figure 30 those scenarios result in a similar significant wave height at the dike toe for the wave overtopping scenario. On the other hand, during lower water depths, occurring on the foreshore with the failure mechanism of erosion, the foreshore height increase shows a far greater influence than including Madsen bottom friction in the erosion scenario (Figure 31). Moreover, denser vegetation shows to have a larger effect on the reduction of significant wave height than an extra increase in foreshore height for a higher water level whereas the opposite is true for a lower water level (compare green and the light blue with the purple scenario for the two failure mechanism). Doing so, it can be concluded that during high water levels, bottom friction is the prominent physical process for the reduction in wave height whereas during lower water levels the reduction in wave height due to depth-induced wave breaking takes the lead.

4.3.2 Wave overtopping

By implementing the different scenarios, the wave conditions will differ leading to a change in the required dike height to fulfil the maximum allowed overtopping discharge of 10 l/s/m. By fulfilling this norm, the dike crest and inner dike slope are ensured not to be eroded to prevent failure of the dike. Since no significant difference can be detected between the hydraulic boundary conditions of scenario 4 and scenarios 5,6 and 7, it is decided that the results for the dike dimensions to prevent failure due to wave overtopping or erosion of scenarios 5,6 and 7 will not be presented to prevent an overload of information.

When the significant wave height decreases in the different scenarios as seen in Figure 30, the dike can be made lower to still fulfil the maximum allowed overtopping discharge. As seen in the formulas stated in equation 3 also the peak wave period and the angle of incidence influence the required dike height. In general, it can be said that the peak wave period follows the trend of the significant wave height such that a lower significant wave height results in a lower peak wave period. The angle of incidence is more or less the same for the different scenarios and will therefore not influence the required dike height.

In Figure 32 the results of the change in significant wave height and peak wave period on the required dike height can be seen. As seen, implementing the different scenarios, a small reduction in the required dike height compared to the standard scenario occurs. However, more promising reductions in required dike height can be seen if the outer dike slope is reduced. With a change in the outer dike slope, more wave energy will be required for the waves to run up the dike slope and overtop the dike crest (EurOtop, 2018). This effect can clearly be seen in Figure 32 when looking at the different outer dike slopes visualised with the different blue bars. For the standard scenario, a reduction in dike slope from 1:5 to 1:7 already results in a reduction in required dike height of 30 cm.

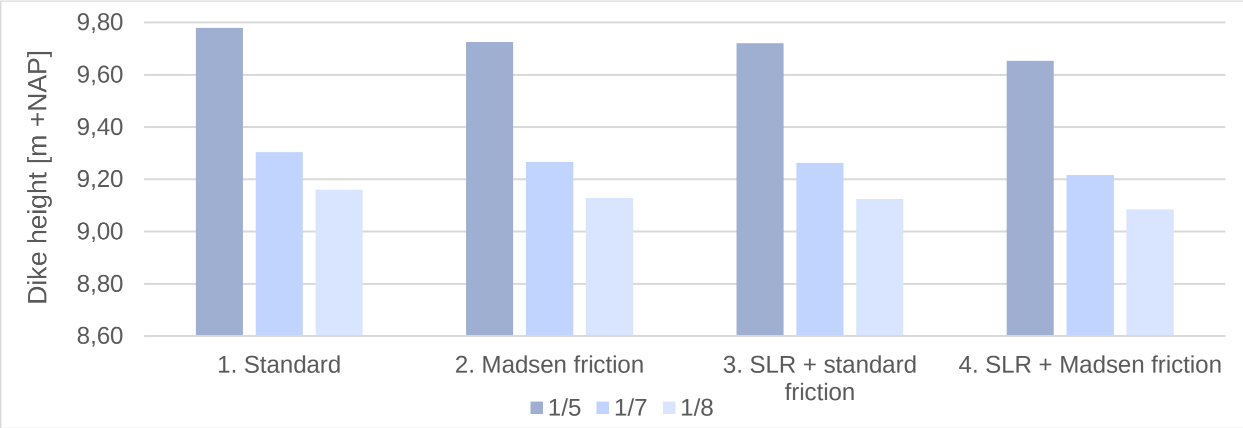


Figure 32. Dike height for wave overtopping for the different scenarios and dike slope

4.3.3 Erosion

To prevent dike failure due to erosion of the outer slope, the clay layer needs to have a certain thickness such that it can cope with the wave energy inserted on the outer slope when the waves will impact the dike. With the WGD principle however, the slope may erode as long as the sandy core of the dike will not be reached. However, the larger the erosion volume of the dike, the greater the repair cost of the adaptive dike. Finally, the erosion length of the erosion pit may become of importance when it has such a length that the crest of the dike will be reached. If this occurs, the width of the dike crest becomes important in the dike design as well such that the dike height will not be decreased due to erosion of the outer dike slope. If done so, the chance of failure due to overtopping will increase.

As shown with the erosion equations before, the excel model of Sweco can calculate the erosion volume as well as the erosion depth erosion length due to wave impact. This calculation is done for the different scenarios from Table 4 and for the original slope of 1:5 and the adjusted slopes of 1:7 and 1:8. In Figure 33 the different output values can be seen for the different scenarios and slopes.

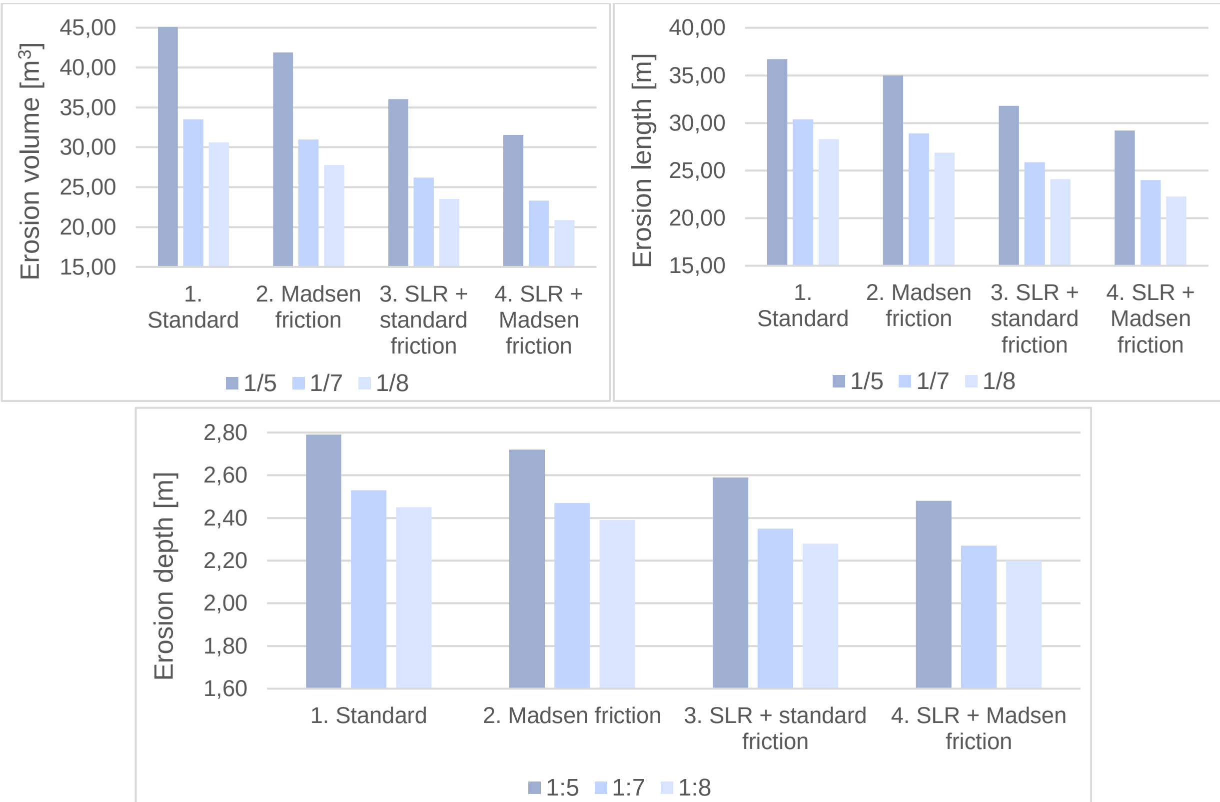


Figure 33. Results erosion hole and clay layer thickness for the different scenarios and dike slope

In Figure 33 it is shown that the parameters that define the erosion hole are connected to a great extent since they all result in the same pattern in the decrease in values for the different scenarios. Moreover, it can be seen that for the different scenarios, the dimension of the erosions pit decreases significantly.

To visualise the location and shape of the erosion pit to determine if the crest of the dike is being reached, the shape of the pit can be drawn with the help of the Excel model as well. An example of the erosion hole for the standard scenario and an outer dike slope of 1:7 is shown in Figure 34 where the light green lines represent the erosion pit. As seen, the crest will not be reached for the normative water level of 7.17 m +NAP when the dike height required to prevent failure due to wave overtopping is implemented.

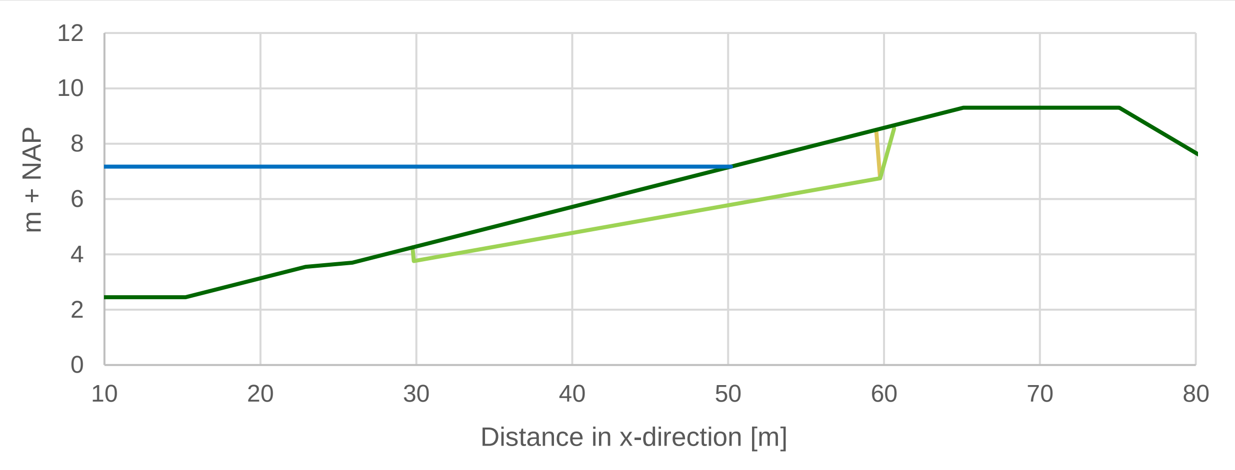


Figure 34. Erosion hole standard scenario and outer dike slope of 1:7

4.4 Discussion

The greatest discussion point of the second sub-question lies within the way the output of the SWAN calculations of the different scenarios is scaled to the Hydra-NL output of the standard Hydra-NL run (standard scenario). A way to calibrate the SWAN model with Hydra-NL calculations had to be found as the probabilistic calculation of wave conditions of the SWAN model is missing. Hereby, there was chosen to scale the wave conditions of the SWAN output to the Hydra-NL output based on percentual differences in the wave conditions. However, since it was not possible to implement the scenarios in Hydra-NL due to the black box character of Hydra-NL, the output of the different scenarios are scaled with the same percentages as resulted with the standard model run. Still, it is expected that when the different foreshore dimensions and vegetation inclusion are implemented in Hydra-NL directly, different percentual differences would occur between the probabilistic Hydra-NL and deterministic SWAN output due to the change in the normative storm conditions in Hydra-NL. Doing so, the results of the different dike reinforcement strategies to prevent erosion of the inner and outer dike slope would look (slightly) different.

