Robust Flood Defence in response to Climate Change

Westkapelle Case

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Technische Universiteit Delft

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Challenge the future

Robust Flood Defence in response to Climate Change

Westkapelle Case

by

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in partial fulfillment of the requirements for the degree of

Master of Science
in Civil Engineering

at the Delft University of Technology,
to be defended publicly on Friday June 15, 2018 at 10:00 AM.

Project duration:	February 13, 2018 – June 8, 2018	
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Executive Summary

In the Netherlands flood protection is immensely important for the safety of the nation. The shocking outcome of the 1953 flooding proves this point. In modern days, the development of socio-economic and climate change factors casts doubt on the effectiveness of conventional approaches to flood risk management (Klijn, Knoop, Ligtvoet, & Mens, 2012). Consequently, this project explored new approaches to flood risk management via a case study.

The team started off by familiarizing with the region and its local issues. This was done through a literature review of available sources and material provided by the client. A site visit followed and confirmed the team's perceptions. An analysis of climate change effects led to estimation of future loading conditions. Subsequently, a detailed hydrodynamic analysis was conducted. It highlighted the significant levels of uncertainty that climate change introduces into loading conditions. Also, it confirmed the team's perception, that the case region requires additional safety measures to guarantee an acceptable level of safety in the future.

But how to guarantee the acceptable level of safety in the most efficient way? The team adopted the concept of robustness to find an answer. In a keynote publication Mens (2015) describes robustness in the following way: "Robust flood risk systems have some degree of resistance and some degree of resilience: the system can withstand some floods (no response), and for other (larger) floods impacts are limited and the system can recover quickly from the flood impact (response and recovery)." The team set out to include robustness as an integral part of the design process to handle uncertainties. The project shall be seen as an explorative study how this can be done, revolving around Westkapelle as a case study that proves the methodology's feasibility.

Robustness and uncertainty were included on multiple levels throughout the design process. Firstly, the range of uncertainties was quantified. Secondly, the effect, that single parameters have on the magnitude of uncertainties, was assessed. Thirdly, the system's capacity was analysed to find the required overtopping reduction for guaranteeing sufficient safety. Fourthly, constructive measures were assessed on their robustness potential and satisfaction of stakeholder needs via a Multi Criteria Analysis (MCA). The MCA was then employed to select the type of constructive and non-constructive measures to achieve the required levels of overtopping and safety. With the information on uncertainties, the measures were combined to form a robust design, consisting of living breakwater, dike heightening, surface protection and two policy measures. A thorough comparison between the conventional design, that has been applied to the project location, and the robust design followed. The robust design came out slightly on top.

Robustness was found to be an effective tool in countering uncertainties. Where conventional design methodologies are lacking flexibility and precision, the robust design methodology makes use of the system and its resilience to find an optimal solution. Its applicability may not be limited to flood risk management only but stretch out to other civil engineering disciplines.

Chapter 1
Introduction

1 Introduction

The Netherlands has been strategically managing its coastline for centuries to defend the country from flooding. Roughly one third of the country is below sea level, and the expansive dike system is a key defense mechanism to prevent the ocean water from entering the flood prone areas. The cities of Westkapelle and Middelburg are located within dike ring 29 in the South-West of the Netherlands. Westkapelle and Middelburg are home to roughly 3000 and 48000 inhabitants respectively. Figure 1 illustrates the many dike rings in the Netherlands and the designed failure probabilities. Dike ring 29 is designed with a failure probability of less than 1/4000 a year.

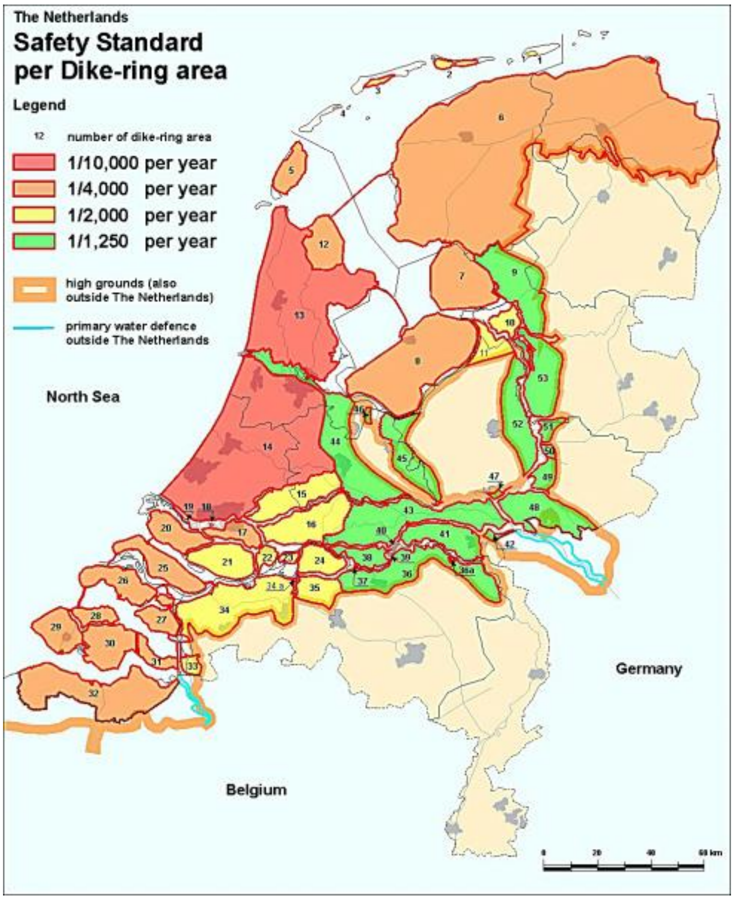


Figure 1: Safety Standard per dike-ring area

The coastal dike near the town of Westkapelle is most sensitive to the failure mechanism of wave overtopping, contributing 43% to the overall failure probability. If this dike were to fail, the entire polder would be flooded. A major concern for the Netherlands now is how climate change will affect the coastal flood protection systems. How climate change will affect the intensity and frequency of extreme events is unknown, and therefore it is difficult to prepare the coastal defence system for the future.

1.1 Background

Different paths of action could be taken to overcome such a hazard and to prepare for the years to come. Conventional strategies mostly focus on dike heightening, but recent research casts doubt on the effectiveness of such methods (Brinke, Bannink, & Ligtvoet, 2008). Socio economic factors are only a minor consideration in conventional strategies, but research shows that socio economic factors have a high significance for the impacts of flooding (Brinke et al., 2008). The socio-economic factors' future development, just like climate change's development, is not accounted for sufficiently – as Mens (2015) states: "… risk may not suffice as decision-criterion …, … it is uncertain how it will change over time following socio-economic developments and climate change.". This fact gives robustness-based solutions a great potential, as socio economic factors are considered much more profoundly than in conventional solutions. Therefore, while managing the dangers of climate change an additional minimization of flood risk (the product of consequences and probability of flooding) can be achieved.

1.2 Problem Definition

This is a system-based study consisting of two phases. The first phase is a 'cause-effect' study, and the second phase is a 'problem-solution' study. The thought process to tackle the problem is shown in Figure 2.

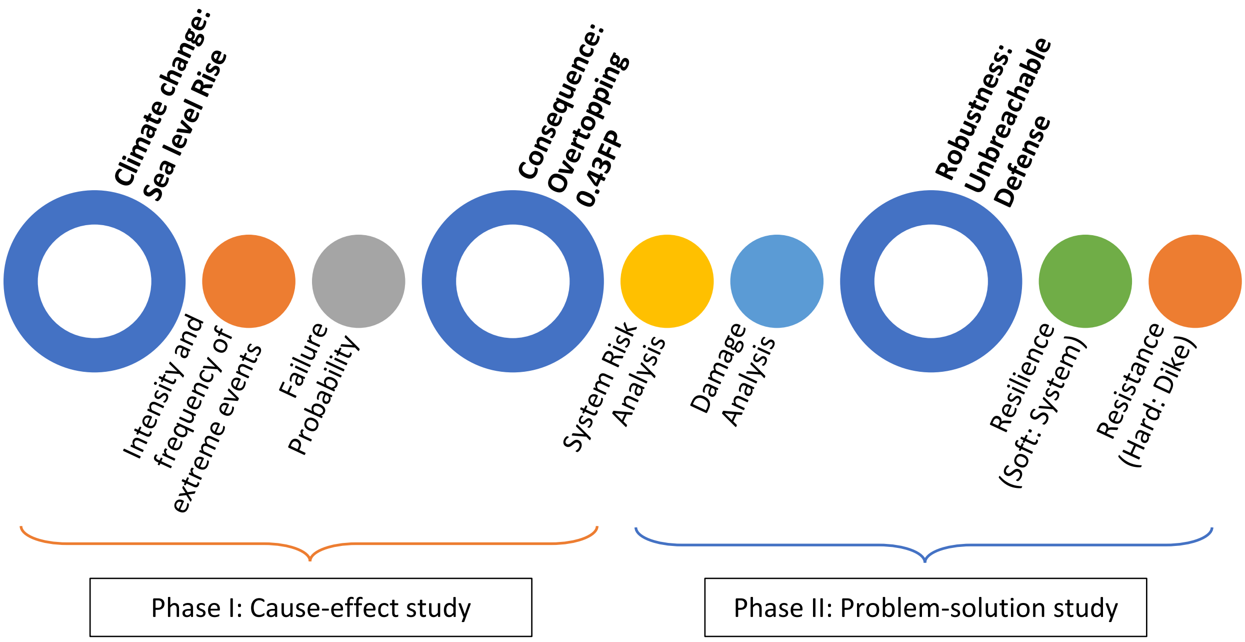


Figure 2: Project scope

The 'cause' for this project is the global climate change due to increasing global warming. Of the many adverse effects caused by global warming, the most relevant to this project is sea level rise. The change of mean sea level has a direct impact on the wave heights and wave depths that are expected at the defense structures built to counter extreme events such as storms, increased wave loads, flooding etc.

However, there is limited understanding of the direct impacts that climate change has on the intensity and frequency of extreme events, and, in turn, how climate change will have an impact on the coastal defence structures. This leads to significant levels of uncertainty and generates the need for thorough sensitivity analysis of design proposals. The team decided to tackle the uncertainty via designing in a robust way.

The concept of robustness has only recently been introduced into the realm of flood risk management (Bruijn 2004). Case reports of creating and implementing a robust design are still scarce. Therefore, the objective of this project was to research ways of designing in a robust way and showing the viability of such a design via a case study. Further elaborations on this topic follow under chapter 2.4.

1.3 Project scope

The scope of the project is to analyze the effect of climate change on the coastal dike system of Westkapelle and Middelburg, and to determine an appropriate design solution to cope with the unknown, yet certain, increased loading effects. This will be done by quantifying the effectiveness of robustness-based approaches and to compare it to the effectiveness of conventional solutions. Via different decision-making tools a combination of options to increase resistance and/or resilience will be found and designed for the case of Westkapelle and Middelburg. The design is then compared to the conventional measures that would be taken to guarantee an acceptable level of flood risk up until year 2100.

1.4 Site Description

Westkapelle is located along the coast of the Netherlands in the province of Zeeland, west of Vlissingen, Middelburg and Domburg. Although this case study includes Middelburg and Westkapelle, the main focus is on the coastal dike system directly protecting Westkapelle. This is because the coastal dike at Westkapelle is most exposed to wave forces, with direct interaction from the predominant SW waves and NW swell. Middelburg is not directly located on the coast and therefore will only be included in the project as a major city that could be flooded due to the failure of the dike system. Figure 3 displays a map of the region including both Westkapelle and Middelburg. A detailed view of the coastal area near the city of Westkapelle is shown in Figure 4. The yellow circle highlights the position of the sea dike (upper half of the figure) whereas the coastal zone in the southern half (red circle) of the figure is not protected by a dike, but rather by a sandy beach and a dune area.

Figure 3: Map of Westkapelle and Middelburg, Google maps

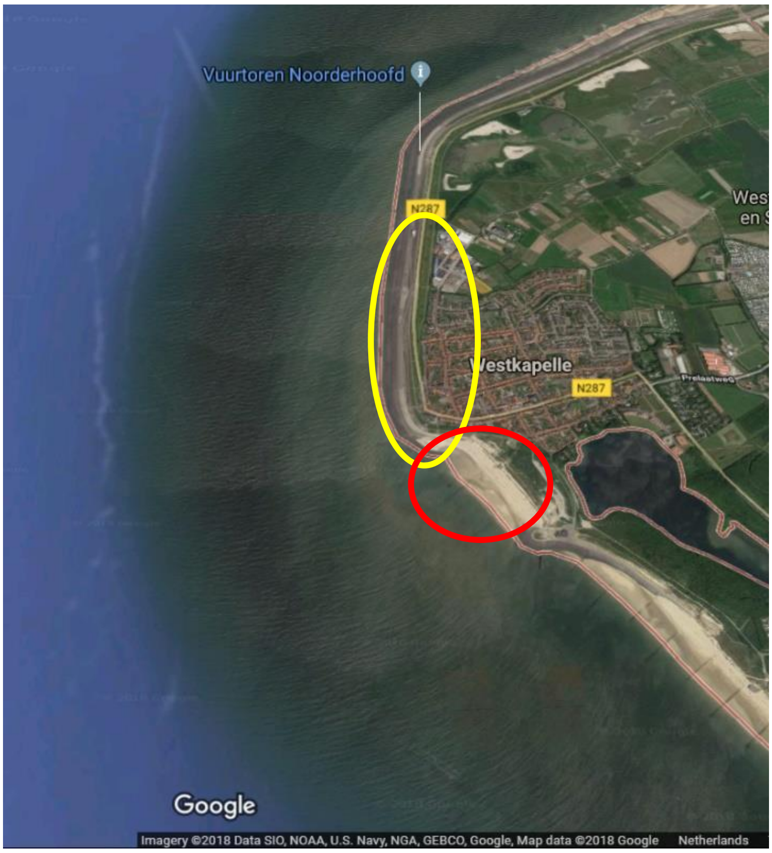


Figure 4: Coastal dike and beach dune section, Google maps

Chapter 2
Project Outline and Methodology

2 Project Outline and Methodology

Phase I of this project is now complete. It focused primarily on obtaining the background information necessary for the analysis of the flood defences for the Westkapelle region, as well as the identification of alternative flood defence and mitigation designs. Having selected what is considered the most advantageous design, a more in-depth analysis and detailed design will be addressed in Phase II of the project. The methodology is roughly illustrated in Figure 5 below.