Besides the fact that it is doubtful how well the scaling procedure works for the different scenarios, the uncertainties for the implementation of vegetation as stated in the first sub-question remain. The choice of the value chosen for increased bottom friction of Madsen still depends on an assumption of the quality/type/location of vegetation present on the foreshore. Hereby, for the bottom friction, a value equal to 4 m of water depth on the foreshore was used as also used in the first sub-question (Wamsley et al., 2010). However, for the normative conditions of wave overtopping a water depth between 5.4 and 6.4 meter arises on the foreshore depending on the scenario. For erosion, this water depth is between 5.3 and 4.3 meter. Doing so, the Madsen bottom friction should have been higher in the SWAN model as the roughness height increases with increasing water depth according to Wamsley et al. (2010). As a result, the inclusion of vegetation would be more effective for lowering the significant wave height as implemented in the SWAN model of the study resulting in a reduction in required dike height and clay layer thickness.

Another interesting discussion point is the fact that an increase in foreshore height by 75 cm results in about the same significant wave height at the dike toe as the implementation of Madsen bottom friction with the wave overtopping conditions. Where the inclusion of the Madsen bottom friction the significant wave height decreases when to waves move towards the dike, the inclusion of the increase of foreshore height shows an increase in significant wave height which seems to be rather strange. For the failure mechanism of erosion, this effect could not be detected. Although it could be explained that the inclusion of Madsen bottom friction has more influence on decreasing the waves approaching the dike than an increase in foreshore height due to the greater effect of increased bottom friction by vegetation compared to depth-induced wave breaking under the presence of higher water depths, no explanation could be found focussing on the increase in wave height over the foreshore for the increased foreshore height scenario of wave overtopping. However, the fluctuating significant wave height resulting in the scenarios where an increase in foreshore height is included, which is more pronounced for the wave overtopping scenarios than for the erosion scenarios, could point out some discontinuities occurring in the SWAN model due to the way the foreshore height increase is implemented. By increasing the foreshore height, the slope of the foreshore cliff increases since no width increase is included. In the scenarios for the 1/3.000 year storm, as shown in Figure 25, this effect could however not be (clearly) detected. This could be the case however since for those scenarios a foreshore height increase of only 25 cm was applied such that a less steep foreshore cliff would arise.

The last discussion point concerns the space and money needed for the different design options. However it seems to be more beneficial to increase the outer dike slope to 1:8, since this lowers the required dike height and clay layer thickness, the space needed at the seaward side of the dike increases as well as the needed material/money needed to reduce the dike slope. Since the foreshore of the WGD is located on Natura2000 area, severe restrictions are set on what is allowed to build on the foreshore. Therefore, constant trade-offs must be made between the effects of increasing the outer dike slope on the dike design and the set space/area restrictions and budget.

4.5 Conclusion

Due to the high water levels occurring for the normative storm of wave overtopping, the influence of the foreshore on the significant wave height is limited as already seen in Figure 30. By implementing the expectation of the foreshore to increase with SLR (scenario 2), a decrease in significant wave height at the dike toe of about 6 cm occurs for the 1:5 slope resulting in the dike height being 5 cm lower compared with the standard scenario. This influence decreases when the slope decreases. If in addition an extra height increase is implemented by TLP or the foreshore width is increased by human intervention, compare scenario 4 with scenarios 6 and 7, no significant decrease in hydraulic boundary conditions occurred. If however in addition to height increase with SLR the method of the model is changed such that it can account for the influence of vegetation, the waves could be reduced about 7 cm more (compare scenario 3 with scenario 4). When a decision has to be made between an extra increase in foreshore height or making adjustments to the type of vegetation, based on the results shown in Figure 30, it can be stated that an investment in the type of vegetation would be more beneficial for this situation since this has a greater effect on lowering the significant wave height than the extra foreshore height. Based on the results as shown in Figure 32 however, the biggest improvement to reduce wave overtopping remains in the shape of the outer dike slope as a slope of 1:7 results in a reduction in significant wave height of 25 cm compared to the outer dike slope of 1:5.

Regarding failure due to erosion of the outer dike slope, it can be stated that the height of the foreshore influences the significant wave height at the dike to a far greater extent than for the normative wave conditions occurring for wave overtopping as seen when comparing the third scenario with the first scenario in Figure 30 and Figure 31. Due to the smaller water depths occurring for erosion, depth-induced wave breaking is the prominent mechanism able to lower the waves travelling over the foreshore. Doing so, the importance of the foreshore to be able to grow with SLR becomes even more important than firstly stated to prevent failure due to erosion. To do so, it might be interesting to install brushwood dams and ensure that the foreshore is covered with vegetation that will be able to catch sediment when the foreshore floods. If it results that that is not enough, it might be advised to manually increase the height of the foreshore by TLP. By ensuring the foreshore height increase with SLR, a reduction in the required clay layer thickness of 20 cm arises compared to when no height increase is assumed for a 1:5 outer dike slope. For the implementation of vegetation in the model there can however still be stated that this will result in a significant extra reduction compared to when only foreshore height increase is included. Also for erosion, chancing the outer dike slope has a great influence on reducing the dimension of the erosion pit such that a reduction in clay layer thickness can be applied and less clay needs to be replaced after a storm has occurred. When the foreshore can grow with SLR and the bottom friction according to Madsen is applied for the current outer dike slope of 1:5, the thickness of the clay revetment should be as thick as the current calculation with an outer dike slope of 1:7. This shows the great potential of the foreshore to lower the dimensions of the dike that are needed to overcome erosion of the outer dike slope.

The most important lesson learned from translating the wave conditions of the different scenarios to the needed dike design to prevent erosion and wave overtopping, is the great importance of the foreshore to (be able to) pace with SLR. Moreover, a major opportunity of limiting the dike dimensions occurs when the method of the used SWAN model can account for the effect of vegetation on lowering the hydraulic boundary conditions at the dike toe.

5. Implementing the foreshore potential in the assessment and design procedure of a primary flood defence

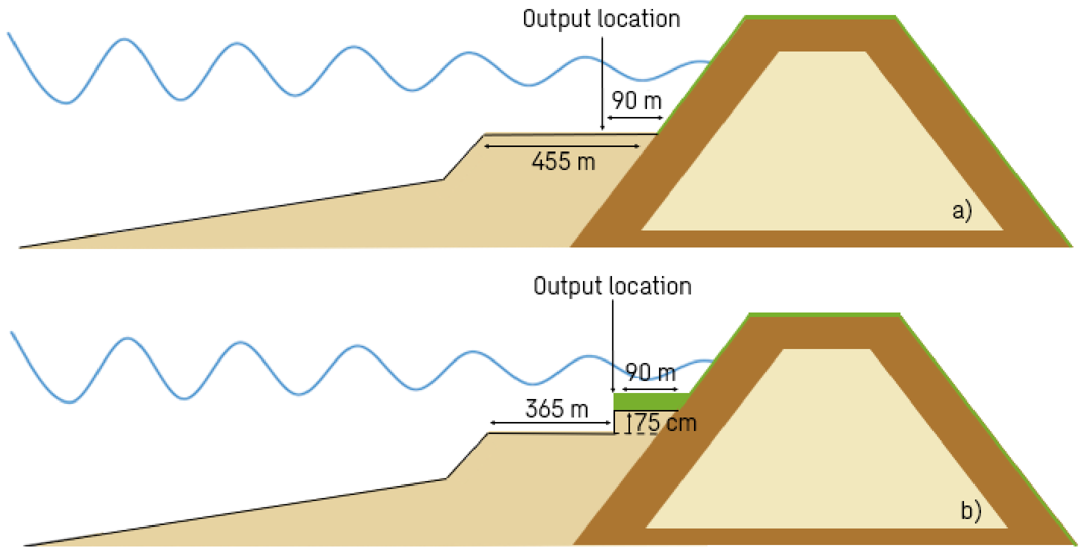
As stated before, Hydra-NL determines the hydraulic boundary conditions at about 50-100 m in front of the dike toe. However, between the output location and the dike toe, a dam or other foreshore characteristics may be present which can lead to a significant change in wave conditions over the stretch from the output location to the dike toe. Based on the results of the previous sub-questions it was concluded that it is indeed very interesting to include the foreshore in the flood defence due to its potential in influencing the hydraulic boundary conditions at the dike toe. By doing so, the flood defence could succeed in assessment criteria it did not succeed before or the dike dimensions could be reduced in the reinforcement design.

In the last sub-question, the challenge was to include the potential influence of the foreshore, as a result of both a future increase in foreshore height (75 cm over 90 years) as the inclusion of the effect of vegetation, into the current assessment and design procedure for primary flood defences. To be able to include the foreshore potential, different implementation approaches are proposed. Hereby, all the approaches are accompanied by their advantages and disadvantages.