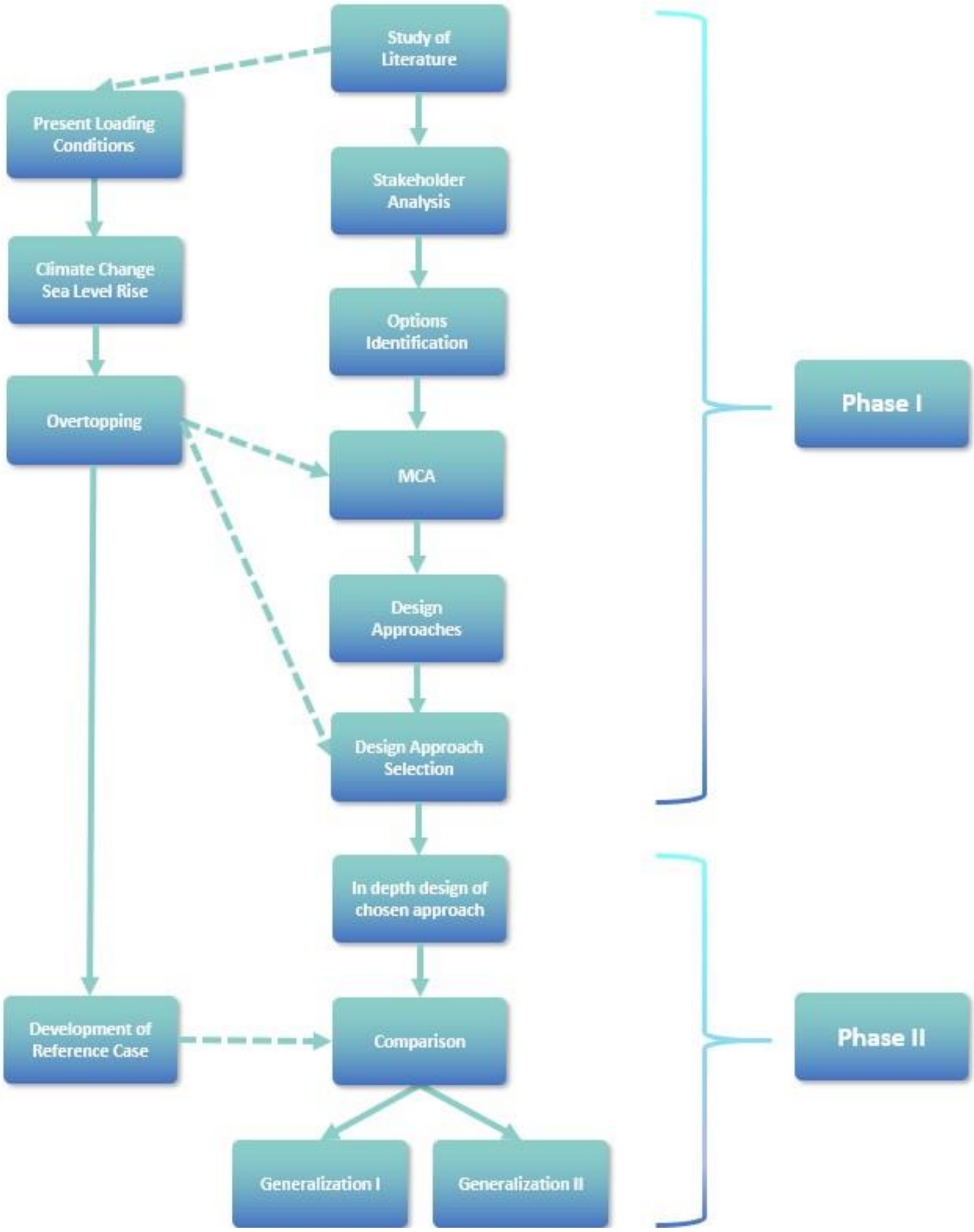


Figure 5: Project Methodology

As can be seen from Figure 5, the analysis in phase I was divided into multiple steps. These steps briefly are:

Study of Literature

The literature study focused on the project region and processes that are relevant for its flood protection. Regarding the study of the project region, the current state of flood defences, the climate and hydraulic conditions in the region, the location and the topography have been considered. Considering the relevant processes, the focus was placed on dike safety, flood protection policies, resilience and robustness, sea level rise due to climate change, as well as alternative options of flood protection and mitigation of its effects.

Loading conditions – Climate change - Overtopping

An overtopping analysis was deemed necessary for three reasons. First, to get an insight in the present situation by means of overtopping of the dikes and flooding of the area under investigation. Second, to compare the present situation with a future situation that accounts for sea level rise. Finally, for the detailed design in phase II. For that purpose, the most important physical processes were considered. Boundary conditions in the area of study such as wave heights, sea level, meteorological conditions and bathymetry were defined. Two scenarios were considered, an extreme storm in the present situation and an extreme case with future sea level rise due to climate change.

Stakeholder Analysis

The first step before researching solutions is to identify the stakeholders. Thus, a stakeholder analysis was carried out that was later used to formulate design solutions that satisfy the most important stakeholders related to the project.

Identifying Options

The next step was to identify a variety of different design options. Each option is a measure of increasing system robustness. An important consideration was to find options that approach the design requirements from different points of view, i.e. not only finding constructive measures.

Multi-criteria Analysis (MCA)

As this project incorporates the involvement of multiple stakeholders, it requires the investigation of the problem from different approaches. These are however coupled and therefore it is essential to identify the importance of each of the considered options. That was done through a multi-criteria analysis where the effect of each option was assessed for various criteria and ranked accordingly.

Formulation of Design Approaches

Using the results from the MCA, weighted according to a choice of criteria, design approaches were formulated.

Site visit

By visiting the town of Westkapelle, exploring the shoreline, it’s dunes and dikes and by having the chance to discuss with local residents, the team became more familiar with the area and gained a better insight into the stakeholders’ opinions.

<u>Choosing Design Approach for phase II</u>

Having the design approaches from the previous step, the final step of phase I was deciding which design approach would be the optimal solution for the region. A more detailed design of the approach and an economic optimization of the design will follow in the next phase.

2.1 Schedule (Work plan)

The project is scheduled to last for a period of five months, beginning in February 2018 and ending in June 2018. The total number of hours of work is approximated to be 1560 hours, equalling roughly 310 hours per consultant which is adequate for the scope of this project. A global work plan has been created outlining the major activities and milestones (deliverables) necessary for successful completion of this project. The work plan can be seen in Figure 6. A table with the Gantt-Chart data can be found in Appendix A - Scheduling.

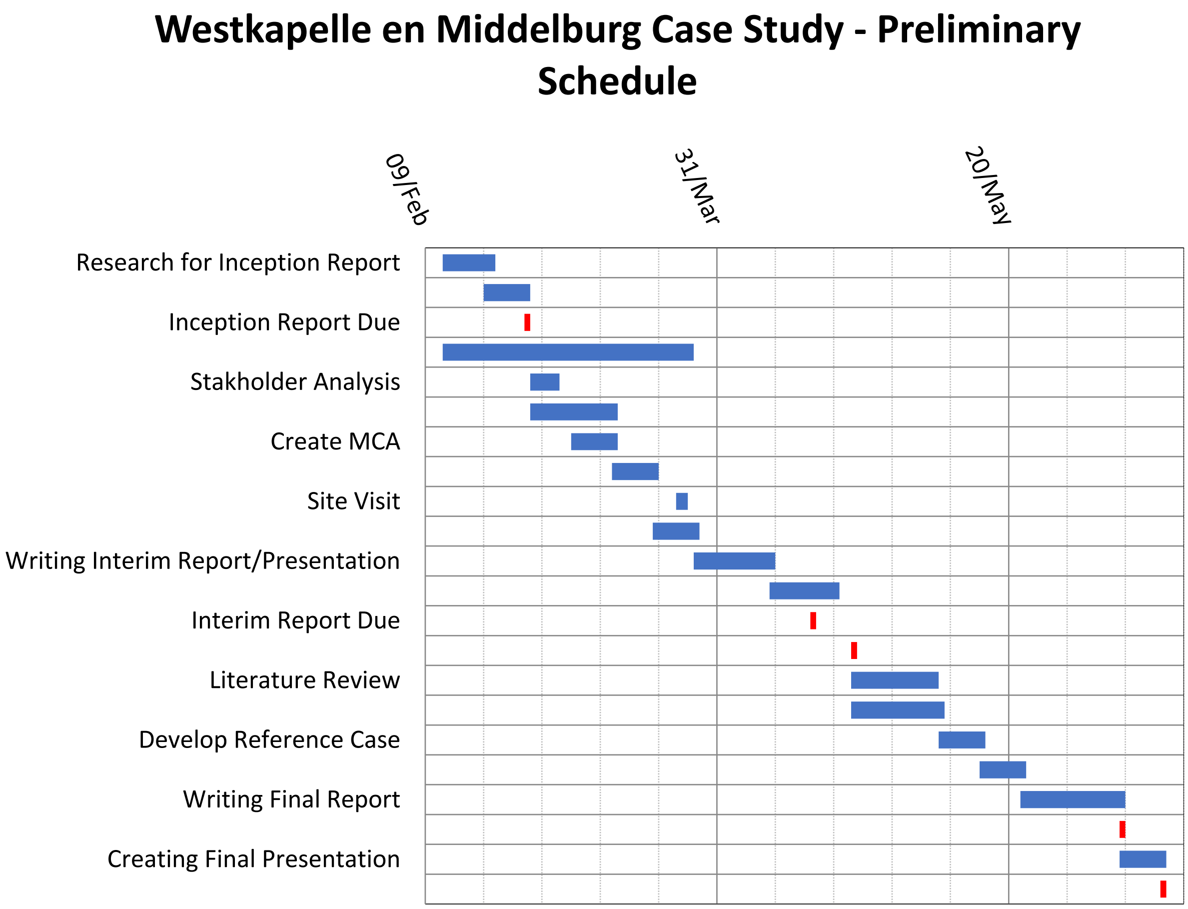


Figure 6: Work Plan Gantt-Chart

As seen in the figure, the due dates for reports and presentations are marked in red, and the blue bars outline the duration allotted to each specific task. It can be observed that up to now the project is on schedule. This is a preliminary schedule which for phase II may be altered if deemed necessary.

2.2 Individual Roles and Responsibilities

Separating tasks for the project between team members is done on a weekly basis. This allows the team to ensure that all members are working effectively on the most current and important tasks. That being said, there are concepts that certain members will have more of a responsibility over others. The individual responsibilities can be seen in Table 1.

Table 1: Team member responsibility

Team Member	Responsibility
Florian	Project Manager
Fred	Location and Dike Specialist
Hassan	Failure Probability Specialist
Khaled	Climate Change Specialist
Nikolaos	Wave Loading Specialist

2.3 Project Cost

The cost-estimate for the work provided by 1950-Free Consultants can be seen in Table 2. The hours are total man hours, meaning that a meeting with all 5 team members for 1 hour is equal to 5 hours in the table. The work with an external consultant for specialist advice will be charged at a rate of €200/hr. The external consultant could be a researcher or professor that the team contacts to obtain more information. The estimated total cost is roughly €188,000.

Table 2: Cost estimate of project

Consultant Rate (€/person/hr)	**€ 120.00**	
External Consultant Rate (€/person/hr)	€ 200.00	
Task	**Hours**	**Cost**
Literature Review	300	€ 36,000
Stakeholder Analysis	40	€ 4,800
Loading Conditions & Calculations	100	€ 12,000
Multi Criteria Analysis	100	€ 12,000
Generation of Design Approaches	100	€ 12,000
Generation of Reference Case	45	€ 5,400
Design and Verification	200	€ 24,000
Comparison of Design Approach and Reference Case	100	€ 12,000
Generalisation Strategy	100	€ 12,000
Report Writing	230	€ 27,600
Presentations	50	€ 6,000
Meetings	150	€ 18,000
External Consultant	8	€ 1,600
Administrative and Liaison	20	€ 2,400
Site Visit	15	€ 1,800
	Total Hours	**1558**
	Total Cost	**€ 187,600**

2.4 A Robust Design Methodology

As mentioned in section 1.2, the team had to develop a robust design methodology in order to account for uncertainties. To do so, a number of steps were identified as can be seen in Figure 7.

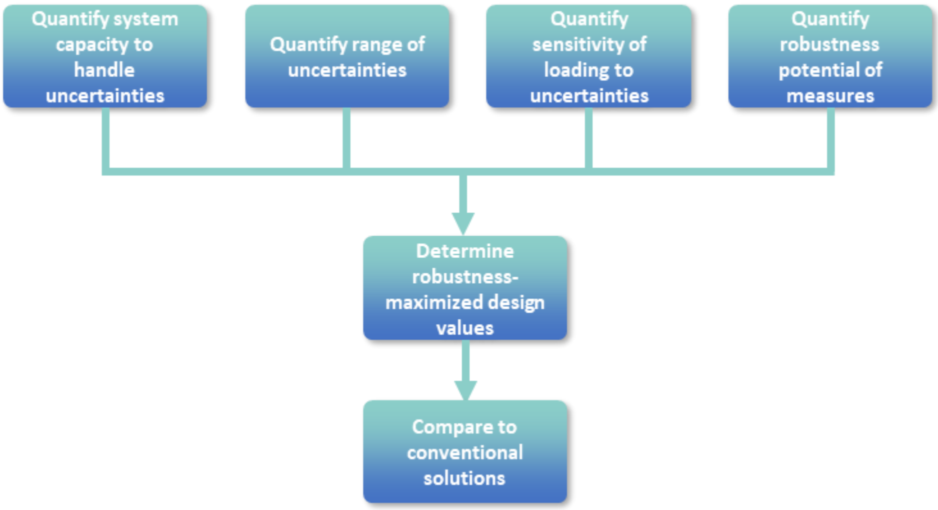


Figure 7: Robust Design Methodology

Firstly, the team started with a quantification of the range of uncertainties in loading conditions. The design horizon is 2100 so that climate change is the main driver of future uncertainties and its effects were quantified in chapter 0. The assessment of hydrodynamic conditions on site, refer to chapter 0, highlights the range of possible loading conditions. This range is further increased by the interactions of normal loading conditions with climate change. Chapter 0 aims to quantify this range of uncertainties and to give a good idea of the severest and lightest loading conditions that can be expected.

Secondly, the sensitivity of loading to uncertainties was analysed. The team wanted to know how influential certain parameters, like water level or wave height, are on the loading conditions. In this way, the design could take special notice of those parameters and minimise uncertainties by addressing them directly. The quantification was partly done in chapter 0 and more thoroughly in chapter 0.

Thirdly, the team was interested in finding out about the system's capacity to handle uncertainties. The concept of robustness entails that the system resists certain loadings and recovers quickly from more severe loadings without taking severe damages. In order to gauge how much loading is required for severe damages to occur, a thorough analysis of the effects of different overtopping values on the system was undertaken in chapter 0. To gain an understanding of the importance of lighter damages, a stakeholder analysis was undertaken in chapter 0.

Fourthly, the robustness potential of single constructive measures (and policy measures) was analysed. A variety of constructive measures is available for decreasing flood risk. Some measures do this via resistance whereas others focus on resilience. In order to be aware of the differences, the robustness potential was assessed with a Multi Criteria Analysis in chapter 0. Single measures were then combined in different ways to maximise their potential, refer to chapter 0. In this way, a preselection of measures was undertaken which guarantees that the design will be made up of robust measures.

Subsequently, a more detailed design step was undertaken with the preselected measures, refer to chapter 0. The multicriteria optimization in chapter 14 confirms the design choices. The above-mentioned points were of central importance to choosing sensible design values. By being aware of the range of uncertainties in loading, the design can handle different climate change scenarios. By being aware of the sensitivity of loading, the design effectively counteracts those loading parameters that have the biggest influence. By being aware of the system's capacity to handle uncertainties, failure can be defined and designed for in a more efficient way, accounting for resilient reserves of the system. By being aware of single measures' robustness potential, a well-educated choice of measures can be made that accounts for the potential of constructive measures more thoroughly.

Finally, the design is compared to a reference case, refer to chapters 0 and 0. The comparison marks the differences to conventional engineering solutions and highlights the benefits and shortcoming of the robust design. The conclusion in chapter 0 gives recommendations on the strategy that Westkapelle should adopt. The project makes use of the Westkapelle case study to show how robustness can be adopted as a key driver of the design process.

Chapter 3
Climate Change

3 Climate Change

There is a strong correlation between global temperature rise (caused mainly by greenhouse gas emissions) and the predicted sea level rise (Attema et al., 2014; Deltacommissie, 2008). Regarding climate change, CO2 is the most detrimental gas. It is a result of fossil fuel combustion, cement production and changes in land use (Deltacommissie, 2008). The CO2 concentration in the atmosphere has increased since the pre-industrial period by almost 40 percent (van den Hurk, Siegmund, & Tank, 2014).

The Intergovernmental Panel on climate change (IPCC) expects an increase in the global temperature by 6°C if the atmospheric CO2 concentration increases to about 750 ppm (currently 408ppm) (CO2.earth). For modelling of climate change the IPCC has formed four different families of scenarios named A1, A2, B1, and B2, with four combinations of demographic change, social and economic development, and broad technological developments each (Deltacommissie, 2008; Nakicenovic et al., 2000). Figure 8 shows the IPCC's emission scenarios. The A1 scenario family, is of great interest as most of the climate change simulations, particularly the mean sea level rise off the Dutch coast, are based on its three groups (A1B, A1T, A1FI).

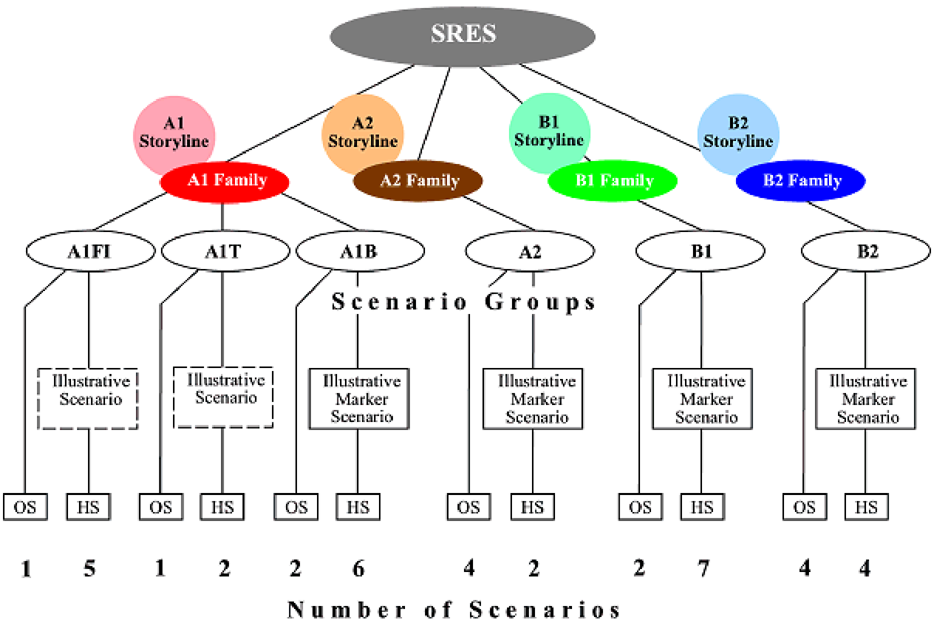
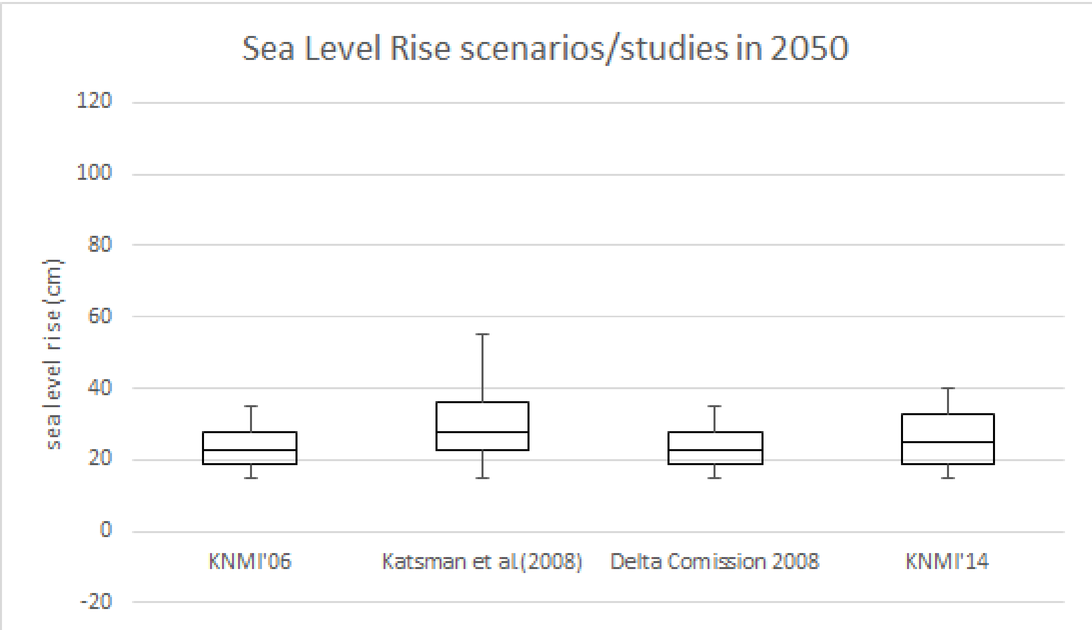


Figure 8: Families of emission scenarios (Nakicenovic et al., 2000)

The A1 scenario family describes a world of rapid economic growth and a global population with alternative directions of technological change in the energy system. The A1 scenario family develops into three groups, namely, A1FI (fossil intensive), A1T (non-fossil sources) and A1B (under the assumption that similar improvement rates would apply for all energy supply and consumption technologies) (Deltacommissie, 2008; Meehl et al., 2007).

3.1 Sea level rise off the Dutch coast

There is a wide variety of different models and simulations that make use of the climate change scenarios to forecast Sea Level Rise (SLR). Figure 9 and Figure 10 display the ranges of predicted SLR from different sources for the year 2050 and 2100 respectively.



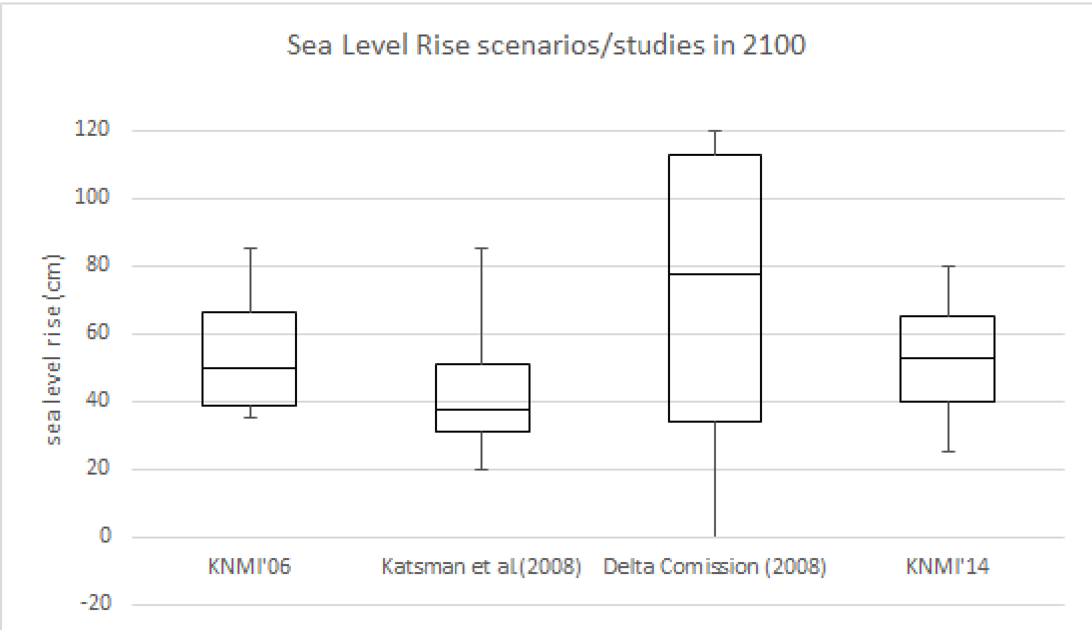


Figure 10: Summary of most important studies on sea level rise off Dutch coast in 2100

Due to its high level of detail regarding the region of interest, the A1FI scenario is the one that is used for the sea level rise predictions. It is the worst-case emission scenario for this project when considering the range for sea level rise scenarios until 2100. Therefore, the modelling and analysis mentioned in the Delta Committee (2008) report is adopted. Those adopted estimates slightly vary from other findings in the literature since the A1B emission scenario was adopted in contrast with the worst-case scenario A1FI.

The Royal Netherlands Meteorological Institute (KNMI) has identified four scenarios (Figure 11) for the Netherlands, varying in terms of mean temperature increase and air circulation patterns. Each of the four KNMI'06 scenarios may occur under each IPCC emission scenario for 2050 (Deltacommissie, 2008; Vellinga et al., 2009). For 2100, KNMI assumes a temperature increase in 2100 of 2°C in its 'low-temperature scenario' and 4°C for the 'warm scenario'. The latter is more likely under a high A1FI scenario (Deltacommissie, 2008; Vellinga et al., 2009).

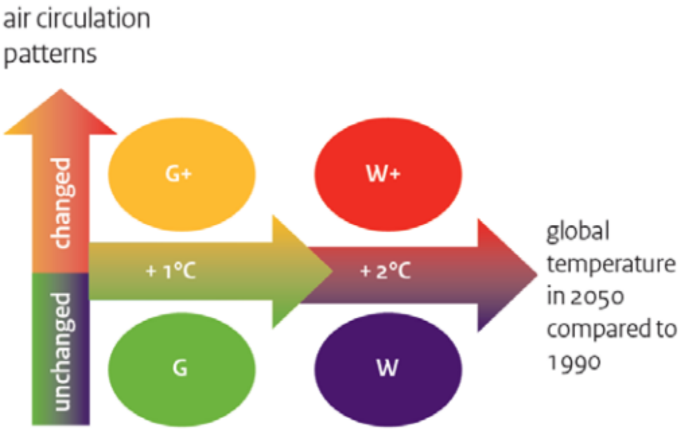


Figure 11: Classification of the four KNMI'06 scenarios (Deltacommissie, 2008; Vellinga et al., 2009)

The KNMI'06 scenarios result in a possible sea level rise along the Dutch coast of 15 to 35 cm by 2050 and 35 to 85 cm by 2100, as shown in Table 3. The estimates exclude land subsidence. This indicates that SLR is on the rise and all the estimates of SLR along the Dutch coast incorporated this value in their calculations.

Table 3: KNMI'06 scenarios for sea level rise in 2050 and 2100 (Van Minnen et al., 2013)

		G/G+		W/W+	
		2050	2050	2100	2100
Global temperature rise		+1°C	+2°C	+2°C	+4°C
Sea level Rise (cm)	Rise off Dutch coast without land subsidence (= absolute rise)	15–25	35–60	20–35	40–85
	Rise off Dutch coast, corrected for gravity effect (based on Katsman et al. 2008)	15–25	30–55	20–35	40–85

Interestingly, the sea level along the Dutch coast has increased by approximately 20 cm with respect to Dutch Ordnance Datum NAP over the past century as shown in Figure 12 (Van Minnen et al., 2013).

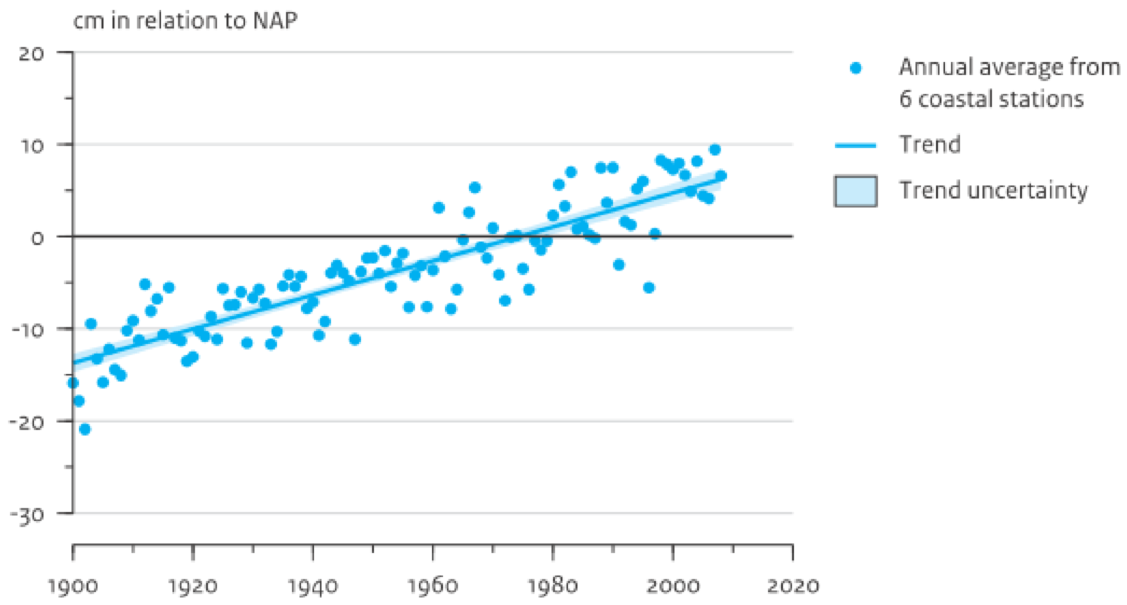


Figure 12: Sea level at the Dutch coast (Van Minnen et al., 2013).

Besides KNMI06, The Delta Committee (2008) has adopted some climate scenarios in relation to the IPCC's (2007) global climate scenarios and the KNMI (2006) regional climate scenarios. These scenarios were based on an investigation request of global sea level rise and the rise along the Dutch coast for the years 2100 and 2200. Therefore, The Delta Committee has used the IPCC A1FI scenario 's (2007) findings as the basis for its estimates of several major components of sea level rise in 2100 / 2200. The IPCC A1FI scenario's (2007) estimate of 6°C temperature increase may occur in 2100 if the atmospheric CO2 concentration at that time increases to about 750 ppm. Given a temperature increase of 6°C, the Dutch coast would experience a sea level rise of 0.65 to 1.3m in 2100, a 0.11 ± 0.07 m contribution due to vertical land movement is included in this projection which is due to glacial isostasy and subsoil compaction (Van Minnen et al., 2013). Similarly, Katsman et al. (2011) projected the SLR value in the North Sea basin for the extreme climate change scenario for 2100 to be 1.30 m (Katsman et al., 2011).

Clearly, the future sea level rise is highly uncertain. Three main causes for this uncertainty can be identified. Firstly, the lack of full understanding of the climate system. Secondly, the melting of the Greenland and West Antarctica ice sheets is extremely difficult to predict. Thirdly, the uncertainties about future emissions of greenhouse gases (Van Minnen et al., 2013). Figure 13 illustrates the uncertainties of SLR for different scenarios.

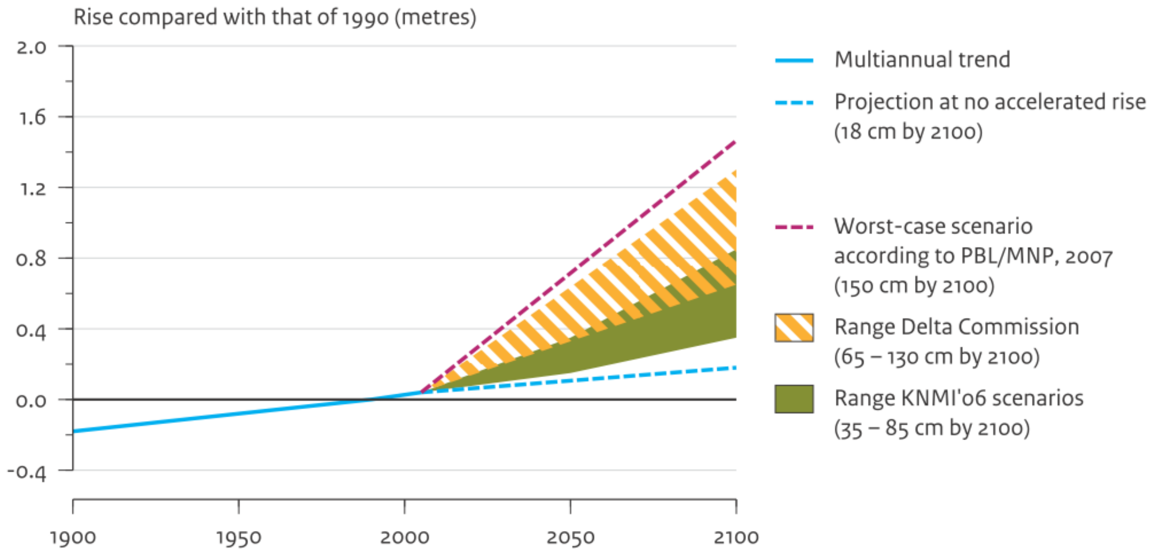


Figure 13: Sea level rise for various project's scenarios (Van Minnen, Ligtvoet et al. 2013).