5.1 Method

To be able to include the beneficial processes of the foreshore on the hydraulic boundary conditions, there will be elaborated on different approaches of which can be implemented easily or with some effort in the current assessment and design process of primary flood defences in the Netherlands. Besides the effort to implement, the approaches will be scored on the following criteria: accuracy of the model output based on the model calculations, the way the foreshore is represented with the use of the approach (both in dimension as in vegetation), the limited amount of uncertainties accompanied based on the implementation of the foreshore in the procedure and the flexibility of the foreshore input/the ability of the approach to take into account the mitigating behaviour of the foreshore. At the end of the result section, the approaches have been scored on the different criteria both for assessment and for design. The different approaches are visually shown in Figure 35 where they are divided in:

a) Not considering changes to foreshore dimension or vegetation at all since the input bathymetry will not be changed and no difference in bottom friction will be assumed (current most used approach)
b) Make use of the Dam and Foreshore (DaF) module in Hydra-NL as designed for
c) Fictively increase the foreshore width in the DaF module of Hydra-NL
d) Move the output location of the Hydra-NL towards the foreshore fringe before using the DaF module
e) Use a separate SWAN model from the output location on the foreshore fringe onwards



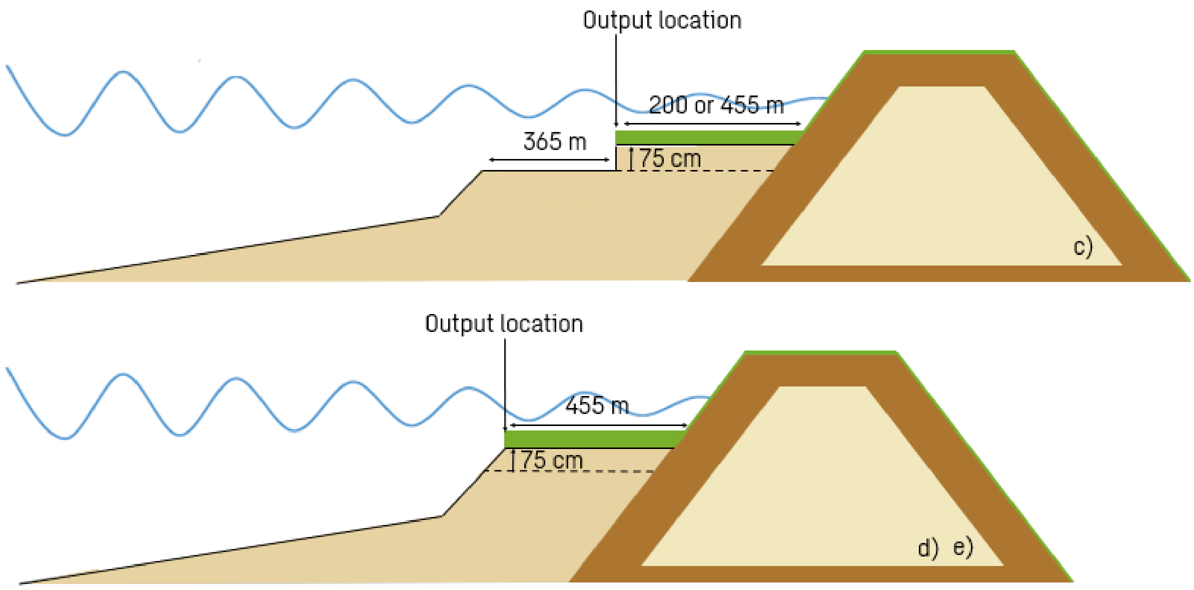


Figure 35. Visualisation of the different implementation methods; no inclusion of foreshore vegetation or increased height (a), use DaF module current approach (b), fictively increase foreshore width (c), replace output location in Hydra-NL whereafter implement the DaF module or a use separate SWAN model (d and e).

For the first three implementations, calculations were conducted in Hydra-NL (with the DaF module) to investigate the influence of different foreshore configurations on the Hydra-NL output. This output could then be compared with the accompanying results of the SWAN model developed for this study to investigate the accuracy of the implementations. For the last two implementations, it was unfortunately not possible to implement these approaches in Hydra-NL/the DaF module since it is not possible to change the output location. However, it is expected that due to a longer distance of increased foreshore with vegetation can be taken into account, the values will be much closer to the calculated wave heights with the SWAN model, especially for the final approach since this approach is also using the 2D model settings.

5.2 Results

In the next sections, the approaches as visualised in Figure 35 will be discussed upon their usability and accuracy (with reality) and a suggestion will be given on when to use which approach. The final section resembles the functionality of the different approaches based on the different criteria.

5.2.1 Not considering vegetation or changes in foreshore dimensions (a)

If with the assessment of a dike section the dike fulfils the safety norms, it is not necessary to take the potential of the foreshore to influence the hydraulic boundary conditions into account. Doing so, Hydra-NL could be used to its full 2D potential and no other calculations or configurations are needed to be made which could increase the uncertainty in model output. However, also when a negative influence of the foreshore is expected, due to erosion of the foreshore (in height or width), cannot be considered since the bathymetry used in the background model of Hydra-NL uses the bathymetry of 2010 and is thus not adapted to a future increase/decrease in foreshore dimensions. Concluding, no effort is needed to implement this approach and since no changes are made to the output generated at the output location there is no uncertainty in the output based on the input of the approach. However, since no changes can be made to the foreshore dimension and it is not possible to include vegetation this approach is not able to include the foreshore potential.

If within the design process it is expected that the foreshore dimensions will not change significantly over the design period and a neglectable effect on increased bottom friction is expected due to the location of the foreshore or the conditions of the current/future foreshore, it might be advised to use the standard output of Hydra-NL. Moreover, by making use of one of the other approaches, an assumption is being made about the change in foreshore dimensions and the state and type of the vegetation. Doing so, an extra risk arises concerning the ability of the foreshore to influence the hydraulic boundary conditions. If the foreshore does not behave as assumed since the foreshore is not able to grow along with SLR for example, an additional intervention needs to be done to make sure the flood defence suffices the assessment. The risk of the possible extra effort may not outweigh the benefit of a limit in dike dimension.

5.2.2 Dam and Foreshore module current approach (b)

When the dike does not meet the safety standards in the assessment process and the foreshore has such a width and vegetation present such that a decrease in hydraulic boundary conditions over the foreshore is expected, it might be interesting to make use of the DaF module in Hydra-NL. Since this module is already present in Hydra-NL, this approach is very easy to implement in the current assessment and design procedure. However, the current implementation of the DaF is designed such that it can be used to transform the hydraulic boundary conditions from the output location in Hydra-NL to the dike toe and not transform the conditions over the full width of the foreshore. Doing so, the influence of the potential increase in foreshore height or the inclusion of the effect of vegetation on the foreshore can only be taken into account for the few tens of meters between the output location of Hydra-NL and the dike toe if the DaF module is begin used as designed for. This explains that the representation of the foreshore dimensions and vegetation on the foreshore is limited. In Figure 36 a schematisation of the objects and transformation of wave conditions from the output location to the dike toe is shown. However, since the foreshore in front of the output location is quite wide, a great part of the foreshore potential is still neglected when implementing this approach.

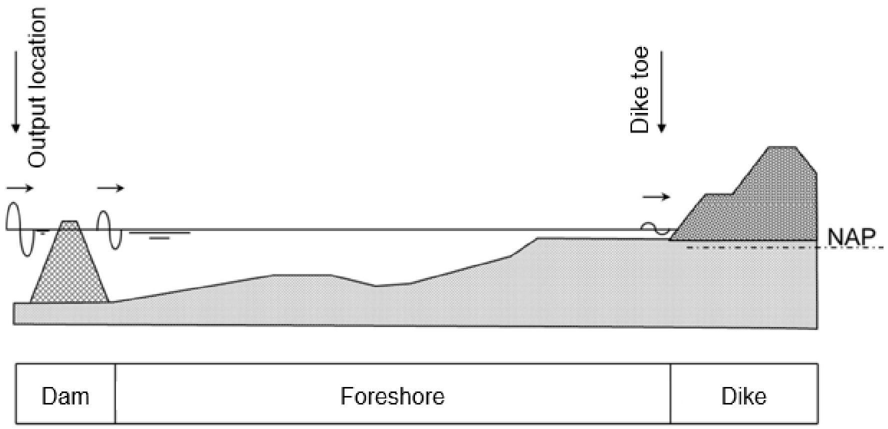


Figure 36. Schematisation Dam and Foreshore module Hydra-NL (Rijkswaterstaat, 2019)

Although the height, as well as the width of the foreshore, can be chosen in the DaF module, another disadvantage of using this approach is that no choices can be made in the type of vegetation present on the foreshore. Together with the lack of representation, this leads to a lack of the approach to be able to determine the wave attenuation by the foreshore.

The DaF module solves the 1D energy balance equation by taking into account the change in wave characteristics caused by depth-induced wave breaking (formulation of Baldock et al. (1998) revised by Janssen & Battjes (2007)), bottom friction (formulation of Jonsson (1966)) and shoaling (Kramer, 2017). Moreover, the change in wave angle due to refraction is included (Kramer, 2017). However, the change in wave height due to refraction as well as the effect on the water level by wave set-up and local wind set-up are not taken into account in the module (Kramer, 2017). If dams are present between the output location and the dike toe, also the effect on the wave characteristics caused by transmission over the dam is included in the calculation conducted within the DaF module. The overall 1D character of the DaF module decreases the accuracy of the 2D output generated at the output location.

To investigate the influence of using the DaF module, different calculations are done in the DaF module with different foreshore configurations. The first configuration represents the basic model run in SWAN wherein no changes in foreshore height and bottom friction are implemented due to the presence of vegetation. In Hydra-NL this configuration is equal to not taking into account the DaF module at all (implementation explained in 5.2.1). Thereafter, two configurations wherein the 90 meter present between the output location and the dike are set up wherein for one configuration the current bottom level is assumed and in the other configuration, an increase in foreshore height with SLR is included.

In Table 8 the comparison between the significant wave height conducted with the SWAN calculations and the calculations conducted with the DaF module of Hydra-NL are shown. Hereby, the first configuration represents the situation without considering the DaF module and the second and fifth configurations represent the use of the DaF module as designed for. As seen, the inclusion of the DaF module with a foreshore width of 90 meters does not result in such promising decreases in significant wave height when compared with the reduction in significant wave height as obtained with the SWAN calculations. Based on these results it would only be advised to use the DaF module for the failure mechanism of erosion. Due to the lower water depths, the increased bottom friction in the DaF module could have a significant influence on the significant wave height. Although the effect of the foreshore, especially when an increase in foreshore height is assumed in configuration 5, is still highly underestimated by the DaF, it does result in a decrease in significant wave height of 16 cm for the failure mechanism of erosion. This could still be the difference between approval or disapproval of the dike section or a significant decrease in dike height or needed clay layer. For the failure mechanism of wave overtopping the underestimation of the DaF module results in such a low decrease in significant wave height that it will not contribute to the approval of a dike section or a lowering of the dike height or a decrease in the clay layer thickness of the outer dike slope to prevent erosion.