To conclude, Table 4 summarizes the high-end projections for sea level rise along the Dutch coast.

Table 4: High-end projections for sea level rise

		Moderate scenario G/G+		**Warm scenario** W/W+		
		2050		2100		
Global Temperature rise		+1°C	+2°C	+1.5°C	+2°C	+6°C
Sea level	KNMI'06 scenario (Without land subsidence)	15-25 cm	35-60 cm	20-35 cm	40-85	
	Katsman et al. 2008 (corrected for local steric and elasto-gravity effects)	15–25 cm	30–55 cm	20–35 cm	40–85 cm	-
	KNMI'14 scenario (Without land subsidence)	15-30 cm	20-40 cm	25–60 cm	45 - 80 cm	130 cm
	Delta Committee (2008)	High-end projection A (using Mitrovica et al, 2001)		High-end projection B (using Plag & Juettner, 2001)		
	Excluded subsidence	0.40 - 1.05 m		-0.05 - 1.15 m		
	Included subsidence	0.50 - 1.15 m		+0.05 - 1.25 m		
Sources: KNMI (2006), : KNMI (2014), Katsman et al. (2008), Deltacommissie (2008)						

As mentioned earlier, “the lack of knowledge of some of the relevant responses of components of the climate system to greenhouse gas emission leads to sea level rise uncertainties and thus a wide range of sea level rise projections” (Vellinga et al., 2009). In this project, a range of sea level rise scenarios are used due to the uncertainty associated with climate change. The Delta Commission (2008) range of sea level rise is adopted for this project, with an upper bound of 1.3m and 0.65m is used for the lower bound.

3.2 Wave Climate Projection

Different emission scenarios and model combinations forecast variations in the future wave climate. The variations can be attributed to many different causes but variations in wind fields play a central role. In KNMI09, Vellinga et al (2009) state that they "... have investigated possible impacts of climate change on wind and wind-related quantities in the North Sea and especially along the Dutch coast. Global and regional climate modelling employing different climate models as well as different forcing scenarios suggest a slight increase in extreme wind speeds in the southern North Sea, which is reflected in a slight increase in the height of wind waves" (Vellinga et al., 2009). Therefore, an increase in wave height can be expected for the project location.

Different modelling approaches yield significant differences in the predictions for the wave climate. Along the Dutch coast, Debernard and Røed (2008) found a slight increase in the 99 percentile of significant wave height but with large differences between forcing models and forcing scenarios (Vellinga, Katsman et al. 2009). Similarly, Grabemann and Weissethere (2008) analyzed four-emission scenario/GCM combinations (two global circulation models GCMs: HadAM3H and ECHAM4/OPYC3 for two emission scenarios A2 and B2) and concluded that in the North Sea the wind speed and significant wave height with 99 percentile confidence intervals could increase by up to 7% and 18% of present values towards the end of the twenty first century (Grabemann & Weisse, 2008).

Considering the significant wave height, most of the climate change experiments and simulations show small and statistically non-relevant variations (Vellinga, Katsman et al. 2009). The reason behind that could be either the ensemble size is too small or simulations considerably underestimate the present-day annual mean and annual 99 percentile significant wave heights (Vellinga, Katsman et al. 2009). Although there is high uncertainty in the future of the North Sea's wave climate, a slight increase in the wind speed and wave climate needs to be considered (Grabemann & Weisse, 2008). Therefore, the most conservative value that was given in the literature, an increase in significant wave height of 18%, will be adopted.

Variations in the wind field may also affect storm surge conditions. Sterl et al. (2008), who adopt the A1B emission scenario, predict an increase in south-easterly winds in the northern part of the region of interest and an increase in south-westerly winds in the southern part. For the Dutch coast, including the region of interest, the most relevant surges result from north-westerly winds, due to the long fetch. Therefore, the predicted change in wind climate is very unlikely to have a significant effect on the expected surge height (Vellinga et al., 2009).

Flood protection standards, according to the Dutch law, require coastal defence systems to withstand a water level which on average would occur only once every 10,000 years (Vellinga et al., 2009). Figure 14 shows a Gumbel plot of modelled annual maximum surge heights with 95% confidence intervals for the 10,000-year return value at Hoek van Holland station for the present and the future climate from previous studies. The black (thick) denotes observations for the existing 118 years of data. The GEV-fit (black thin) yields the best estimate of 3.6 m for the 10,000-year return value, but the 95% confidence interval ranges from 2.9 m to 6.4 m (Vellinga et al., 2009). For future projections, unquailingly, extrapolation is necessary. Hence, the Essence-WAQUA/DCSM98 ensemble for present-day climate (1950-2000, blue) and future climate (2050-2100, red) is shown, as well, in the Figure 14 and their respective GEV fit yields best estimates ranging from 2.8 m to 3.6 m for the present-day climate and the 95% a confidence interval for the future climate estimates ranging from 2.9 m to 3.7 m. As a result, the plot confirms that no significant change of surge heights due to climate change along the Dutch coast is to be expected (Vellinga, Katsman et al. 2009).

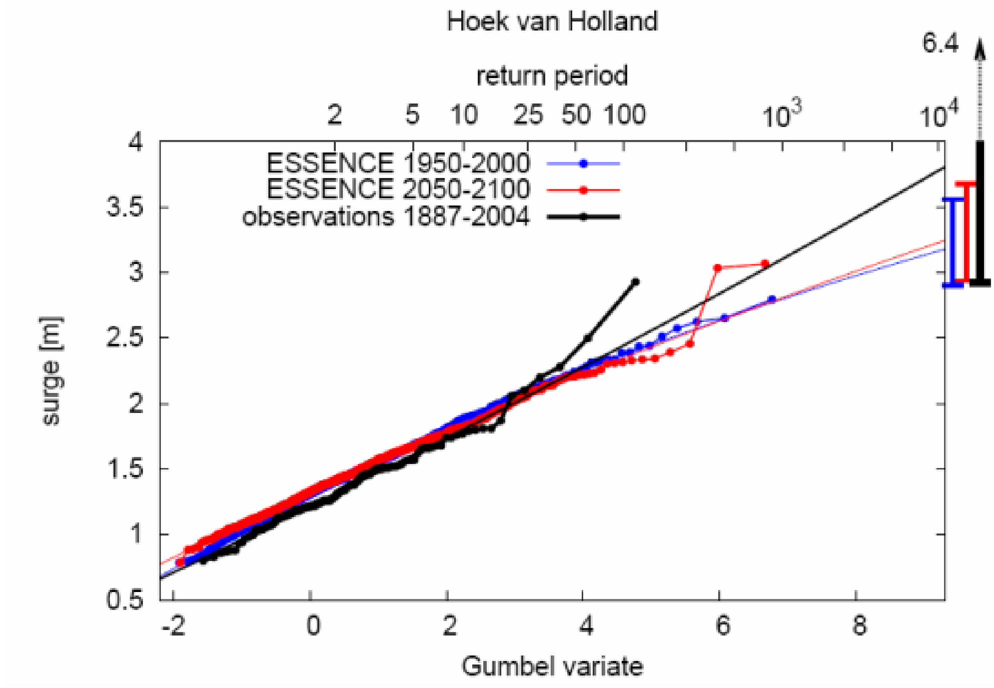


Figure 14: Gumbel plot for surge heights at Hoek van Holland (Vellinga et al., 2009)

3.3 Subsidence

According to the Delta commission report (2008), “local land movement on average, the Netherlands experiences about 0.03 ± 0.05 m/century subsidence as the result of post glacial rebound, about 0.07 m/century tectonic subsidence and about 0.01 ± 0.05 m/century subsidence as the result of deep layer compaction. Therefore, a 0.11 ± 0.07 m contribution due to vertical land movement is included in the projections for 2100. This number does not include the (usually very local) subsidence due to peat oxidation in polders and subsidence due to drainage and ground water and gas/oil extraction (Vellinga et al., 2009).” Based on this analysis, a value of 10 cm was added to the estimate of sea level rise (SLR) projection for 2100.

Chapter 4
Hydrodynamic Analysis

4 Hydrodynamic Analysis and Loading

The approach to model a wave loading scenario acting on the flood defence under investigation is to combine the current boundary parameters with the expected future increased values due to climate change.

4.1 Current hydraulic boundary conditions

Firstly, the current parameters need to be collected. They are grouped into parameters that are linked to the dike, to the water level, to the currents and to the wave height.

4.1.1 The Dike

According to Bossenbroek and Bardoel (2014), the dike sections and structures that are contributing the most to the flooding probability of the region under the present loading conditions are on the south side of the dike ring 29 near Vlissingen. In Figure 15 the dike sections which displayed the greatest contribution to the probability of flooding are shown.

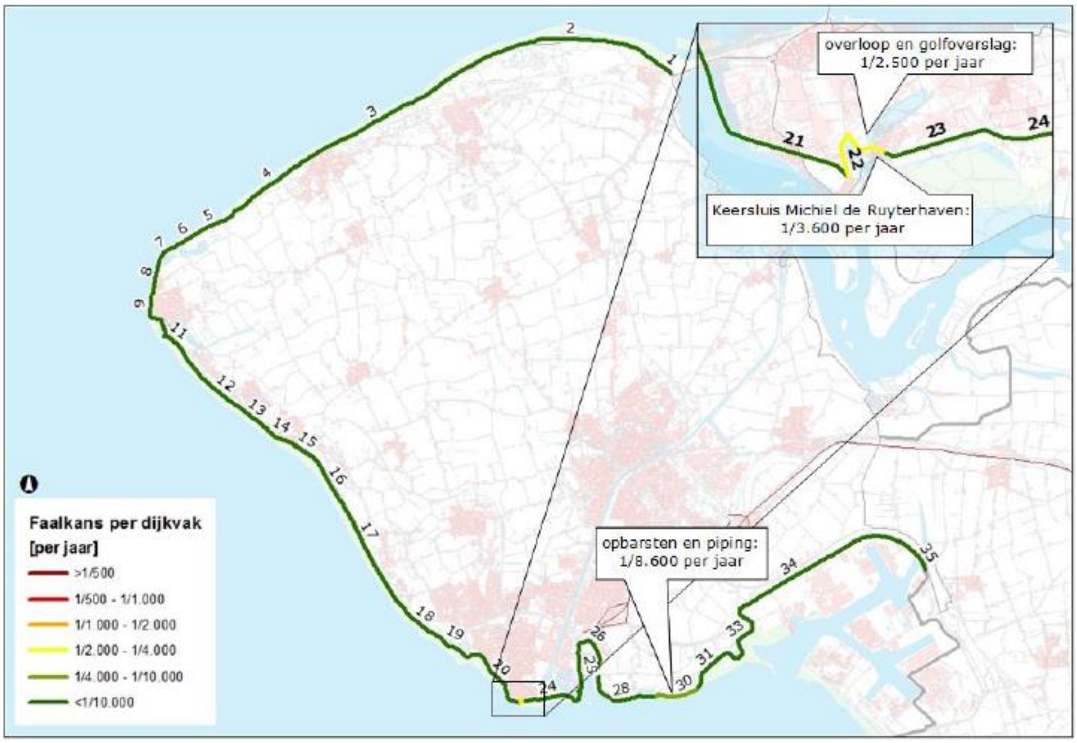
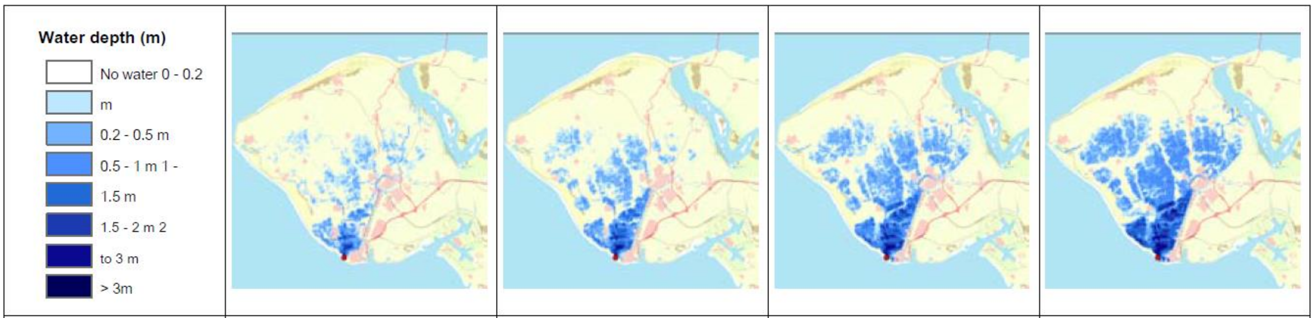


Figure 15: Failure probability for dike ring 29 sections (Bossenbroek & Bardoel, 2014)

However, as the focus of this case study is on the area of Westkapelle and its coastal dike, and a breach of the dike ring near Vlissingen is probably not going to have a great effect on Westkapelle due to the distance between them, the focus is on a dike section in Westkapelle. The effect of a breach of the dike in Vlissingen can be seen in Figure 16, presented by Bossenbroek and Bardoel (2014).

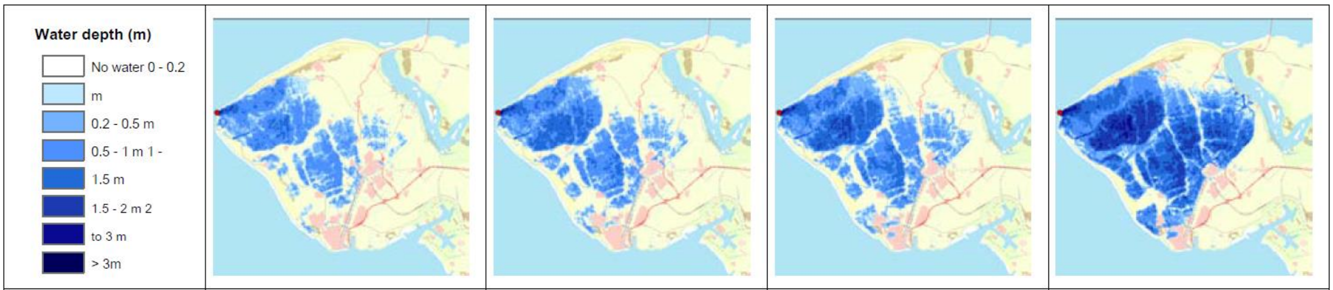
The damages and casualties would most likely be greater given a breach at this location compared to a breach at the coastal dike of Westkapelle, but due to the scope of the project the Westkapelle dike is being analysed.



Damage [M €]	555	905	1245	1550
casualties	10-45	20-75	40-140	65-225

Figure 16: Maximum water depth, damage and victims for breakthrough at Vlissingen, (Bossenbroek & Bardoel, 2014)

On the other hand, one can see a similar projection for the town of Westkapelle in Figure 17:



Damage [M €]	200	335	470	1145
casualties	0-5	5-15	5-20	20-70

Figure 17: Maximum water depth, damage and victims for breakthrough at Westkapelle, (Bossenbroek & Bardoel, 2014)

To make an approximation of the wave loading and thus the overtopping on the dike, a cross-section of the dike was needed. Although the cross-sections of the dike vary along its length, it has been approximated that the response to wave loading will be similar for all sections, and therefore the one cross-section will provide general results for the whole dike ring around Westkapelle. The location of the selected cross-section is illustrated in Figure 18. This location was selected for the calculations because cross sections of this location were provided from the client and it is a good representation of a general cross section.

Figure 18: Dike Location, Google maps

The specific design drawings of the cross-section as provided from Rijkswaterstaat Zeeland, Sea Defence Project Office can be found in Appendix B- Hydrodynamic Loading.

4.1.2 Water Level

Estimation of the water level is significant for prediction of wave run-up levels and wave overtopping of a flood defence. Furthermore, in shallow areas the extreme water level is a factor that usually determines the water depth and thus the upper limit of wave heights. Water levels in design or assessment of structures can consist of the following components: the mean sea level, the astronomical tide and surges related to (extreme) weather conditions. (J.W. Van der Meer et al., 2016)

4.1.2.1 Mean Sea Level

The mean water level can be taken as a site-specific constant when referring to coastal waters in open communication with the sea (J.W. Van der Meer et al., 2016). As it was previously discussed however, due to expected global warming, sea level is expected to increase. The present mean sea level in the region around Westkapelle as acquired by the Permanent Service for Mean Sea Level (PSMSL) webpage is around 2.63 cm above NAP (PSMSL, 2018). The way this was calculated can be found in detail in Appendix B- Hydrodynamic Loading.

4.1.2.2 Astronomical Tide

The tide can have a great effect both on the waves that can reach the coast, as well as on the overtopping calculations. The basic driving forces of tidal movements are astronomical and therefore entirely predictable, which enables accurate prediction of tidal levels (and currents). The tidal amplitude for the region is around 2.05m according to Bosboom and Stive (2015) as also shown in Appendix B- Hydrodynamic Loading.

4.1.2.3 Storm Surge

Extreme high-water levels are caused by the combination of high tidal elevations and a positive surge. This storm surge usually consists of three main components. First, a barometric effect caused by a variation in atmospheric pressure from its mean value. Second, a wind set-up, which in shallow seas such as the North Sea in this case, a strong wind can cause a noticeable rise in sea level within a few hours. Lastly, a dynamic effect due to the amplification of surge-induced motions caused by the shape of the land (e.g. seiching and funneling). Surges may become several meters for large return periods (J.W. Van der Meer et al., 2016). The maximum storm surge expected at Vlissingen is about 2 m (Klein, 2015), which is assumed to be roughly the same for Westkapelle.

4.1.3 Currents

Along the coast most currents are related to the tide and often they are the only ones considered in the design process. For marine design conditions, wind and wave-induced currents may also have to be considered. Currents affect the stability of rock armour of structures, interfere with wave propagation and indirectly, currents may affect a structure through erosion of the sea bed. However, "...in a marine environment waves are usually the dominant loading..." (CIRIA, CUR, & CETMEF, 2007). It is considered that currents have little effect (except the longshore sediment transport currents which can cause erosion of the foreshore and the dunes but have negligible effect to the selected rock-covered cross-section), thus they are not included in the analysis.

4.1.4 Wave Height

One could say that the waves are the most important variable to affect coastal structures. They are defined by their height, length and period. In literature opinions on the current wave height (significant wave height) at the region around Walcheren, and more precisely near the Westkapelle area were found to vary greatly.

Van Santen and Steetze modelled the area in order to calculate sediment transport (Giardino, Heijer, & Santinelli, 2014). According to them, "The wave climate of the outer delta shows two dominant wave directions: from the North and from the South-West, with a wave height which usually ranges between 0 and 2 m, and that during occasional storms is above 5 m." In Table 5 the hydraulic conditions that they used for their modelling is shown. It can also be seen that they use a return period of once every 4000 years which is the same return period being used for this project.

Table 5: Values for the hydraulic conditions in Westkapelle, (Van Santen, Steetzel, Van Thiel De Vries, & Van Dongeren, 2012)

Norm Frequency [1/a]	Max. Water Level [NAP+ m]	Max. Sign. Wave Height [m]	Max. Wave Peak Period [s]	Storm Duration [h]	Tidal Wave Amplitude [m]
1/4000	4.9	3.65	12.2	30	1.7

Another source, (Waterstaat, 2006) gives a range of significant wave heights from 3.7m to 5.2m with the same peak period Tp=12.2s and by considering the same return period of 4000 years. The lower margin of this range corresponds to the significant wave height that Van Santen and Steetze used for their modeling.

Perquin (2005) found that the wave heights (Hs) vary between 3.66 m and 4.89 m with the corresponding wave peak periods being between 10.90 s and 13.14 s. Again, the values are in a similar range but a difference of 20 to 40 cm in wave height is considered large and will give quite different results in the calculations.

From Boers (2008), Figure 19 is taken which gives the hydraulic preconditions for the whole dike ring 29 in 2006 (assumed to be the same in the present date). For the project focus (RSP 15-24) the values for spectral wave height Hm_0 (assumed equal with the significant wave height in deep waters) can be seen to be in the range of 3.5 to 5m with a peak period of 12.2 sec.

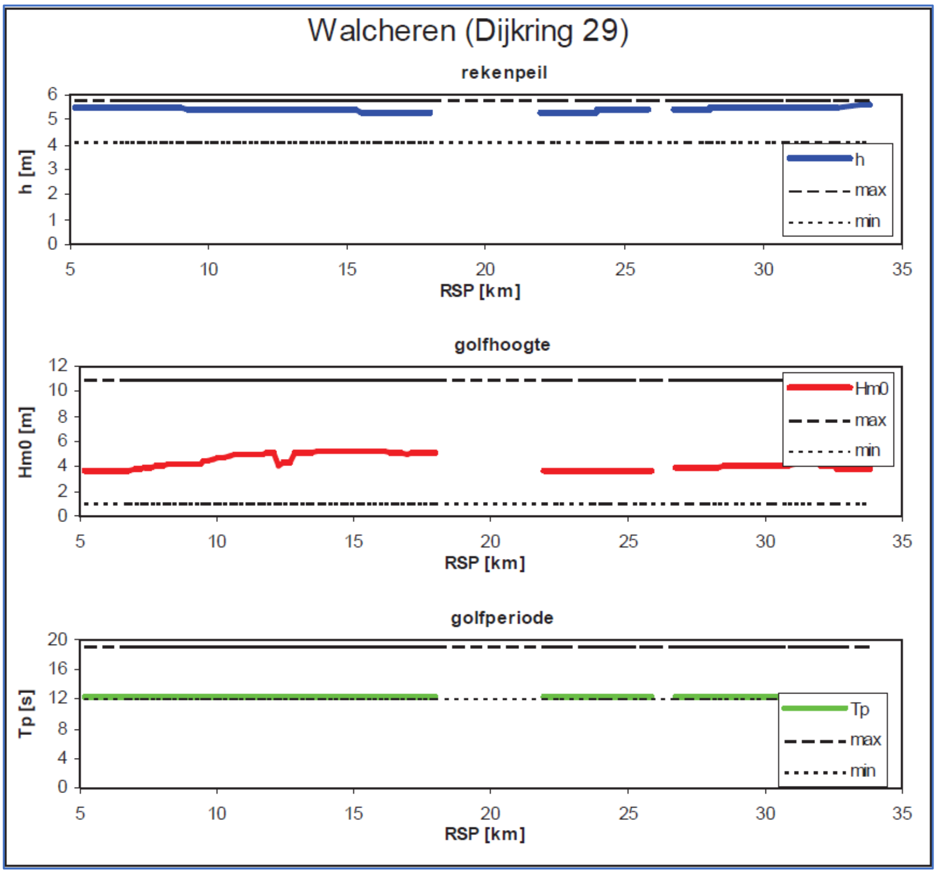


Figure 19: Hydraulic Preconditions (2006) for Walcheren: Sea Level, Hm_0 and Tp for dike ring 29 (max and min values refer to Netherlands), (Boers, 2008)

The data webpage of Rijkswaterstaat (Rijkswaterstaat, 2018) provides Figure 20 in which one can observe the maximum values for significant wave heights to range between roughly 2m and 4m (with the exception of 2016), while the average values are converging around a value slightly lower than 1m. These lower values can be explained as they refer only to the measured data and not to extrapolated wave heights for a return period of once every 4000 years.

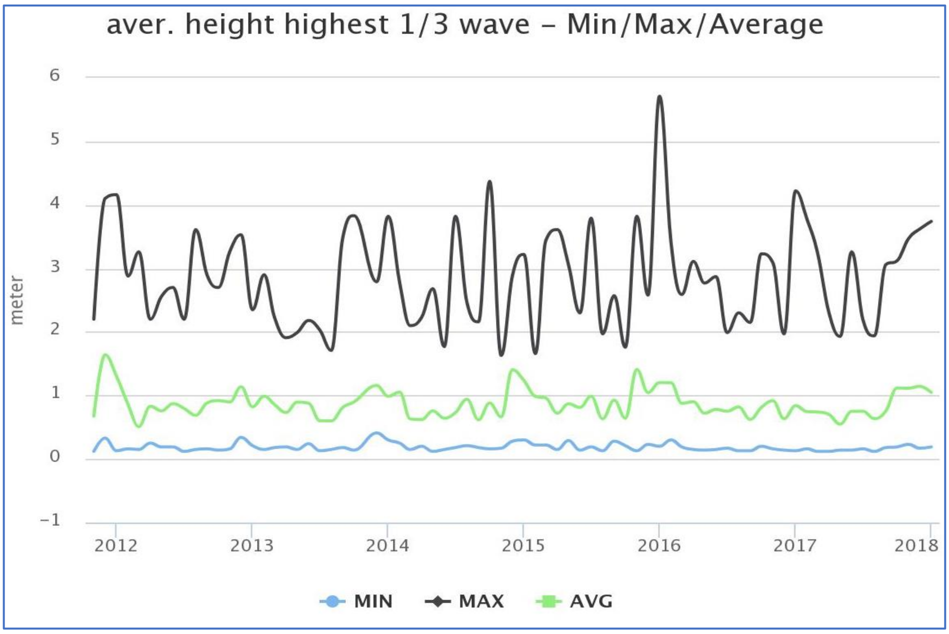


Figure 20: Values of significant wave heights near Westkapelle over the years (Rijkswaterstaat, 2018)

Finally, Giardino et al. (2014) provides Figure 21 which shows two dominant wave directions, one from the North and the other from the South-West. According to them "... wave height which usually ranges between 0 and 2 m, and that during occasional storm is above 5 m."

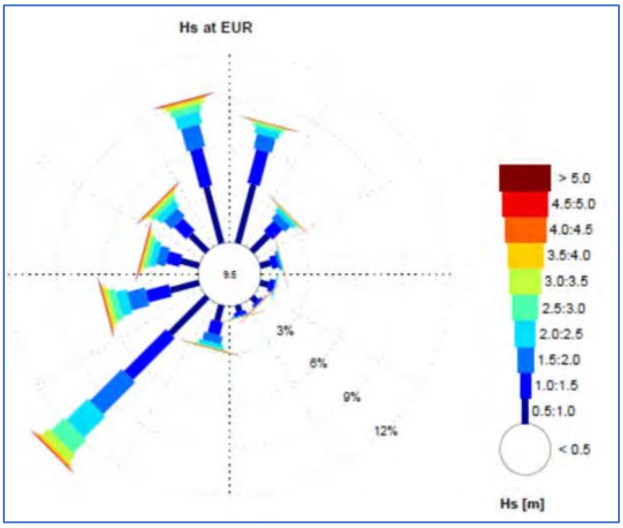


Figure 21: Wave climate at Europlatform station (Giardino et al., 2014)

One can conclude that the peak period is Tp = 12.2 sec as all the sources agree on the same value. The wave height data is summarized in Figure 22.

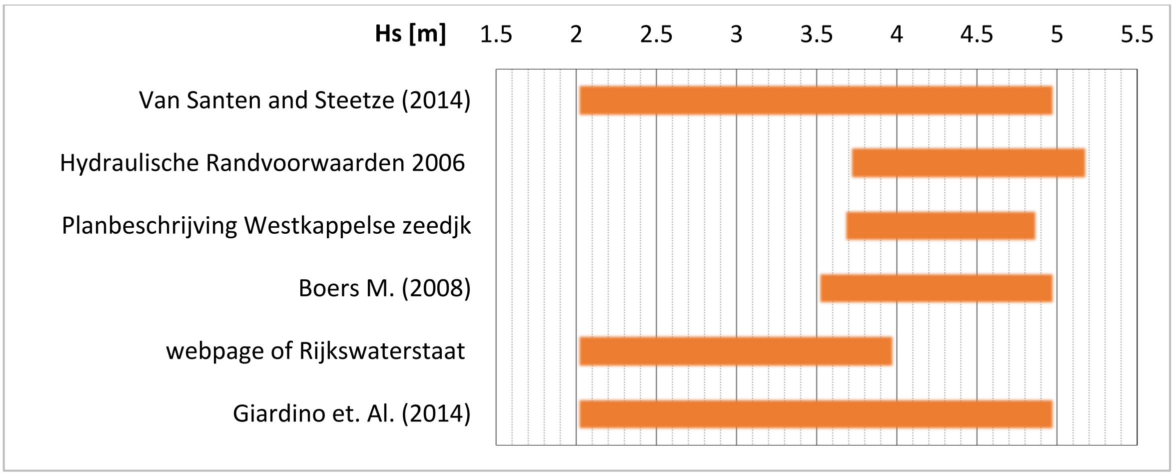


Figure 22: Ranges of Significant Wave Height (Hs) for Westkapelle from different literature sources

One solution would be to pick a value that fits within all these ranges to continue with the analysis, however the differences between the values are too great for a significant value such as the significant wave height which has a great impact on the results, and the uncertainty is deemed too large. In addition, these sources introduced the values without any reference to how they were calculated. Without knowing where their data originated, the return period that they considered for their calculations, in what type of depth these wave heights refer to (deep water or shallow water), as well as other assumptions that probably have been made for their calculation, it was determined that a thorough and more precise estimation of the value was necessary.

In order to estimate the wave height at the toe of the structure, the first step is to acquire the deep-water wave heights for the region. "Data in deep water, offshore of a site will be available either through the use of a computational wave prediction model based on wind data, or on a wave model. In both of these cases the offshore data can be used in conjunction with a wave transformation model to provide information on wave climate at a coastal site." (J.W. Van der Meer et al., 2016). A more recent option is to use data bases for a certain area that have been gathered by remote sensing (satellites).

The BMT Argoss wave climate tool provides deep water data for the region (significant wave heights, wave periods, wave directions, wind speeds and wind directions) for the last 25 years (BMT, 2018). The tool makes use of gathered data by satellites, combined with a wave model. This dataset then needs to be filtered and transformed through statistical methods and an extreme value analysis (peak over threshold method, fitting with a probability density function and extrapolation) to obtain one significant wave height value in deep water for the desired return period. This process is thoroughly treated in Appendix B.

The next step is to use the deep water significant wave height and with the help of a modelling tool transform it to a nearshore wave height. The SwanOne modelling is employed to calculate the interaction between the waves and the bathymetry, including processes like diffraction, refraction, shoaling, dispersion and wave breaking. The inputs and use of the model, as well as the assumptions made in this process can be found in Appendix B. The resulting values from the aforementioned process are given in Table 6.

Table 6: Present boundary conditions offshore and at the toe of the structure

		Offshore			Nearshore		
Water Level		**Hs [m]**	**Tp [sec]**	**U_{10} [m/s]**	**$H_{m0design}$ [m]**	**Tp [sec]**	**$T_{m-1,0}$ [sec]**
Normal condition WL (present)	+0.0263	7.23	12.2	28.7	0.68	12.2	7.75
Extreme condition WL (present)	+4.076	7.23	12.2	28.7	3.16	12.2	8.51

It can be seen that the calculated spectral wave height has a smaller value than the one given from the sources. Two possible reasons were identified. Firstly, the SwanOne calculations give the spectral wave height exactly at the toe of the structure. A location even a few meters seaward would result in a larger value of significant wave height. The given sources did not further specify which exact location was used. Secondly, the significant wave height from the sources, as it is calculated for actual designs, considers climate change (specifically projections for 2060). The value provided by the SwanOne calculations does not yet account for sea level rise due to climate change as the newest projections for climate change will be considered and added in the next step of the analysis.