Table 8. Results SWAN vs DaF

	Height foreshore [m +NAP]	Width foreshore in DaF module [m]	Significant wave height DaF [m] Wave overtopping	Significant wave height SWAN [m] Wave overtopping	Significant wave height DaF [m] Erosion	Significant wave height SWAN [m] Erosion
1	1.9	0	2.44	2.44	2.77	2.77
2	1.9	90	2.43		2.72	
3	1.9	200	2.42	2.37	2.68	2.68
4	1.9	455	2.39		2.62	
5	2.65	90	2.42		2.61	
6	2.65	200	2.39	2.30	2.52	2.39
7	2.65	455	2.35		2.39	

5.2.3 Dam and Foreshore module fictively increase foreshore width (c)

To be able to increase the width of the foreshore for which the DaF module can be applied, an option is to fictively increase the width of the foreshore by extending the width of the foreshore in the DaF configuration. Doing so, the influence of a vegetated foreshore could be taken into account for a greater foreshore width. However, since the more accurate 2D results of the Hydra-NL calculations are ruled over by less accurate 1D calculations of the DaF for a longer distance it is advised in literature to only use the DaF module for a maximum foreshore width of 100 to 200 meter (Duits, 2020). Still, to investigate the influence on the significant wave height when the total foreshore width of the foreshore could be used to lower the significant wave height, the width of the foreshore has been extended to 200 m, the maximum according to the Hydra-NL guideline, and to 455 m, the “real” width of the foreshore.

Doing so, the third and sixth configurations are set up with the maximum width of the foreshore in the DaF module according to the Hydra-NL manual. To further expand the inclusion of the effect of the vegetated foreshore, the fourth and seventh configurations represent the full width of the foreshore as used in the SWAN model. Hereby, the influence of increased bottom friction by the presence of vegetation and the potential increase in foreshore height with SLR can be taken into account over the full width of the foreshore instead only over the last 90 meters. Still, for these last two configurations, an extra width increase of the foreshore of 365 meters (455-90 m) is present in the Hydra-NL calculation. Since by doing this as well a sudden foreshore height as width increase, is implemented, the foreshore representation is not accurate. On the other hand, since it is possible to include the effect of vegetation on a higher located area, the ability of the method to mitigate wave conditions over the foreshore increases.

As seen in Table 8 the configurations where 200 m of foreshore is assumed (configurations 3 and 6) already show more promising results than the configurations implemented with 90 m of foreshore. However, still with the wave overtopping failure mechanism the reduction in significant wave height remains minimum. Only when including the full width of the foreshore (455 m) the significant wave height kind of reaches the significant wave height as calculated with the SWAN model. When looking into failure due to erosion of the outer dike slope the reduction in significant wave height even seems to be overestimated if the full width of the foreshore is included for the assumption that no increase in foreshore height is present (configuration 4).

5.2.4 Move output location Hydra-NL before using DaF module (d)

By moving the output location of Hydra-NL towards the foreshore fringe, the foreshore becomes part of the primary flood defence and the total width of the foreshore could be implemented in the DaF module without fictively increasing the foreshore width. However it takes some effort to implement this approach in the current assessment and design process since the Hydra-NL calculations are needed to be conducted again at a different location, the foreshore can be represented accurately. A disadvantage of this implementation however is that it decreases the accuracy of the model output since the 2D calculations done in Hydra-NL will be conducted for a larger distance from the dike toe from where the 1D DaF module will take over the calculation. Moreover, no changes can be made to the way the vegetation mitigates wave conditions over the foreshore Concluding, a trade-off needs to be made between the accuracy of the 2D calculation till about 90 m in front of the dike toe or the inclusion of the effect of vegetation and/or potential increase in foreshore height due to SLR.

5.2.5 Separate SWAN model (e)

To retain the 2D model accuracy of Hydra-NL towards the dike toe and to be able to include (more favourable) vegetation characteristics, a separate 2D SWAN model could be conducted. This model can be used to calculate the transformation in hydraulic boundary conditions from the output location of Hydra-NL, at the foreshore fringe, towards the dike toe. By doing so, the disadvantage of the previously stated implementation method could be countered. However, the major disadvantage of this method is that the separate SWAN model needs to be made for the specific location making the implementation a customised assessment which increases the needed time, and money, for the assessment/design process. For example, a large server is needed to be able to probabilistic determine the transformation of hydraulic boundary conditions over the foreshore. However, when this results in the succession of the assessment of a dike section that otherwise needed to be reinforced, this time and money are quickly recouped. Also when the dike still needs to be reinforced, but to a much lesser extent, it may still be worth the time and money to build the SWAN model.

5.2.6 Comparison different approaches

Based on the above-mentioned and elaborated approaches the pros and cons have been summed up in Table 9 based on the criteria as shortly introduced in section 5.1. In the table, the criteria have both been scored for the use of the approach in the assessment and design of a primary flood defence with a vegetated foreshore. To be able to make a recommendation about which approach should be used for a WGD assessment/design where a wide foreshore is present, a sum is conducted such that the overall score of the approaches can be compared. However, since not all criteria are equally important to determine which approach to use, the scores of the different criteria are weighted. This weighting, as well as the score of the criteria, may differ depending on the goal of the assessment/design as well as the state of the present foreshore (dimensions and vegetation). For the scores and weighting as applied for the different approaches in Table 9, the goal of the approach to be able to implement the foreshore potential in the assessment and design weighs heavily due to its great influence on reducing the hydraulic boundary conditions. By doing so, vegetated foreshores can support the application of wide green dikes in The Netherlands. Moreover, since in general it is assumed that a WGD is supported by wide a foreshore, the uncertainty of the inclusion of the foreshore potential is less than when a smaller foreshore is present since a reduction in foreshore width has not that great of an influence in the mitigating capacity of the foreshore.

Table 9. Criteria analysis of the different approaches to include the foreshore potential (first column based on assessment (Ass.), second based on design (Des.))

	Weight	No inclusion foreshore potential (a)		Use of DaF module – no changes (b)		Use of DaF module – fictively increase foreshore width (c)		Use of DaF module – move output location (d)		Move output location + use separate SWAN model (e)	
		Ass.	Des.	Ass.	Des.	Ass.	Des.	Ass.	Des.	Ass.	Des.
Implementation	1	++	++	+	+	+	+	-	-	--	--
Accuracy based on input	1	++	++	+	+/-	-	--	+/-	-	++	++
Representation foreshore dimensions and vegetation representation	2	+/-	-	+	-	-	--	++	++	++	++
Limit in total uncertainty	1	++	++	+	+	-	--	+/-	-	+/-	-
Flexibility of input / Ability to implement wave attenuation by the foreshore	3	+/-	--	+	-	+	-/+	+	+	++	++
Total score		+/-	--	+	--	-	--	+	+/-	++	++

For the assessment, it is assumed that the foreshore dimensions are not differing a lot from the bathymetry as used in the calculation of the hydraulic boundary conditions. Doing so, the lack of foreshore representation is only focused on the representation of the vegetation. As a result, the importance of using a more complex approach, which is harder to implement, is more important for the design process than for the assessment of a flood defence.

As seen in Table 9, the first three approaches score (very) good concerning the implementation in the current assessment and design procedure since Hydra-NL offers the DaF module such that the width of the foreshore can be determined easily by the user. For the other two approaches changes to the output location has to be made which decreases the ease of the implementation. For the final approach, a separate SWAN model must be built which is time-consuming to be made and hard to include in the current assessment and design procedure. Since the score of the implementation does not depend on assessment or design, the approaches score the same.

The accuracy of the different approaches is focused on the way the approach calculates the changes in hydraulic boundary conditions from the output location of the Hydra-NL output. Since the first approach does not change this output, the score of accuracy is very high. This score is also seen for the last scenario since this approach makes use of the same 2D calculations as done in the background model of Hydra-NL ensuring the accuracy of the approach. Since the other approaches calculate with 1D model principles, for a shorter or longer foreshore distance, there will be a decrease in accuracy of the Hydra-NL output at the output location compared with the model input before the approach is used.

As for the criteria where the foreshore representation is scored, the score is based on the way the approaches represent the foreshore dimensions and present vegetation. To do so, the visualisations of the approaches as seen in Figure 35 are used. As for the first approach, the foreshore lacks height increase and vegetation. For the second approach, the sudden height increase at the output location is not realistic. The third approach includes the sudden height increase as well as an increase in width of the foreshore which do not result in a good representation of the foreshore. Doing so, the worst score for this criterion of all the approaches is given to approach c. The last two approaches both reach the optimal score since they represent the foreshore height increase and vegetation over the full width of the foreshore.

By increasing the accuracy of the foreshore representation, the uncertainty in the conclusions of the chosen approach increases since the resilience of the foreshore to grow along with SLR and the ability of the vegetation to mitigate wave conditions is not certain. Moreover, the wave mitigating capacity of the vegetation is assumed to be the same for the whole area where the vegetation is modelled to be present in the different approaches. However, the vegetation type or the quality of the vegetation may not be the same over the whole foreshore and might not be the same in every season.

The score of the flexibility of the input parameters, which ensures the ability of the approach to account for wave attenuation by foreshores, is based on the way the foreshore dimensions and vegetation properties can be included in the different approaches. This increases for the different approaches since the dimensions of the foreshore and the implementation of the total width of the foreshore in the calculation are more and more included when moving to the next approach. The final approach hereby reaches the maximum score since for that approach also changes can be made to the type of vegetation/the value implemented for increased bottom friction if vegetation is present.

Based on the total score assigned to the different approaches, it can be stated that the use of the DaF module in Hydra-NL with the current output locations, approach b, might not be as bad if used for the assessment of primary flood defences with a wide foreshore. Although the influence of vegetation will be underestimated since only implemented for the last 50-100 m of foreshore, the dimensions of the foreshore will be represented accurately. For the design of wide green dikes with a wide vegetated foreshore, it is however strongly advised to make use of a separate SWAN model to be able to maintain the accuracy of the wave calculations over the foreshore while also being able to include foreshore height increase and vegetation (approach e). However, when it is not feasible to construct a separate SWAN model, it is advised to set aside the lack of accuracy of the 1D DaF module calculation and the disadvantage of not being able to make changes to the increased bottom friction factor for vegetation and make use of approach d. By doing so, the potential of the full width of the vegetated foreshore can be included in the design of the primary flood defence system.

5.3 Discussion

The greatest uncertainty of including/excluding the potential of the foreshore to limit the hydraulic boundary conditions at the dike toe in the assessment and design process remains the resilience of the foreshore and the uncertainty in future SLR scenarios. How certain is the capacity of the foreshore to grow along with SLR and how certain is the state and influence of the vegetation when no experiments are conducted under severe storm conditions? However, as motivated in the study of Best et al. (2018), where a marsh-mudflat system was exposed to 100 years of SLR scenarios, the salt marsh was able to survive the SLR scenarios if higher accretion rates were imposed. This could be reached by lowering the flow velocity of water flowing over the foreshore during a flood by installing brushwood dams or increasing the vegetation quality. Doing so, more sediment will settle on the foreshore and the foreshore would increase in height. However, since the resilience of the foreshore cannot be guaranteed, a safety factor should be included to cope with the uncertainty in foreshore resilience if the foreshore potential is included in the assessment or design of a primary flood defence. Besides including a safety factor to cope with the uncertainty in foreshore resilience, it is advised to critically observe the behaviour of the foreshore-dike systems such that action could be taken if the foreshore shows behaviour that is outside the expected/designed boundary. However, since the foreshore-dike combination is a dynamic system, it will be hard to determine when a certain foreshore is no longer able to catch up with its predicted behaviour. When it is decided that the flood defence will no longer be able to recover from certain damage, an additional dike reinforcement or intervention of the foreshore must be conducted such that the assumed influence can be ensured. Hereby, the adaptive capacity of a WGD system with vegetated foreshore is highly valued.

In the SWAN model designed for the study, the vegetation is included explicitly by increasing the bottom friction where vegetation is present with the formulation of Madsen et al. (1988). The value used for bottom friction in the SWAN model (0.041) is based on a water level of 4 meter and the type of vegetation present on the foreshore of the WGD (Wamsley et al., 2010). In Hydra-NL the bottom friction is increased in the DaF according to the formulation of Jonsson (1966). This is done with a fixed value (0.021) and cannot change if the water depth on the foreshore is higher, or the vegetation is more/less dense. In Figure 37 the comparison between bottom friction values for Madsen and Jonsson can be seen. Hereby, it can be concluded that much lower bottom friction is included for the vegetated areas in the DaF module than in the SWAN model. This gives a first explanation for the lower influence of the presence of vegetation in the DaF module compared with the output of the SWAN model.

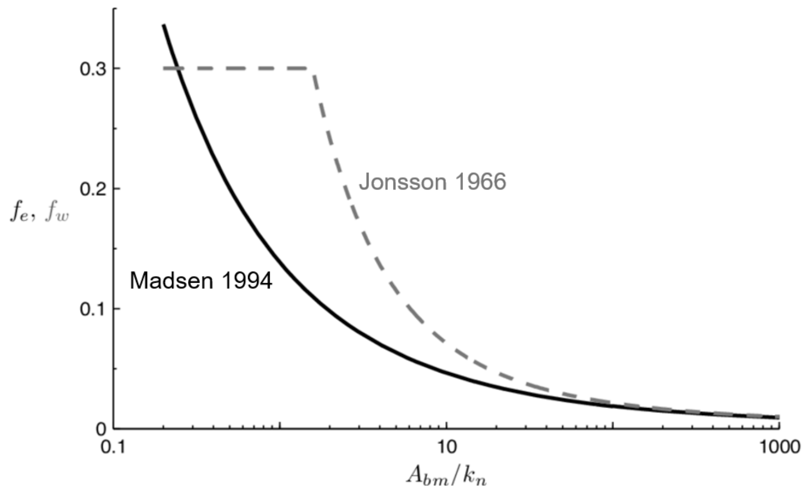


Figure 37. Madsen vs Jonsson bottom friction (Humbyrd & Madsen, 2011)

Besides the underestimation of the bottom friction in the DaF module, another more fundamental issue is present when implementing the DaF module in Hydra-NL. By implementing the DaF module, only after the output location of Hydra-NL, bottom friction due to vegetation and foreshore height increase can be included. This could already clearly be seen in implementations b and c in Figure 35. Doing so, processes that arise due to the presence of the vegetated foreshore in front of the output location are not taken into account. Besides not taking into account the vegetation present before the output location, which is tried to be tackled in implementation c, the increase in foreshore height due to SLR is only accounted for from the output location onwards. As seen in Figure 25 already, the greatest decrease in significant wave height over the segment arises when climbing the foreshore cliff. Therefore, there is expected that increasing the foreshore height in implementations b and c does not have the same influence on decreasing the significant wave height as when the height increase would be present from the start of the foreshore platform onwards as done in the last two implementations.

5.4 Conclusion

When the effect of including a (heightened) vegetated foreshore is expected to significantly influence the hydraulic boundary conditions at the dike toe such that the assessment/design process would benefit from the inclusion of this effect, several implementation methods could be applied.

Which approach is best to implement in the assessment or design procedure is site- and goal specific. With the help of Table 9, a first estimation of the approach that should be used can be made based on the goal of including the foreshore potential of a wide foreshore. When moving towards the next approach, the approach can better represent the foreshore dimensions and vegetation but the effort that needs to be made to include the approach into the current procedures increases.

By taking into account the foreshore potential, the conservative assumption that the foreshore is not able to develop over time or the vegetation is not able to mitigate wave conditions is replaced by the uncertainty of the resilience of the foreshore. To cope with this uncertainty, a safety factor needs to be taken into account if the foreshore potential is included and the behaviour of the foreshore-dike system needs to be monitored. Doing so, a trade-off needs to be made between the adherence of a conservative assessment or design or including the foreshore potential which leads to a limitation in the needed dike dimension but increases the risk of an extra measurement in the longer term. Hereby, also the amount of SLR and other predicted wave characteristics taken into account in the assessment and design is prone to change over the years which can change the decision about the needed reinforcement.

6. Discussion

Based on the different discussion points already stated, the overall meaning of significance of the results of the study came forward. In this chapter, a connection will be made between those shortcomings and the academic as well as practical significance and usability of the study.

6.1 Research

From an academic point of view, some points are needed to be addressed related to certain choices made in the conducted study which lead to limitations or uncertainty in the results of the study. Although the use of a SWAN model to quantify the foreshore potential has been a supported choice since used in multiple studies (e.g. van Zelst et al., 2021; Vuik et al., 2016, 2018; Willemsen et al., 2020), knowledge gained from these, and other studies related to the topic of this study, should be compared with the results found in the conducted study such that conclusions can be drawn. Hereby, first, strong consideration should go to how the changing foreshore dimensions and the influence of vegetation are implemented in other studies. Moreover, since a great uncertainty for the inclusion of the foreshore potential in the flood defence system remains the foreshore resilience, a critical look must be given to this subject.

6.1.1 Changing foreshore height over time

Vegetation on the foreshore reduces the flow velocity of waves moving onshore due to which suspended sediment travelling over the foreshore can settle. As a result, the foreshore will grow in height such that it has the potential to pace with SLR (Temmerman et al., 2013; Willemsen, 2020; Zhu et al., 2020). Since in the design of sea dikes the expected SLR should be taken into account, in the conducted study it is assumed that the foreshore height will increase alongside this SLR. This assumption is supported by multiple studies such as Calderon & Smale (2013) and Vuik et al. (2018). Whereas in the conducted study a fixed value is assumed for the increase in foreshore height, other studies determine the change in foreshore height over time with the use of a marsh deposition model (e.g. Allen, 1990; Marijnissen et al., 2020; Temmerman et al., 2003). Doing so, the accretion on the foreshore depends on the amount of suspended sediment on the foreshore with each tide (Marijnissen et al., 2020). Doing so, by using a marsh deposition model, the foreshore height depends on the available sediment rather than on SLR alone. Due to the complexity of including a marsh deposition model and the increase in parameters needed it was however chosen to use a fixed value for the foreshore height increase. Moreover, since the amount of suspended sediment and the water level of each tide depend on the climate scenario, the SLR and accretion values are still intertwined in the approach used in the conducted study.

However, by assuming a fixed foreshore height increase, it is assumed that the foreshore has the fixed height increase at the start of the design period. In reality, the foreshore has only reached this height at the end of the design period when the amount of SLR is also present. This leads to an overestimation of the foreshore potential in case the storm occurs before the end of the design period since the amount of SLR is lower than the increase in foreshore height at that moment. However, as the implemented wave conditions are also based on the conditions occurring at the end of the design period when the total SLR is present, the implementation of the height increase from the start of the design period might not be as strange.

6.1.2 Implementation of (effect of) vegetation

Related to the influence of the vegetation to mitigate wave conditions under severe storm conditions, some uncertainty remains due to a lack of experiments (van Loon-Steensma, 2015; Zhu et al., 2020). Moreover, as the effect of vegetation on mitigating wave conditions reduces with increasing water depths, as found in this study as well as stated in multiple other studies (e.g. Vuik et al. (2018)), the effect of vegetation remains uncertain and possibly limited.

In the study of Marijnissen et al. (2020), the authors assumed that the vegetation on the worst-case inundated salt marsh was represented by a Manning coefficient for vegetation under winter-storm conditions represented by a roughness height. This approach is also used in the study of Willemsen et al.

(2020) when extreme storm conditions are assumed. Due to the extreme storm conditions, it is assumed that no standing-up vegetation will be present. Since such extreme conditions were also simulated in the conducted study, the assumption that no standing-up vegetation is present is also made in the conducted study. When the vegetation properties were taken into account, such that the standing-up vegetation is assumed, an overestimation of the effect of a vegetated foreshore was shown since no measurements on the drag coefficient were available under extreme storm conditions.

6.1.3 Foreshore resilience

According to Vuik et al. (2016), the capacity of saltmarshes to attenuate waves depends both on the vegetation properties as on the hydraulic characteristics such as wave height and water depth. Since the state of the vegetation under different weather/flood conditions is hard to determine for the vegetated foreshore and only little data is available on the ability of saltmarshes to reduce wave impact under extreme conditions, studies admit that the value of the coastal ecosystems is uncertain (van Loon-Steensma, 2015; Zhu et al., 2020). However, even though extreme storm conditions are less beneficial for wave attenuation by vegetation, its mitigating capacity under these circumstances is still widely supported (e.g. van Loon-Steensma (2015); Vuik et al. (2016); Willemsen (2020)). Thus, although some uncertainty arises in the conducted study for the inclusion of the effect of vegetation on mitigating the hydraulic boundary conditions, a great underestimation would have been made if this influence was not included at all.

The ability of the foreshore to grow in pace with SLR depends on both the sediment availability as well as the amount of SRL. In the study of Marijnissen et al. (2020), it was concluded that the foreshore accretion rate would increase for higher SLR scenarios. However, it was also found that, with the highest predicted SLR scenario, the foreshore cannot catch up in height (Figure 38). As a result, several studies state that foreshores are prone to drown when severe SLR occurs (Best et al., 2018; Marijnissen et al., 2020; van Goor et al., 2003). The biggest cause for vegetated foreshores not being able to pace with extreme SLR is that the vegetation requires a minimum period of dry conditions to survive (Marijnissen et al., 2020). In the conducted study, possible drowning of the foreshore is not taken into account. To prevent drowning of the foreshore, it might be necessary to stimulate accretion rates with the implementation of brushwood dams for example. By doing so, it would be possible for the foreshore to survive SLR (Best et al., 2018).