4.2 Change in hydraulic boundary conditions due to Climate Change

As discussed before, the climate change projection that gives a value for sea level rise of 1.3m (including subsidence of land) was adopted. Furthermore, an estimated increase in deep water significant wave height of 18% of the original value was adopted. Additionally, it was assumed that astronomical tide levels remain constant, as well as the storm surge.

Thus, the deep water significant wave height, that had a value of 7.23m from the previous analysis, will become 8.53m for the case that considers the effects of climate change. The extreme conditions mean sea level will increase by 1.3m. However, one must consider that these climate change effects projections refer to a past date as a baseline. So, to analyze the change from the present date to 2100, one must subtract the effects that have already occurred. According to NASA the current sea level rise has been measured via satellite data to be 84.8 (+/- 4) mm (latest measurement October 2017) (NASA, 2018). Thus, the extreme conditions mean sea level is calculated to 5.29m.

The same SwanOne calculation procedure is conducted. This time climate change effects are considered which yields the results shown in Table 7. The details of the nearshore transformation, inputs and assumptions can be found in Appendix B. These values assume that there are no bathymetry changes.

Table 7: Boundary conditions offshore and at the toe of the structure

		Offshore			Nearshore		
Water Level		**Hs [m]**	**Tp [sec]**	**U_{10} [m/s]**	**$H_{m0design}$ [m]**	**Tp [sec]**	**$T_{m-1,0}$ [sec]**
normal condition WL (present)	0.0263	7.23	12.2	28.7	0.68	12.2	7.75
extreme condition WL (present)	4.076	7.23	12.2	28.7	3.16	12.2	8.51
extreme condition WL (climate change)	5.291	8.53	12.2	32.65	3.98	12.2	8.65

4.3 Overtopping

The analysis of the overtopping is done by closely following the directions of the EurOtop manual (J.W. Van der Meer et al., 2016) and by using the recommended formulas for the design and assessment approach. Both water level and wave height have been determined for the return period of 4000 years. However, the overtopping discharge for the combination of these extreme conditions will be larger than the actual overtopping occurring with that return period. According to J.W. Van der Meer et al. (2016) this is caused by the fact that "the combination of these two extreme values will have a lower probability of occurrence if the two are not fully correlated. Therefore, if the joint probability of occurrence is taken into account, a lower overtopping will be calculated". Assuming the simultaneous occurrence of the high-water level with high wave height, is therefore conservative. Be that as it may, determining the joint probability of occurrence of the extreme event water level and wave heights is beyond the scope of this project, thus the conservative option of simultaneous occurrence was selected.

The wave run-up height is given by $R_{u2\%}$. This is the wave run-up level, measured vertically from the still water line, which is exceeded by 2% of the number of incident waves. The number of waves exceeding this level is hereby related to the number of incoming waves and not to the number that runs up the slope. There is no constant discharge over the crest of a structure during overtopping. The process of wave overtopping is very random in time, space and volume. However, a mean overtopping discharge is widely used. Wave overtopping is the average discharge per linear meter of width, q in l/s per m of structure length (J.W. Van der Meer et al., 2016).

As wave set-up is a physical phenomenon that significantly affects the local water levels (order of $x10^1$ cm), thus it is also added to the water level for the overtopping calculations. The magnitude of wave set-up was also taken from the SwanOne modelling results. By using the methodology of the Eurotop manual, the run-up and overtopping are calculated. The results can be seen in Table 8.

Table 8: Results of run-up and overtopping calculations

	$R_{u2\%}$ [m]	q [l/s/m]
Present extreme scenario	4.89	2.28
Climate change extreme scenario	6.04	64.52

According to van der Meer (2002), one criterion of assessing the dike safety is the mean overtopping discharge rate, which depends on various conditions, from which three are most important. First, how passable or trafficable the dike crest and inner berms must be in view of emergency measures under extreme conditions, which is of great importance for dike managers. From experiments it was concluded that if people should be present on the crest of the dike the overtopping discharge should be less than 10 l/s per m. Second, the total volume of overtopping water regarding storage or drainage. Storage or drainage problems behind the dike may influence safety especially in the case of Westkapelle where the city is directly behind the dike. If so, the overtopping should be limited. Last, the resistance against erosion and local sliding of crest and inner slope due to overtopping water. (Meer, 2002)

The Dutch Guideline on river dykes (TAW, 1989) quotes "Which criterion applies depends of course also on the design of the dike and the possible presence of buildings. In certain cases, such as a covered crest and inner slopes, sometimes 10 l/s per m can be tolerated". In Dutch Guidelines it is assumed that the following average overtopping rates are allowable for the inner slope:

- 0.1 l/s per m for sandy soil with a poor turf
- 1 l/s per m for clayey soil with relatively good grass
- 10 l/s per m with a clay protective layer and grass according to the standards for an outer slope or with a revetment construction

However, tests have been performed for mean overtopping discharges in 2007-08, starting at 0.1 l/s per m up to 75 l/s per m. According to J. van der Meer (2018) “It seems unlikely that an inner slope with a clay cover topped with a grass cover (in Dutch situations) will fail due to erosion by overtopping waves with a mean discharge of 30 l/s per m or less”. Many dike sections withstood 50 l/s per m and some of them even 75 l/s per m. An obstacle like a concrete staircase on the inner slope was destroyed at a stage with 75 l/s per m overtopping but the dike itself was not in danger. No section failed for 30 l/s per m, which gives the basis for the preliminary conclusion (J. van der Meer, 2018).

From 2006 and onward, destructive tests have shown the behavior of various inner slopes of dikes, embankments or levees under simulated wave overtopping, up to a mean overtopping discharge of 125 l/s per m (Maarten, 2013). Future research may result in a conclusion.

Thus, it is concluded that guidelines and current research have not yet clarified specific acceptable limits of overtopping for dikes from which a comparison with the results can be made. In phase II of this project a more thorough analysis of an acceptable overtopping value will be made.

Chapter 5
Uncertainties and Sensitivities

5 Uncertainties and Sensitivities

There is a wide range of uncertainties in the present analysis. Some uncertainties are related to natural phenomena, while others are related to data collection, calculations and modeling. Consequently, it becomes very important to have a good understanding of the sensitivity of calculated wave heights at the toe of the structure, run-up and overtopping at the dike to changes in various parameters. Below, the sources of uncertainties and sensitivities are presented.

5.1 Uncertainty Analysis

5.1.1 Physical Factors

5.1.1.1 Failure Mechanism

In the current project, only the failure mechanism of overtopping is considered for the dike as other mechanisms are out scope. All other failure mechanisms (e.g. piping, geotechnical stability etc.) that are not included could impact the design. These other failure mechanisms are considered to make logical decisions, but they are not the deciding factors.

5.1.1.2 Difference in Cross-Sections

The team used one specific cross-section for the analysis. However, along the dike there is significant variance of the cross-sections with different lengths, heights, slopes and materials which lead to differences on their loading and resistance.

5.1.1.3 Overflow over multiple dike sections

In the current analysis, an average overtopping for the investigated cross-section was calculated and it was assumed that the same average overtopping applies on all the cross-sections around the area simultaneously. That is not necessary the case and a probability analysis should be done in order to investigate the probability and frequency of having a specific amount of overtopping over multiple dike sections at the same time.

5.1.2 Hydraulic Boundary Condition

5.1.2.1 Joint probability of waves and water levels

In the analysis a worst-case scenario was assumed. In the scenario the highest astronomical tide and the highest storm surge happen simultaneously with an extreme storm. The probability of a simultaneous occurrence of all three is however relatively low. Thus, an analysis which produces the joint probability of occurrence of these events (J.W. Van der Meer et al., 2016) (which will result in a lower overtopping than calculated) is recommend. Subsequently identifying the combinations of magnitudes of these events would then give the failure of the dike ring system. As pointed out by J.W. van der Meer et al. (2016) "Assuming the occurrence of the high water level together with high wave height (with the same return period), is therefore conservative."

5.1.2.2 Water Levels

In the present analysis wind set-up was not included. However, it can be considered small as the area is open and not a closed basin or an area encircled by land formations.

5.1.2.3 Currents

As already mentioned in chapter 4.1.3, currents were not included in the present analysis. However, wave and tide induced currents will have an effect on the wave propagation and on the morphology of the sea bed. This, in turn, might affect the wave height and the overtopping. More specifically, stronger currents may change the wave height, wave period and angle of energy towards the dike slope. According to J.W. van der Meer et al. (2016) "..wave periods become shorter if the waves are against the current and longer when they are along with the current."

5.1.2.4 Wave Heights

As already discussed in 4.1.4, the wave height at the toe of the structure is one of the most important parameters for the calculation of the overtopping. It has been shown in chapter 4 that various credible sources propose significantly differing values for the significant wave height for this region. This outcome demonstrates the uncertainty in the calculation of this parameter. A further explanation for the reasons of uncertainty in the wave height can be found in Appendix D.

5.1.2.5 Wave Period

The wave period was set to Tp=12.2 sec. However, one cannot know for certain that this value will remain unchanged in the future. In order to roughly estimate the sensitivity of this parameter, sensitivity calculations with Tp=9.75 sec and Tp=14.65 sec (increase and decrease by 20%) were conducted as shown in Table 10.

5.1.3 Climate change

5.1.3.1 Sea Level Rise (SLR)

Global sea level predictions are uncertain. Similarly, local sea level rise along the Dutch coast has different types of uncertainties that need to be considered (Delta Committee, 2008). The Delta commission (2008) classified the uncertainties into five broad areas based on their origin:

- incomplete or imperfect observations;
- incomplete conceptual frameworks
- inaccurate prescriptions of known processes;
- chaotic, or inherently unpredictable responses;
- lack of predictability due to non-physical factors (e.g. policy-decisions).

The local sea level rise along the Dutch coast potentially differs greatly from the global mean rise (Deltacommissie, 2008). The uncertainty of SLR projections results mainly from the local expansion of the ocean, as a result of changing ocean currents, and from the large ice sheets of Greenland and West-Antarctica (van den Hurk et al., 2014). The uncertainty of local ocean expansion, on the one hand, is estimated by making use of an ensemble of climate models. These models are also a major source of uncertainty (including future forcing and limited model skill) (van den Hurk et al., 2014). The uncertainty of ice sheet effects arises from the poor understanding of the dynamics of the large ice sheets of Greenland and West-Antarctica over the ocean. The quantification of this effect, called the gravity effect, is currently a matter of scientific debate (Deltacommissie, 2008).

Uncertainty in each of the SLR scenarios is included by expressing the scenario values as ranges (lower and upper bound). As sown in chapter 3, for each scenario, an upper and a lower bound is given which corresponds to a 5 – 95% confidence interval ranging from 0.40m to 1.30m of SLR. These values are estimated from the model simulations for the 21st century (Deltacommissie 2008). The range of sea level rise chosen for this analysis is +65 to +130 cm. Therefore, the sensitivity to water level variations is included by calculating the effects for overtopping for the full range of SLR scenarios.

5.1.3.2 Wave Height

To estimate the effects of wave height due to climate change, a wave model study of four emission scenarios/Global Circulation Models (GCM) combinations was one. These showed that the future projection for wind and significant wave height for the long-term 99th percentile increased by up to 7% and 18%, respectively, in the North Sea (Grabemann and Weisse 2008). However, according to the analysis result of Grabemann and Weisse (2008), for both extreme wind speeds and significant wave heights, "the uncertainties introduced by different models are generally much larger than those caused by different scenarios (Grabemann and Weisse 2008)."

The source of these uncertainties introduced by different models could be generally due to the uncertainties in the future development of society, as well as the uncertainties in the formulation of the global climate models (Grabemann and Weisse 2008). This results in different climate change signals for the different emission/GCM combinations (Grabemann and Weisse 2008). The result of this analysis including the uncertainties is shown in Table 9. For the long-term 99th percentiles of a given emission/model combination, the model uncertainties for the significant wave height ranges between about 0.1 and 0.6 m (Grabemann and Weisse 2008).

Table 9: Uncertainties within SLR and wave climate

	Range	Confidence Interval	Uncertainties Range
SLR (Delta Commission 2008)	65-130cm	5-95%	-
Hs	7-18% (of present values)	99%	0.1-0.6m

Sensitivity for deep water wave height variations is estimated by calculating the parameters for a deep water wave height with 7% increase from present value (instead of 18% which was the reference case).

5.1.4 Wave Run-up and overtopping calculations

As previously discussed, one source of uncertainty in the overtopping values is the simultaneous occurrence of the 4000-year return period storm and the maximum storm surge and high tide, instead of the creation of a joint probability of occurrence. Another uncertainty is introduced via the consideration of normal wave attack instead of oblique. Both of these considerations produce a higher overtopping value. Another uncertainty comes from the assumption of γ_f =0.85, whereas the exact number that corresponds to the used materials is not known. Additionally, the formulas from EurOtop were used for the run-up and overtopping calculations. According to J.W. van der Meer et al. (2016), "All of the prediction methods given in this report have intrinsic limitations to their accuracy" and continues that "...it can be concluded that overtopping rates calculated by empirically derived equations, should only be regarded as being within, at best, a factor of 1 - 3 of the actual overtopping rate. This means that the actual overtopping rate could be three times smaller as well as three times larger than the predicted mean value."

The calculations were done using EurOtop manual's 'Design or assessment approach' formulas. These formulas were derived from empirical data which are scattered, thus contain uncertainty. To include that, the formulas follow a semi-probabilistic approach with a partial safety factor. The formulas are given with a mean value of the stochastic parameter(s), but with the inclusion of the uncertainty of the prediction. The stochastic parameter(s) becomes $\mu + \sigma$ to include the safety factor for the design and assessment approach. In graphs, the 5%-exceedance line or 90%-confidence band is given to complete the comparison as can be seen in Figure 23 and Figure 24 (J.W. van der Meer et al., 2016). As these parameters are stochastic and so using a specific value includes an uncertainty.

5.1.4.1 Run-up formula:

In the equation for the calculation of $R_{u2\%}$, the coefficient 1.65 is considered as the stochastic variable with a mean value of 1.65 and a standard deviation of σ = 0.10. For the design and assessment approach the value of 1.75 was used to be conservative (J.W. van der Meer et al., 2016).

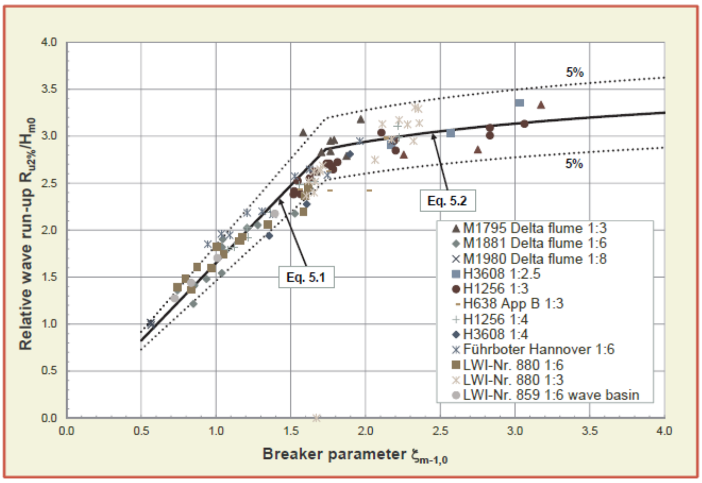


Figure 23. Wave run-up for relatively gentle, smooth and straight slopes. Source: (J.W. van der Meer et al., 2016)

5.1.4.2 Mean Overtopping formula:

In the equation for the calculation of q, the coefficients 0.023 and 2.7 are the stochastic variables with a mean value of 0.023 and 2.7 and a standard deviation of σ = 0.003 and 0.20 respectively. For the design and assessment approach the values of 0.026 and 2.7 were used to be conservative (J.W. van der Meer et al., 2016).