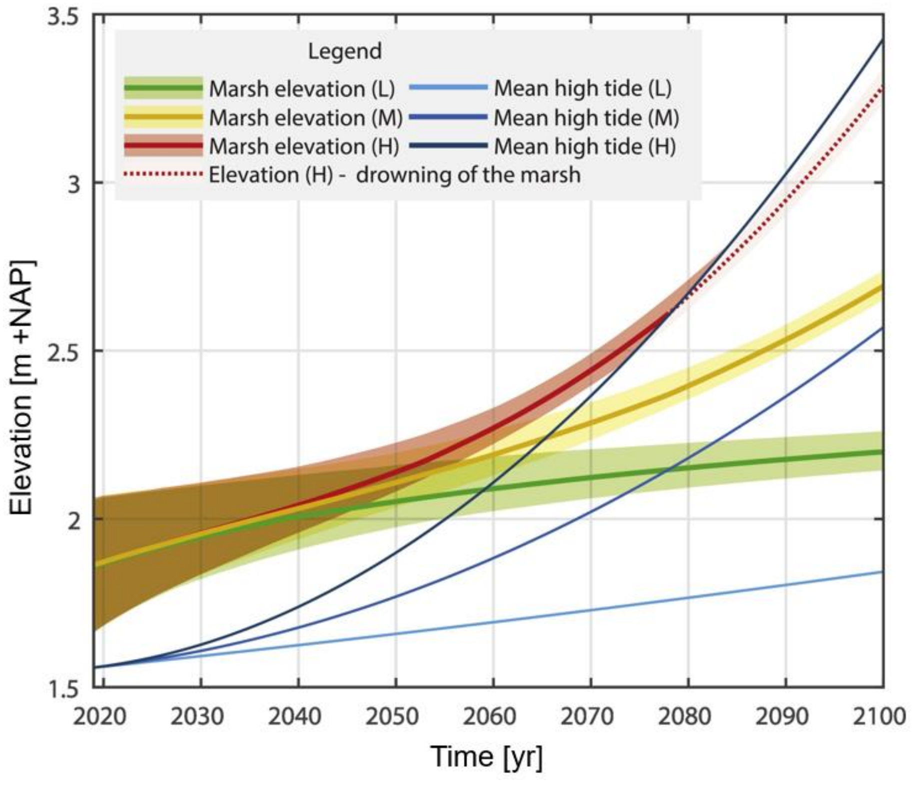


Figure 38. Development of the marsh elevation over time for different SLR scenarios (Marijnissen et al., 2020)

6.1.4 Conclusions from other research

Some interesting results were conducted in the research of Zhu et al. (2020) where the importance of the width of the foreshore was stressed. Based on records, they concluded that with a wider salt marsh, the level of wave run-up, as well as the chance of the dike breaching, decreases (see Figure 39). The general ability of saltmarshes to reduce wave run-up, and thus (failure due to) wave overtopping, is besides widely supported in literature by studies such as Möller et al. (2014), van Wesenbeeck et al. (2017) and Vuik et al. (2016) also found the conducted study. However, the influence of the width of the foreshore on the wave run-up and failure probability could not be detected in this research project. While in the study of Smale (2014) the foreshore did show wave dampening behaviour for the first 1000 m of the foreshore, in the conducted study this was limited to the first 250 m (see Figure 25). This most likely has to do with the strong mitigation of wave height occurring when the waves reach the start of the foreshore cliff where a fast increase in bottom level is present till the foreshore platform is reached (see Figure 16). Since the total width of the foreshore was 650 m, an increase or decrease in width did not influence the wave dampening capacity significantly.

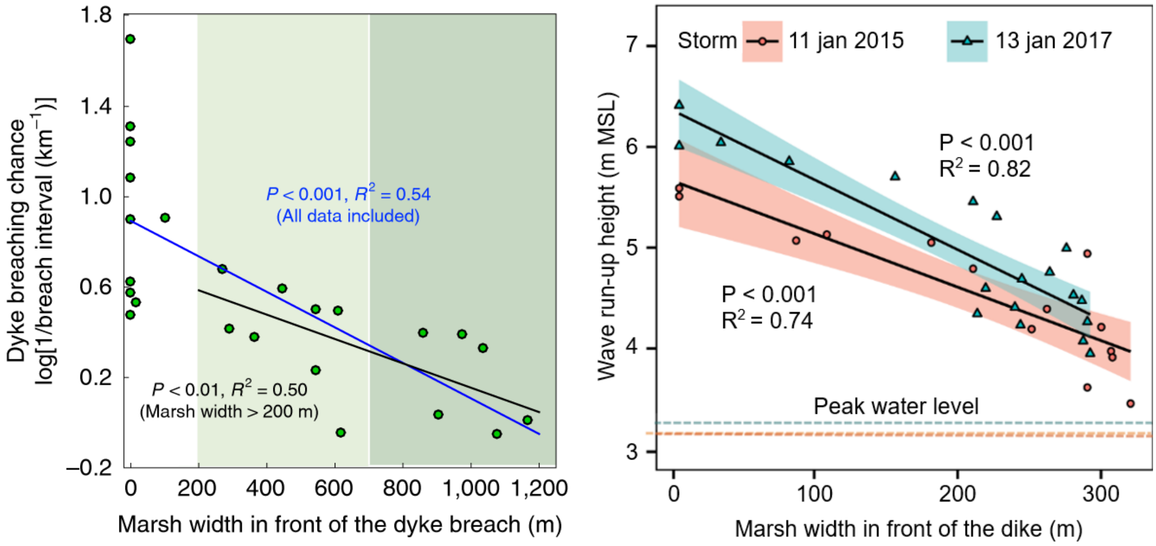


Figure 39. Dike width in combination with dike failure probability (left), Marsh width in combination with wave run-up height (right) (Zhu et al., 2020)

A quantitative comparison with other research could be made with the results of the study by De Boer (2019). De Boer (2019) determined that a reduction in significant wave height of 13-31% occurred for a 1/37.500 year storm if the potential of the 'Holwerd west' salt marsh was taken into account compared with a scenario where no saltmarsh was assumed to be present. The 'Holwerd west' salt marsh is located against the Wadden Sea dike in the northeast of the province of Friesland. As a result, there could have been a reduction in normative dike height of 23-41 cm. Although in the conducted study the biggest reduction was found on the needed clay layer thickness, the reduction in normative dike height would be around 13 cm. Although the results of the De Boer (2019) might be overestimated or the results of the conducted study are underestimated, it is also indicating that the potential of the foreshore to mitigate wave conditions and limit dike dimensions is strongly dependent on the location of the foreshore and the bathymetry and other characteristics of the foreshore.

6.2 Application

A link between the conducted study and application can be made by comparing the results of the final design of the WGD of Sweco with the final dike height and clay layer thickness determined in the conducted study. Resulting from the study of Sweco, a dike height of 9 m +NAP for an outer dike slope of 1:7 and a clay layer thickness of 2.5 m was chosen to prevent erosion of the outer dike slope. If a 1:7 outer dike slope and a foreshore growing along with 75 cm of SLR and Madsen bottom friction is applied for the present vegetation, a dike height of 9.2 m +NAP and a clay layer thickness of 2.3 m are shown for the design following the conducted study (scenario 4). In total, this would result in less material needed since the crest of the dike is less wide than the outer dike slope is long. It has to be mentioned however that the dike height resulting from the study came from the requirement of the maximum allowed wave overtopping discharge whereas the required dike height of the study of Sweco resulted from the outer dike erosion calculation. Doing so, a difference in required dike height is prone to arise as for the wave overtopping conditions, a far greater water level is normative than for erosion leading to a greater dike height. Moreover, the difference in required dike height of the study compared to the result of Sweco could be a result of the year for which the hydraulic boundary conditions are determined. In the design of Sweco, the dike is designed for 2073 whereas the design of the study is made for 2100. By using the predicted hydraulic boundary conditions for 2100 instead of 2073, greater hydraulic conditions are present at the start of the foreshore due to which more is begin asked of the foreshore-dike system. If the dike design of the study was also made for 2073, it is expected that in the second sub-question a dike height lower than 9 m +NAP was found since, among others, a lower water level would be normative.

6.2.1 Use of a foreshore-dike system at other locations

This study was focused on the WGD pilot study in the Ems-Dollard estuary to quantify the foreshore potential and to include this potential in the assessment and design procedure of sea dikes in the Netherlands. However, when extending the findings of this study to other locations where vegetated foreshores are present in front of sea dikes, some discrepancies arise while extrapolating the results. As stated multiple times, as ensuring the foreshore potential of the WGD location, the focus should be on ensuring the increase in foreshore height and the preservation of vegetation rather than focusing on maintaining and especially increasing foreshore width. However, when less sheltered foreshores are exposed to extreme storms, it is expected that higher waves are occurring at the foreshore as also seen for the transect in Germany. As a result, higher but narrower foreshores will occur as also substantiated in the study of Best et al. (2018). If the foreshore reaches a critical width, the ability of the foreshore to actuate waves decreases fast as visualised in the right graph in Figure 28. Doing so, the enhancement of maintaining the critical width of the foreshore might become more important than maintaining the height of the foreshore.

When looking into favourable landscapes and wave conditions the implementation of a WGD in combination with a vegetated foreshore seems to blend well into the Wadden sea coastal landscape due to the shallow tidal sea, sand flats and mudflats, and already present extensive semi-natural salt marshes along the coastline (van Loon-Steensma & Schelfhout, 2017). When favourable abiotic conditions are present (e.g., elevation in relation to tidal range, concentrations of fine-grained sediment and velocity of currents along the coast), there is a great potential for wide green dikes to be successfully implemented due to the mitigating behaviour of hydraulic boundary conditions by the vegetated foreshore as substantiated in the conducted study.

6.2.2 Implementation of foreshore potential in the flood defence system

Despite the aforementioned great potential of vegetated foreshores, the decision to what extent a vegetated foreshore that accretes in line with SLR should be implemented in the assessment and/or design procedure depends on the uncertainty/stability of the foreshore and the severity of assumed SLR. However, since the assessment and design process is strictly defined in the WBI2017 and OI2014, it might cause some difficulties for the proposed approaches to be accepted. Hereby, the uncertainty in the resilience of the foreshores under future scenarios is seen as the main obstacle. However, as seen in the innovation projects currently being executed within the HWBP and within other programs such as the Living Dikes program, a growing interest in including NbS in flood defences is shown. However, small steps have to be taken to make sure no costly reinforcements have to be implemented if the foreshore-dike system shows behaviour that's outside the boundaries the flood defence is designed for. Studies such as this help to get a better image of the potential quantitative influence of including the foreshore in the flood defence to lower dike dimensions. Based on this, the decision has to be made if it is worth the risk of a possible extra (reinforcement) measure or whether it has more benefits to ignore the foreshore potential such that less uncertainty is present. However, due to the adaptive chrematistic of a WGD, it may be worth the risk to make use of the benefits of NbS.

7. Conclusion and recommendations

The objective of this research was formulated as: “To quantify the potential influence of the (changing) foreshore of the Wide Green Dike on hydraulic boundary conditions and dike design and investigate the implementation of the foreshore potential in the assessment and design process of primary flood defences”. For this purpose, the WGD pilot study in the Ems-Dollard estuary was used as a case study.

In this final chapter, the overarching conclusions of the research will be drawn and recommendations will be made towards the academic world (7.1 Research) wherein points for further research are stated and towards the engineering world (7.2 Application) wherein the Dutch government and engineering companies are playing a big role.

7.1 Research

7.1.1 Conclusion

Based on the results of the first sub-question, which was formulated as “*How do different foreshore dimensions, implementations on the foreshore and model settings influence the hydraulic boundary conditions at the WGD?”,* the general influence of foreshore dimensions and implementations on the foreshore on lowering the significant wave height at the dike toe could be shown. Where the foreshore width did not seem to influence the mitigation of significant wave height significantly, most likely due to the great starting width of the foreshore, the water depth on the foreshore, and the inclusion of increased bottom friction, were influencing the significant wave height to a great extent. To determine from which foreshore width lateral erosion would be critical, the critical width of the foreshore should be determined to be able to expend this conclusion to other foreshore-dike systems.

In the second sub-question, the effect of the foreshore under a different foreshore height and vegetation implementation was combined with the effect on the required dike dimensions. Hereby, the expected influence of a higher foreshore could clearly be seen for the needed clay layer thickness. Due to a higher foreshore, the water depth is less due to which the waves are lower as well. Doing so, the wave energy is mitigated such that the energy of the wave impact is reduced leading to a reduction in the required thickness of the clay layer. Although the effect of a higher foreshore did have less effect on the required dike height, the reduction in wave energy also had its effect on this. When the effect of vegetation was included, an extra reduction in required dike dimensions could be seen. For the original 1:5 slope, the dike height could be reduced by 13 cm (from 9.78 to 9.65 m +NAP) while a reduction in clay layer of 31 cm (from 2.79 to 2.48 m) could be reached when the foreshore would be able to pace with SLR and Madsen bottom friction is included. To reduce the required dike dimensions even more, the outer dike should be made less steep, i.e. in the form of a WGD. If the outer dike slope is reduced to a slope of 1:7, a dike height of 9.2 m +NAP and a clay layer of 2.3 m would be required. Based on the 1:7 scenario, the needed clay can be reduced by more than 13 m^3 per linear meter of dike if the foreshore potential, when including height increase and the effect of vegetation according to Madsen bottom friction, is taken into account. For the total length of primary flood protection in the Dollard basin, where a vegetated foreshore is already present (see Figure 9), this would lead to a reduction in more than 133.000 m^3 of clay if a WGD is constructed over this stretch. To sketch an image, this is a reduction of almost 5.000 truckloads for the reinforcement of the Dollard dikes alone.

Despite this promising potential, the implementation of a WGD implies all kinds of implications that should be considered, such as the influence it has on the existing nature values of the adjacent, protected salt-marsh area (van Loon-Steensma & Schelfhout, 2017). Including a vegetated foreshore on the WGD could however help to limit the influence on nature values through lowering of the required dimensions. Moreover, since the grassland vegetation on the dike offers additional space for biodiversity it has beneficial factors over the conventional way of reinforcing with concrete. Finally, since a WGD offers a flexible climate-change adaptation measure which might be cheaper to build compared to the conservative design with conventional reinforcement options, a WGD may be a worthwhile measure to consider if the abiotic conditions for vegetated foreshores are favourable (van Loon-Steensma & Schelfhout, 2017).

7.1.2 Recommendations

If the output location in Hydra-NL can be moved towards the foreshore fringe, it will be possible to quantify the final two approaches stated in the last sub-question. Doing so, a conclusion can be drawn whether moving the output location increases the ability of the DaF module to mitigate wave conditions while not having to construct a separate SWAN model for a specific primary flood defence. However, by calculating the change in hydraulic boundary conditions over the whole width of the foreshore with the less accurate 1D calculations of the DaF module the accuracy of the calculation decreases. To quantify the influence on the accuracy of the model output, a comparison has to be made with the 2D SWAN calculations at the dike toe where foreshore height increases or vegetation are taken into account. If the implementation of the DaF module, when making use of the moved output location, does not significantly differ from the output when using a separate SWAN model, it can be concluded that using the DaF module from the foreshore fringe onward is an appropriate approach to determine the foreshore potential.

Besides further looking into the different approaches to implement the foreshore into the primary flood defence, more research could be conducted regarding the resilience of the foreshore and how to implement this resilience into the assessment and design procedure. Doing so, the safety factor that should be included in the assessment/design procedure if the foreshore potential is taken into account could be based on certain characteristics of the foreshore (e.g. location, bathymetry and vegetation) and SLR scenario. To determine the foreshore under future conditions there also has to be looked at the development of the foreshore under day-to-day conditions since this influences the foreshore dimensions greatly. If this could be done, the possibility of implementing the foreshore into the assessment and design of primary flood defences in The Netherlands could better be substantiated based on the inclusion of the uncertainty.

To be able to design a safety factor for different locations and foreshore characteristics, it has to be investigated to what extent the results of this study can be extrapolated to foreshore-dike systems located at different locations in the Netherlands. This could be reached by conducting the SWAN calculations for different locations where the foreshore bathymetry, as well as the vegetation on the foreshore, vary. For those other locations, which may be less or more sheltered for example, other normative storm conditions will be present. By doing so, the ability of foreshores to affect the required dike dimensions can be based on differing foreshore and location characteristics.

Finally, it would be very useful if more experiments are conducted on the effects of extreme storm conditions on foreshore vegetation. Doing so, the influence of the vegetation on increased bottom friction could be substantiated with experiments and the uncertainty in the additional effect of vegetation under storm conditions could be reduced.

7.2 Application

7.2.1 Conclusion

As for the WGD pilot study, the great potential of the foreshore is shown since including an elevated foreshore height and vegetation on top of that foreshore could significantly limit the hydraulic boundary conditions. If the foreshore does not show the assumed resilience to grow along with SLR, measures can be made to enhance sedimentation on the foreshore. To support sedimentation, brushwood dams or a summer dike could be implemented such that wave conditions will be mitigated. If not enough, TLP can be considered as a measure to increase the foreshore height. However, since there are strong restrictions following the protected Natura 2000 area designation of the foreshores, it is doubtful whether these measures, especially a summer dike and TLP, are allowed. However, if the foreshore dimensions and/or vegetation are/is not present or functioning as was assumed in the dike assessment or design, there can be counted on the adaptive capacity of the WGD by increasing the thickness of the outer dike slope or dike height. By reinforcing the dike with a gentler outer slope with a thick clay layer and making use of the wide vegetated foreshore, it is possible to limit the dike dimensions leading to a more sustainable approach due to the decrease of needed reinforcement material.

However, the acceptance to include a vegetated foreshore in the assessment/design of a flood defence is assumed to be more effective for the WGD location than for locations where higher significant wave heights are normative such as the location in Germany. For those dikes, the foreshore-dike system will be exposed to more extreme floods and storms (higher waves and velocities) such that higher and less wide foreshores occur (Best et al., 2018). Consequentially, the critical width of a foreshore is more liable to be reached if no measures preventing erosion of the foreshore fringe are taken. To conclude this point and generalise the influence of the width of the foreshore, there should be looked critically at the potential development of a specific foreshore before one of the approaches is used to include the foreshore in the current assessment and design procedure.

To include the foreshore potential in the assessment/design of a flood defence, different approaches have been proposed in the last sub-question. Those approaches can be implemented in, or are additions to, the current assessment and design procedure. Some approaches can be implemented quite easily, but others require more effort. If the DaF module present in Hydra-NL is chosen to use, the implementation is relatively effortless, but the results may not be as accurate as those generated with a separate 2D model since the DaF model makes use of 1D calculations. Moreover, in the DaF module, no adaptations can be made to the level of increased bottom friction, consequently, the friction value can be over- or underestimated if the water depth on the foreshore increases/decreases. If a separate SWAN model is used to determine the change in hydraulic boundary conditions over the foreshore, changes in increased bottom friction and foreshore dimensions can be calculated with 2D calculations. This increases the accuracy of the model. However, if the separate SWAN model seems too costly to develop, it is advised to make use of the DaF module in Hydra-NL but conduct those calculations over the full width of the foreshore. To do so, the location of the output location in Hydra-NL must be moved to the foreshore fringe.

7.2.2 Recommendations

The recommendation which can be made towards the engineering world is to try to, step by step, expand the possibility to include the foreshore potential in the assessment and design procedure. By making use of the results of experiments regarding the vegetation’s capacity to mitigate wave conditions under extreme storm conditions, some uncertainty and hesitation to include the foreshore potential can be taken away. Hereby, the final goal, which should be to include the potential of foreshores to mitigate wave conditions, is one step closer. By implementing the foreshore potential in the assessment and design procedure of primary flood defences, NbS can become part of the adaptive capacity of WGD’s which is highly valued in today's and tomorrow’s world. To reach this, it is advised to the Hydra-NL developers to move the output location of the hydraulic boundary conditions towards the foreshore fringe such that the foreshore can be included in the calculations. Doing so, approach d mentioned in the final sub-question can be followed. However, as advised, further research on the influence of moving this output location on the accuracy of the model output needs to be conducted before hard statements can be made.

Besides the approaches addressed in the final sub-question of this study, a more sustainable solution would be if Hydra-NL can account for the increase in the bottom level of areas above MHWL and the presence of vegetation by implementing increased bottom friction based on the quality of the vegetation and water depth on the foreshore. By doing so, a more realistic situation can be sketched while the DaF module or a separate 2D model is not needed to be used anymore. Still, it would be advised to include some kind of flexibility in Hydra-NL such that the increase of areas above MHWL as well as the value applied for increased bottom friction can be adjusted if needed to cope with uncertainties in foreshore dimensions and vegetation properties. Although a lot of changes will have to be made to the background model of Hydra-NL, and into the Hydra-NL environment as a whole, a great added value is seen for the implementation of the foreshore-dike system into the designs of the engineering companies resulting in more sustainable and adaptive flood defences.

Lastly, to decide whether, and if so, to what extent the foreshore is worthwhile to include in the flood defence assessment or design, a framework should be conducted that combines the (un)certainties with the effectiveness and possible adjustments of measures. Multiple NbS frameworks are already present which try to capture the link between management strategies and risk analysis (e.g. Accastello et al. (2019);

Calliari et al. (2019); Raymond et al. (2017)). By doing so, a total image of the risks and limits to implementing NbS in a flood defence system could be made. An example of such a framework can be seen in Figure 40. Based on the conducted study, the greatest focus for the framework that needs to be shaped will be on combining the uncertainty of climate scenarios with the uncertainty of the foreshore to adapt to those uncertain climate changes (as visualised in 'identification of climate proof alternatives' in Figure 40). If those uncertainties are indicated, the link toward adaptive management can be made such that NbS can be included in the assessment and design of primary flood defences (as visualised in 'Adaptive management' in Figure 40).