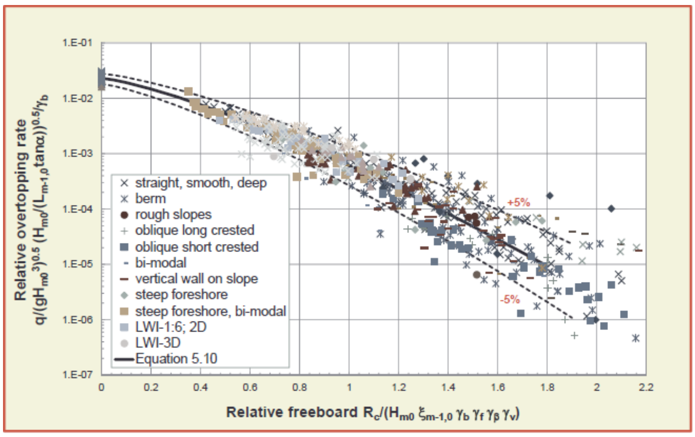


Figure 24. Wave overtopping data for breaking waves and overtopping Equation 5.10 with 5% under and upper exceedance limits (= 90%-confidence band). Source: (J.W. van der Meer et al., 2016)

5.2 Sensitivity Analysis

To assess the sensitivity to changes in various parameters, the wave heights at the toe of the structure, the run-up and overtopping at the dike for different scenarios were calculated and are presented in Table 10. The different scenarios are as follows:

1. Present: The present extreme weather conditions scenario.
2. Climate change reference: The high boundary expected climate change scenario (+1.3m WL) that was presented and used for calculations in chapter 4 and is now used as a reference for comparison with the other scenarios.
3. $H_{S,Deep}$ 7%: Scenario including climate change where a deep water wave height increased by 7% instead of 18% from present situation is considered.
4. 0.5×U_{10}: Scenario where the wind speed is reduced by 50%.
5. 1.5×U_{10}: Scenario where the wind speed is increased by 50%.
6. Perpendicular wind: Scenario where the wind is considered having direction perpendicular from the reference case (90^{o} from wave propagation direction).
7. Opposite wind: Scenario where the wind is considered having the opposite direction from the reference case (180^{o} from wave propagation direction).
8. Bathymetry -1m: Scenario where the depth of the bathymetry line is decreased by 1m (1m shallower sea bed).
9. Bathymetry +1m: Scenario where the depth of the bathymetry line is increased by 1m (1m deeper sea bed).
10. Bathymetry +2m bar: Scenario where a 2m high bar near the toe of the structure was added.
11. 0.8×Tp: Scenario where the wave peak period is reduced by 20%.
12. 1.2×Tp: Scenario where the wave peak period is increased by 20%.
13. Water level -0.65m: Scenario where the water level increase due to climate change is 0.65m lower than the high boundary climate change expectation of 1.3m (reference) (low boundary SLR).
14. Water level -0.325m: Scenario where the water level increase due to climate change is 0.325m lower than the high boundary climate change expectation of 1.3m (reference) (middle of the range of SLR).

Table 10: Calculated values at the toe of the structure for various scenarios.

	OFFSHORE					TOE OF THE DIKE								
Scenario	**Water level [m]**	**Hs [m]**	**Tp [s]**	**U_{10} [m/s]**	**U_{dir} [deg]**	**Wave Set-up [m]**	**Water Level**	**Tm_{01} [sec]**	**H_{m0} [m]**	**H_{m0} Diff. [m]**	**$Ru_{2\%}$ [m]**	**$Ru_{2\%}$ Diff. [m]**	**q [l/s/m]**	**q Diff. [l/s/m]**
present	4.076	7.23	12.2	28.7	0	0.392	4.468	8.51	3.16	-	4.89	-	2.28	-
climate change (reference)	5.291	8.53	12.2	32.65	0	0.407	5.698	8.65	3.98	-	6.04	-	64.52	-
$H_{S,Deep}$ 7%	5.291	7.74	12.2	30.25	0	0.288	5.579	8.69	3.92	-0.06	5.96	-0.08	51.92	-12.60
$0.5\times U_{10}$	5.291	8.53	12.2	16.325	0	0.323	5.614	8.86	3.98	0.00	6.06	0.02	59.74	-4.78
$1.5\times U_{10}$	5.291	8.53	12.2	48.975	0	0.395	5.686	8.14	4.02	0.04	6.06	0.02	63.21	-1.31
perpendicular wind	5.291	8.53	12.2	32.65	90	0.318	5.609	8.73	3.90	-0.08	5.94	-0.10	52.67	-11.85
opposite wind	5.291	8.53	12.2	32.65	180	0.318	5.609	8.88	3.98	0.00	6.06	0.02	59.52	-5.00
bathymetry -1m	5.291	8.53	12.2	32.65	0	0.421	5.712	8.45	3.35	-0.63	5.12	-0.92	25.1	-39.42
bathymetry +1m	5.291	8.53	12.2	32.65	0	0.34	5.631	8.75	4.13	0.15	6.27	0.23	72.58	8.06
bathymetry +2m bar	5.291	8.53	12.2	32.65	0	0.38	5.671	8.61	3.77	-0.21	5.74	-0.30	46.91	-17.61
0.8×Tp	5.291	8.53	9.76	32.65	0	0.271	5.562	7.34	3.79	-0.19	5.66	-0.38	35.2	-29.32
1.2×Tp	5.291	8.53	14.64	32.65	0	0.393	5.684	9.76	4.18	0.20	6.39	0.35	89.91	25.39
Water level -0.65m	4.641	8.53	12.2	32.65	0	0.453	5.094	8.54	3.57	-0.41	5.47	-1.22	15.42	-49.10
Water level -0.325m	4.966	8.53	12.2	32.65	0	0.369	5.335	8.59	3.78	-0.20	5.76	-0.61	30.75	-33.77

In Figure 25, Figure 26 and Figure 27 one can observe the difference of calculated wave heights, run-up and overtopping for each scenario, compared to the reference case.

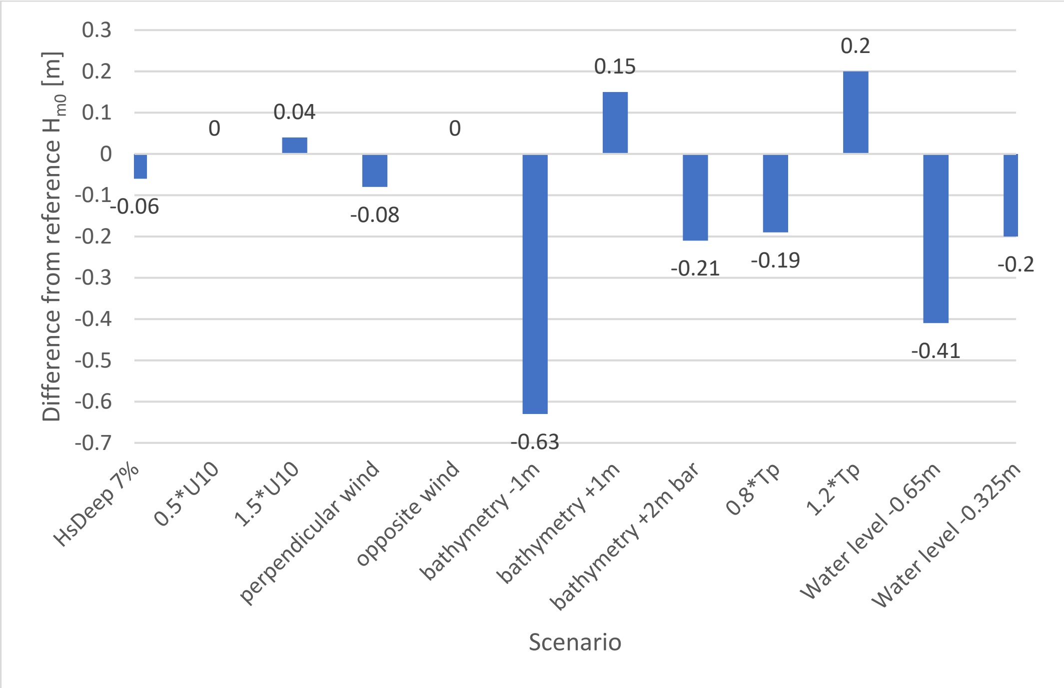


Figure 25: Wave height at the toe of the dike difference, compared to reference case wave height, for each scenario.

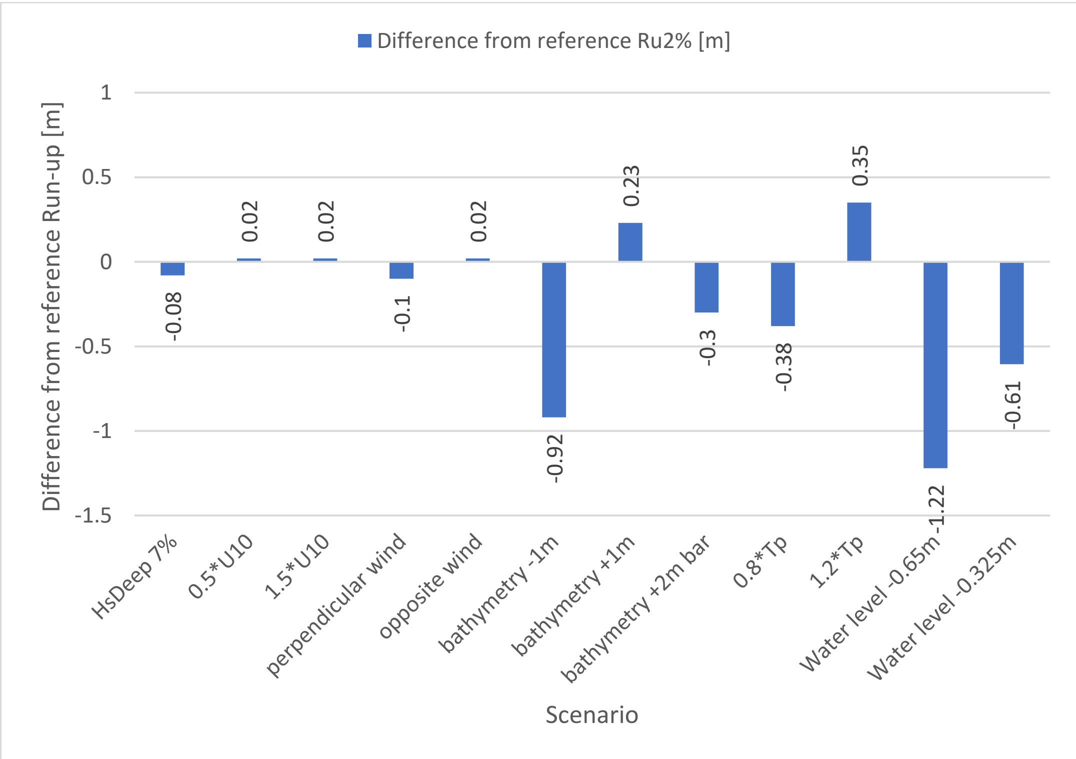


Figure 26: Run-up (2%) difference, compared to reference case run-up, for each scenario.

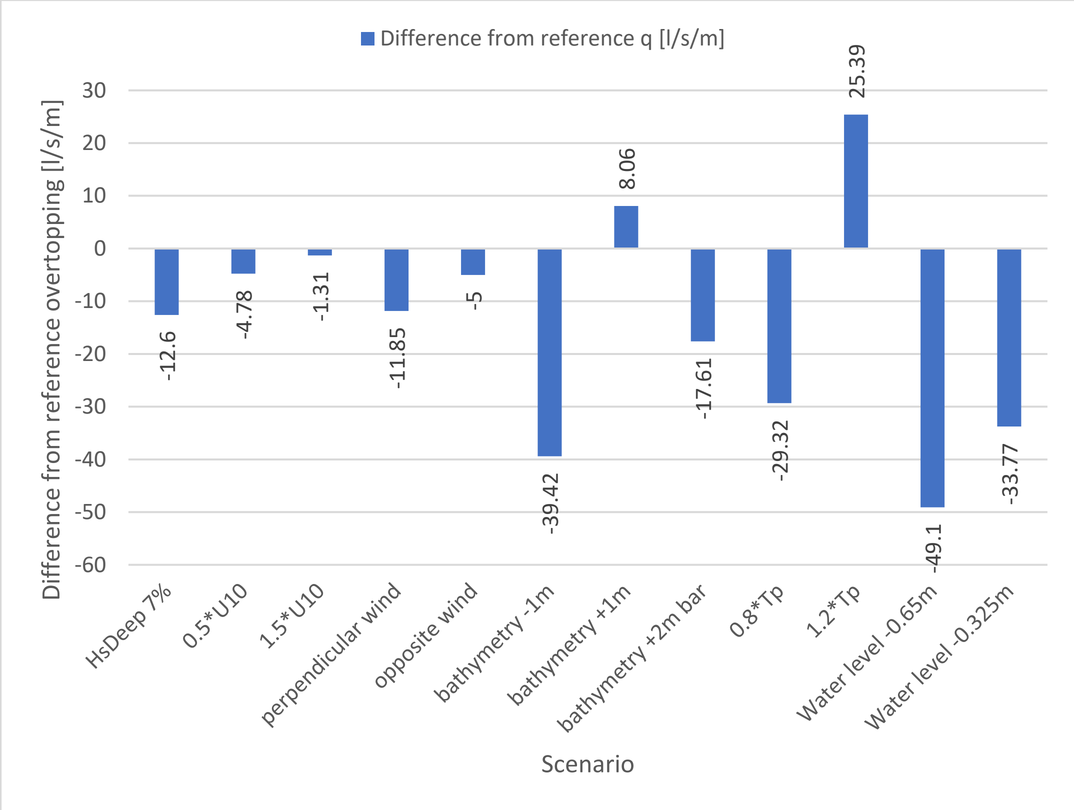


Figure 27: Overtopping difference, compared to reference case overtopping, for each scenario.

As can be observed from the data in Figure 25, Figure 26, Figure 27 and Table 10, changes in wind speed and direction have a negligible effect, thus the assumptions made will not have a great impact on the outcome. It should be mentioned that wind is not included in the overtopping formulas. It should be analyzed how wind can affect the thin layer of water running up a dike slope and how that could affect the mean overtopping. Changes to the peak period, as well as deep water wave height, can be observed to have moderate effects which can easily be neglected. For instance, the wave period is not expected to change as much as the input value for the sensitivity calculation. However, it does provide a good indication of the coupling between peak period and overtopping. That is a useful insight that can be used since the later proposed design could affect the local wave periods. The strongest effect can be observed if there are changes in bathymetry and water level. That can be reasoned as changes in bathymetry affect wave-seabed interactions and wave breaking which greatly dissipate wave energy. Unfortunately, the sea bed changes are very hard to predict as the coast morphology is tightly connected with the weather conditions, water level and local sediment. Water level on the other hand is linearly connected with the freeboard (Rc), an important factor for the overtopping calculations. Not only that, but changes in water level have also the same effect with changes in bathymetry as they 'bring closer' or 'bring further' the surface and thus the waves to the bed.

The sensitivity analysis demonstrates the importance of water level for dike run-up and overtopping, meaning that a small change in climate change induced sea level rise will greatly impact the loading and the measures that need to be taken in order to counter it. The team has taken this into account for the proposed design.

Chapter 6
Stakeholder Analysis

6 Stakeholder Analysis

A stakeholder analysis is a process that outlines all groups or individuals that will be affected by a project, how the project will impact them, and their individual interests related to the project (Brugha & Varvasovszky, 2000). A coastal protection project has many stakeholders since it involves the entire design and construction process, as well as all of the individuals or groups who will be impacted by the new dike design and all groups who would be impacted if the dike were to fail.

6.1 Approach

The stakeholder analysis will be done slightly differently for phase 1 and phase 2 of the project. In phase 1 the aim is to identify all who are affected by this project and their individual interests. This includes a description of each stakeholder and analyzing their relative influence and interest in the project. A visual representation of each stakeholder's value in this project will allow the team to determine which needs are most important to satisfy in the design options, if it is not possible to satisfy all. In phase 2, a further stakeholder analysis will be done to determine how the design specifically exceeds, meets, or fails to meet the interests of each stakeholder.