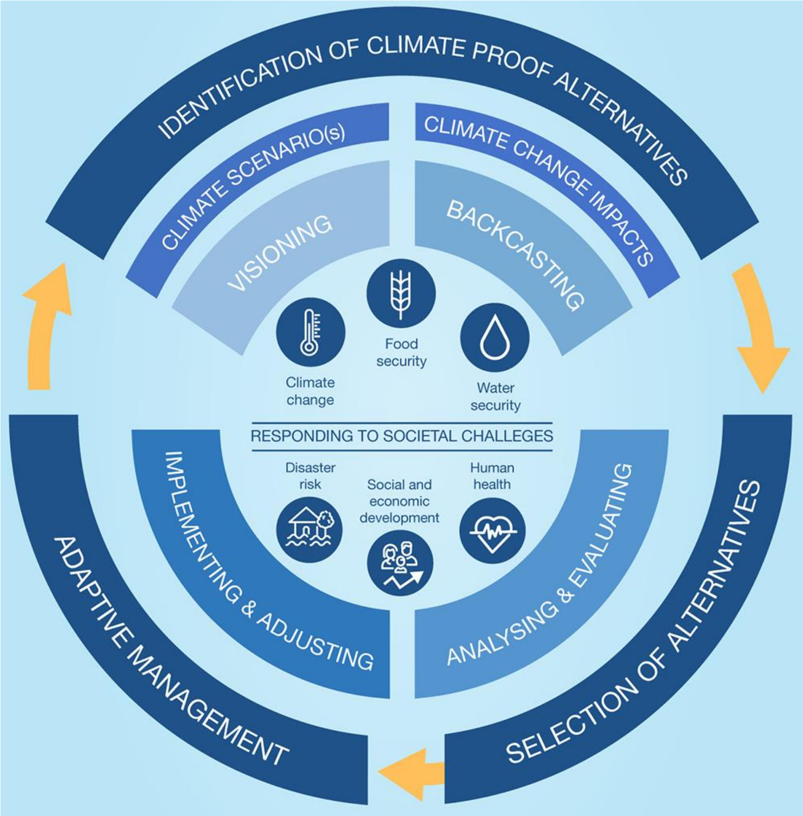


Figure 40. Example NbS framework (Calliari et al., 2019)

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Appendix

A. Model set-up Physics and Numerics

Table 10. Model set-up Physics and Numerics (SWAN User Manual, 2021)

	Model parameter	Value	Source
Physics	Generation wave model	GEN3	Default
	Non-linear quadruplet wave interactions	iquad=2, lambda=0.25, Csh1=5.5, Csh3=-1.25	Default
	Limiter	Ursell=10.0, Qb=1.0	Default
	Constant bottom friction	Jonswap = 0.038	Default
	Constant breaker index	Alpha = 1, gamma = 0.73	Default
	Triad wave-wave interaction	trfac=0.05 cutfr=2.5	Default
Numerics	Maximum absolute change H_s	0.005	Default
	Maximum relative change H_s	0.01	Default
	Maximum curvature iteration curve	0.005	Default
	Minimum fulfilment in wet grid cells	99.5%	Default
	Maximum amount of iterations	50	Default

B. Foreshore height

In Figure 41 the foreshore height changes executed for the sensitivity analysis are shown compared with the original height. With the use of a MATLAB model, the height is increased/decreased if the bottom level was equal to, or higher than, 1.4m +NAP.

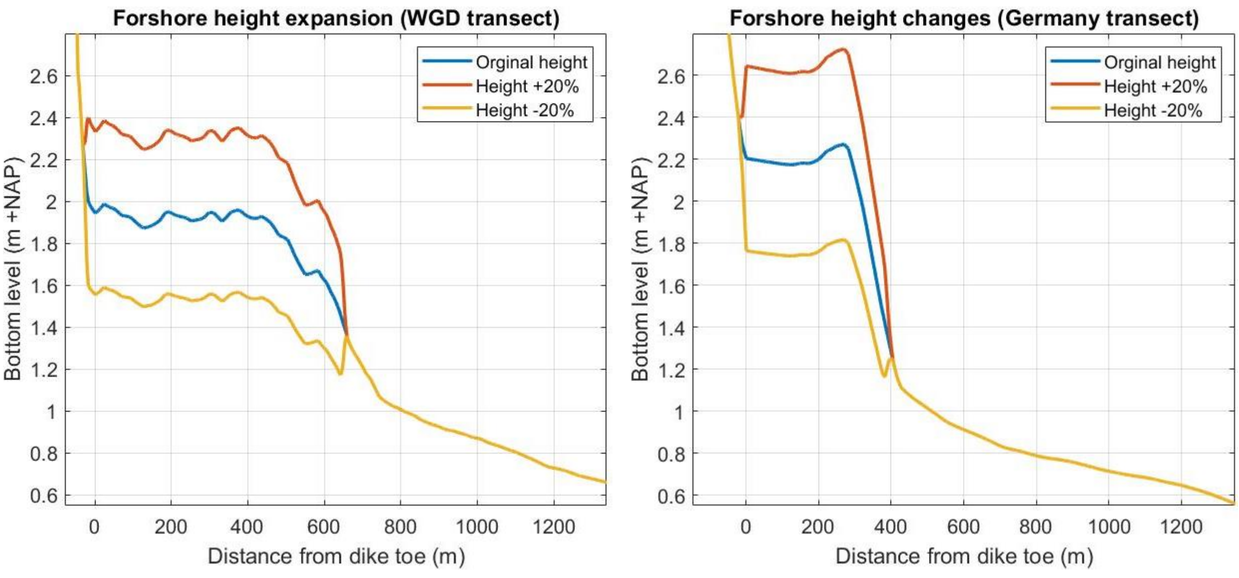

Figure 41. Foreshore height changes visualisation

C. Foreshore width

For the width changes, an Excel model was used to increase and decrease the width of the foreshore by adding/deleting areas of the foreshore. The top two photos in Figure 42 visualise the change in the foreshore front from a top view. The bottom two graphs in Figure 42 visualise the change in foreshore front from a side view of the transect shown with the yellow line in the top two photos.

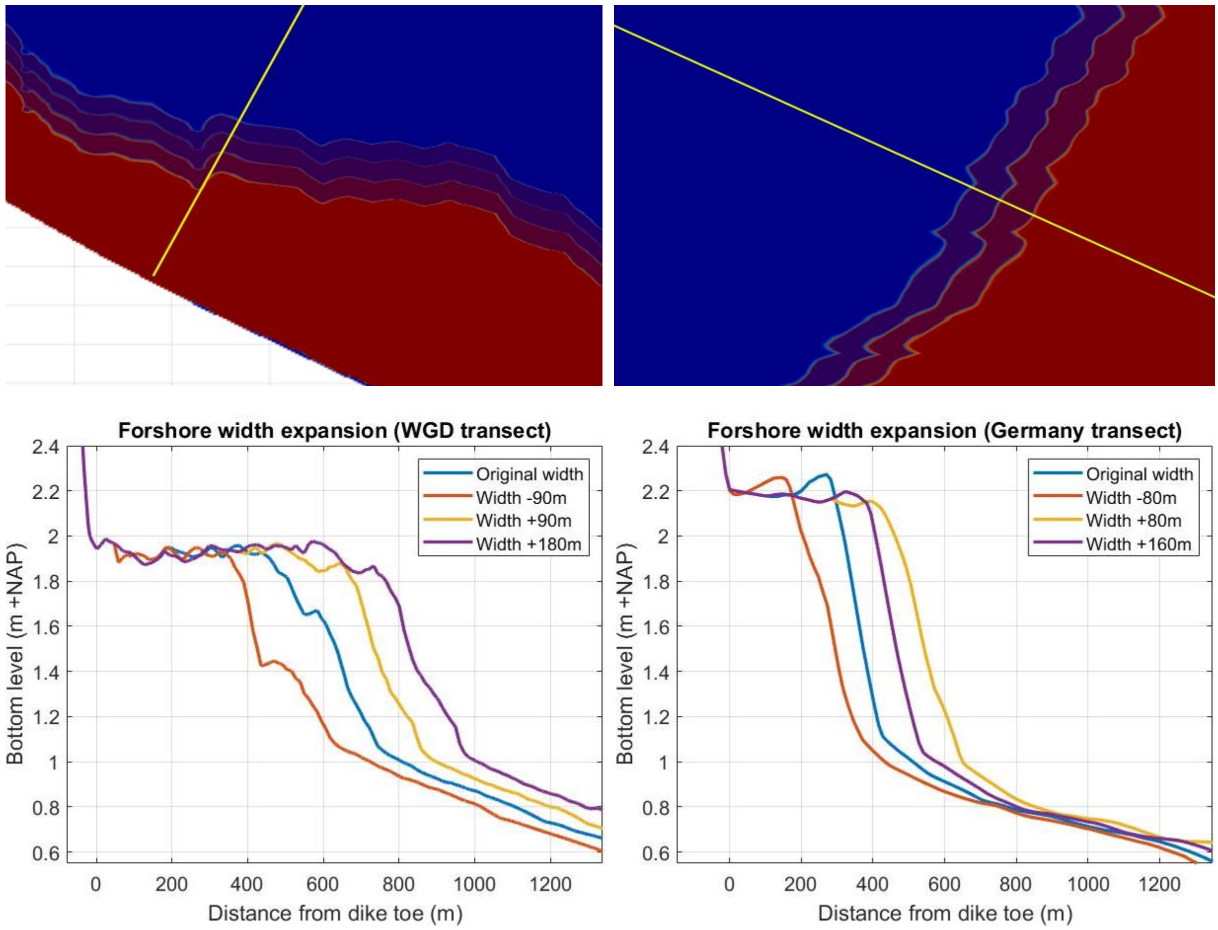

Figure 42. Foreshore width changes visualisation

D. Calculation failure probability

For the assessment and design according to the WBI2017 and the OI2014, the allowed failure probability over a dike segment is the same. With the use of the allowed failure probability of a dike segment the allowed failure probability of a cross-section per failure probability will be determined. To do so, the following formula is used (Rijkswaterstaat, 2017):

$$P_{required,dsn} = \frac{\omega * P_{max}}{N_{dsm}} \qquad (1)$$

Where $P_{required,dsn}$ is the maximum allowed failure probability per cross-section for a specific failure mechanism [1/year], ω is the allowed failure probability factor for a specific failure mechanism [-], P_{max} is the maximum allowed failure probability for the specific dike segment [1/year] and N_{dsm} is the length-effect factor of a cross-section [-]. The parameters and results of the calculation are shown in Table 6.

Table 11. results calculation maximum allowed failure probability per failure mechanism (calculated with values from Maris & Kampen (2018))

	Length effect N_{dsm} [-]	maximum allowed failure probability P_{max} [1/year]	Allowed failure probability factor ω [-]	Max. failure probability per cross-section $P_{eis,dsn}$ [1/year]
Wave-overtopping	3	1/3.000	0.24	1/37.500
Erosion grass outer slope (GEBU)	3	1/3.000	0.05	1/200.000