6.2 Stakeholder Identification

To identify each stakeholder, the team went through the entire design process and noted each group or individual that would have an interest in the project. The list of the stakeholders can be seen below in Table 11.

Table 11: Stakeholder identification

Type of Involvement	Stakeholders
International	EU Commission
Government	Politicians / legislators / mayor
	Waterboard Scheldestromen
	Ministry of Infrastructure and Water Management (Rijkwaterstaat)
	Ministry of Economic Affairs and Climate Policy
	Ministry of Health, Welfare and Sport
	Ministry of Agriculture, Nature and Food Quality
Unions	Farmers
	Fishermen
Economy	Port of Middelburg
	Local Businesses
Public	Local Inhabitants
	Media

6.2.1 International

The EU Commission is a stakeholder for this project because of the laws that the design must abide by, issued by the EU. The law on sustainable management contains a section on coastal protection and outlines what can and cannot be done during construction for coastal projects. It provides guidelines to be followed including not disturbing natural processes unless mandatory, carrying out a risk analysis, and considering the future generations in design (Europe, 2000). The guidelines set by the EU must be considered during design.

6.2.2 Government

The government is a broad category including multiple ministries and people that are stakeholders for this project. The local municipality including politicians, legislators and the mayor are stakeholders because a coastal dike improvement would be part of a local political decision, as well that politicians could use coastal protection improvements as part of their political platform. The local municipality oversees the formulation and adoption of structural perspectives involved in future spatial planning, which the coastal dike directly relates to (ProDemos, 2013). Politicians and legislators would like to see the project improve the community and follow political guidelines.

The most influential governmental stakeholder is the local waterboard, named waterboard Scheldestromen. This waterboard would have authority over the detailed design for this project and would decide whether to implement it or not. They would then go to Rijkswaterstaat to finance the project. Rijkswaterstaat is an executive organization of the Ministry of Infrastructure and Water Management that strictly deals with creating a sustainable living environment by protecting against flooding. The dike improvements would therefore have to merge with their national flood protection plans (Netherlands).

There are multiple other national level governmental ministries that each have a specific involvement towards this project. The Ministry of Economic Affairs and Climate Policy would like to ensure that the project meets their goal of creating an excellent environment for business by paying attention to nature and the environment (Netherlands). This ministry would therefore like the project to encourage business growth in the local communities by increasing protection to the cities of Westkapelle and Middelburg.

The Ministry of Health, Welfare and Sport is a stakeholder because it overlooks and strives for a healthy Netherlands (Netherlands). This means that the people are supported and there is welfare available for those in need. Relating to this project, this ministry would like to see that the public are safer from a catastrophe (such as the case of severe flooding).

The Ministry of Agriculture, Nature and Food Quality aims to ensure good prospects for the Dutch farming, horticulture and fishing sectors (Netherlands). This ministry is therefore a stakeholder because the dike improvements can reduce damage to agricultural land by reducing the flood risk or the amount of nearby land that would be affected by a breach. This ministry would like to see the negative impact on agriculture and fishing sectors from flooding minimized.

6.2.3 Unions

The two unions considered as stakeholders include the local farmers union and fisherman union. The farmers union is known as ZLTO, which is a division of LTO (Agriculture and Horticulture Organization) in the Zeeland region. This union comprises of 15000 farmers, and represents their interests (ZLTO). The agriculture union would strongly like to ensure that their farm land is not damaged, since agriculture is their jobs, but also essential for the Dutch economy. Like farmers, fishermen would like to see that their industry is not impacted negatively by this project either. The construction and design most likely won't have any impact on fisherman, but if the design were to fail and a breach were to occur, damage to the local ports could result in extensive damages to the fishing equipment.

6.2.4 Economy

The local economy contains vital stakeholders related to this project. First, the port of Middelburg located to the east of the town is a stakeholder. The port would like to see that the design does not inhibit port access or routine events. Also, the port would like to ensure that there are not extensive damages if an extreme event were to occur.

Local businesses are stakeholders since they too would like their business to not be impacted negatively. Local businesses located close to the coastal dike would like to not be impacted during construction, or have their business environment altered due to dike improvements. Those not directly near the dike would like damage of extreme events to be as minimal as possible. Local businesses also desire that the design does not reduce business in anyway, such as by limiting the access for customers.

6.2.5 Public

The public stakeholders include local inhabitants and the media. Local inhabitants are extremely relevant stakeholders since it is their property and lives at stake. They are directly impacted by the design and the long-term effects of the design. Local inhabitants would like the design to be aesthetically pleasing but most importantly capable of withstanding an extreme storm event and reducing damage and fatalities as much as possible. This is justified by interviews that the team conducted with locals while on their site visit. The media is a minor stakeholder who are indirectly impacted by this project. The media could get local and national headlines for the project, but no physical impact to the media is made.

6.3 Influence vs Interest

An influence vs interest chart illustrates which stakeholders are most important to the project and therefore who's interests the design should aim to satisfy. The chart can be seen below in Figure 28.

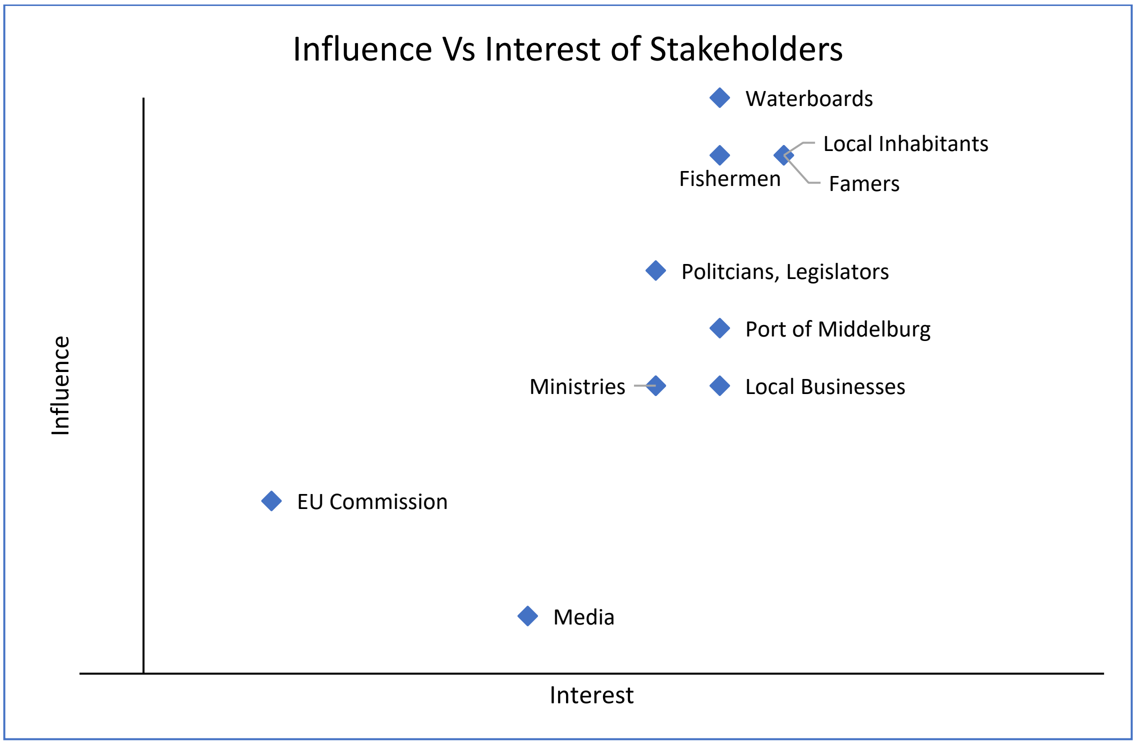


Figure 28: Stakeholder Influence vs. Interest

The local inhabitants and the farmers were noted to have the largest interests in this project due to the direct impact that flood safety has on their lives and the regions greatest economic resource. They also have high influence since through similar projects farmers and local inhabitants have had a large say on the project. The highest influence goes to the waterboards. The regional waterboard will create the design and have most control over the project, and Rijkwaterstaat, the governmental body overseeing national flood defence systems, will be the financer. The least influence and interest are the media and EU commission. This is because the guidelines that the EU commission state will already be included in the design since they are in line with the problem definition, and the media does not have a large impact on coastal engineering.

Chapter 7
Multicriteria Analysis

7 Multicriteria Analysis

A wide variety of options is available for the mitigation of flood risk. The team decided to conduct a MCA to put the options into context. In a MCA every single option is scored with regard to a set of predefined criteria. A weighting is then applied to the criteria to highlight the relative importance of specific aspects. With regard to robustness, one of the biggest advantages of MCAs is that they "… have the potential to capture a wide range of impacts that may not be readily valued in monetary terms, especially those relating to social issues." (Risk and Policy Analysts Ltd, 2004)

The main goal of this project's MCA was to gain an understanding of different options' potential. Through the MCA the strengths and weaknesses of options can be identified. More specifically, it allows the identification of potential for resistance and resilience. On the one hand, the MCA highlights favorable options with its ranking. On the other hand, potential for combination can be identified, e.g. when option A performs bad in a certain group of criteria, but option B makes up for it. Additionally, the MCA was a useful tool in generating the design approaches. All the design approaches focus on a given topic, namely maximum resistance, maximum resilience, stakeholder benefit, cost and good-for-all. They are explained in depth in chapter 7.

Firstly, two basic steps were conducted. On the one hand, criteria had to be formed. This was done by following recommendations in literature and through extensive group discussions. On the other hand, design options had to be collected. This was done by individual research of the team members. Secondly, the design options were graded according to the formed design criteria. Thirdly, weightings were applied to the criteria. Each set of weightings was adjusted to the topic of a design approach, e.g. for the design approach "Cost" all the criteria that had to do with costing were given the highest weighting. As five design approaches were considered, the result was five different rankings of all the options. The order of steps can be seen in Figure 29.

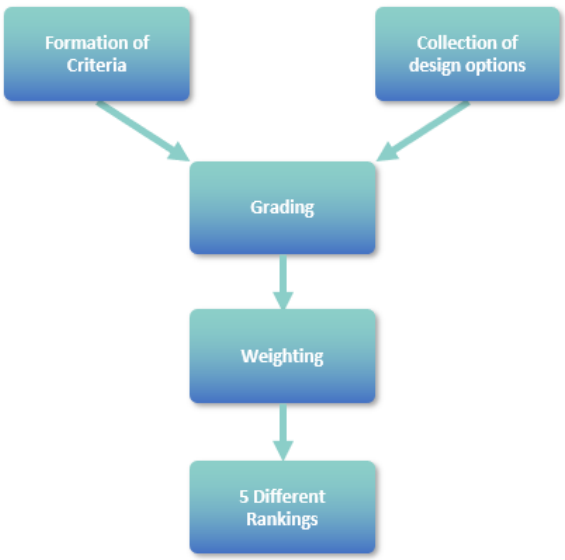


Figure 29: Basic Flowchart of MCA

In the following section of the report the single steps of the MCA will be explained in more depth.

7.1 Design Criteria

Before forming criteria, a discussion led to an outer framework for criteria. Firstly, it was stressed that vague criteria need to be avoided. The criteria were supposed to address a specific topic and should leave as little room for ambiguous interpretation as possible. Secondly, a grouping of the criteria was supposed to be possible. Thirdly, the number of criteria was set to a range from 6 to 20. The team wanted to keep the number of criteria as low as is consistent with forming a well-conceived decision. A brainstorm session yielded many criteria. The criteria were then assessed with regard to their ability to cover all areas of interest and overlapping of criteria, as advised (Middlesex University Flood Hazard Research Centre, 2014). This led to the criteria as given in Table 12.

Table 12: Criteria of the MCA

Group	Single Criteria
Resistance	Prone to breach/catastrophic failure
	Resistance to overtopping
Resilience	Prone to damages
	Resilience gain (system level)
	Expected amount of overtopping
Aesthetics/Environment	Landscape, visual amenity and recreation
	Environmental impact
	Cultural Modification
Economy	Cost
	Maintenance
	Flexibility/Adaptability to future situations
	Impacts on local businesses
	Land use
	Multi-Functionality

The resistance criteria reflect on the options' potential to increase the system's ability to withstand disturbances. The criterium "Prone to breach/catastrophic failure" highlights how the option affects the probability of a dike breach with its catastrophic consequences. Certain options might decrease this probability, whereas others might have the side-effect of increasing it. The criterium "Resistance to overtopping" takes the level of damages, that overtopping causes, into account. The resistance to overtopping clears the path for certain resilience options, that allow a level of overtopping and flooding of the system. Overtopping can only be allowed if the structural safety is guaranteed and erosion of the dike surface does not lead to a breach. That is what options with a high score in "Resistance to overtopping" ensure.

The resilience criteria reflect on the options' potential to increase the system's ability to quickly recover from the response to disturbances. The criterium "Prone to damages" refers to minor to medium damages to the dike that may or may not require repairs. Major damages, like breach and catastrophic failure, were already accounted for in the criterium "Prone to breach/catastrophic failure", that was mentioned above. The criterium "Resilience gain" stresses the option's impact on resilience on a system level. Certain options might, for example, decrease the recovery time of the system so that it gets back to 100% functionality more quickly. The criterium “Expected amount of overtopping” refers to the option’s impact on the amount of water masses that can be expected to averagely surmount the dike due to wave overtopping.

The aesthetics/environment criteria reflect on the option’s potential to impact peoples’ aesthetic perception of the environment and the environment itself. The criterium “Landscape, visual amenity and recreation” stresses this aesthetic impact and includes influences on recreational activities (e.g. a dike made from glass might have a disturbing appearance). The criterium “Environmental impact” focuses on the ecosystem and natural processes. The criterium “Cultural Modification” highlights the impact on historical sites and cultural buildings. This specifically refers to the changes that the option makes to the dike system (e.g. dike heightening that requires the demolition of historical sites).

The economy criteria reflect on the option’s economic aspects. The criterium “Cost” takes the option’s costs relative to other options’ costs into account. The criterium “Maintenance” refers to the required level of maintenance that is necessary under design conditions. Repairs, following from extreme events, are excluded. This is because the damages from extreme events are accounted for in the criterium “Prone to damages”. A clear distinction between construction costs and maintenance costs was made because the national agency Rijkswaterstaat only covers the construction cost of the concerned flood safety measures. Maintenance cost must be covered by the local waterboard which may influence the preferences of the local waterboard as a stakeholder significantly (guidance by supervisor, 4th meeting). The criterium “Flexibility/Adaptability to future situations” stresses future developments. How flexible is the option and is it possible to adjust to unforeseeable changes in loading conditions (as there is no guarantee that the chosen climate change scenarios are correct)? The criterium “Impacts on local businesses” stresses the influences on local businesses like farms, fisheries, supermarkets, shops and so on. Apart from flooding’s direct damage the means of transport, that they require to run their business, might also be impacted, for example. The criterium “Land use” stresses the amount of additional land that will be blocked from other use due to the option. The criterium “Multi-Functionality” highlights the potential for multifunctional use. A dike heightening, for example, might enable the conversion of the dike into a parking lot.

In three stages the criteria are used to compare the options. Firstly, every option is summarized, to provide every team member with an overview, and then scored in all the criteria. The lowest score is set to -10 and the highest to 10. A score of 0 stands for no-effect on the given criterion. Therefore, scores between -10 and -1 indicate a negative influence and scores between 1 and 10 a positive influence. The more accurate the scoring, the better for the second stage. In this stage the subjective scoring will be transferred into rankings. The result is a ranking of options for every criterion. This reduces the ambiguity of the scoring and introduces a causal relationship between the options – one option is better than another one but worse than a third, and so on (Middlesex University Flood Hazard Research Centre, 2014). The full scorings can be seen in the MCA tables that are given in Appendix E.

In the third stage, a weighting is applied to each category. This weighting stresses the relative importance of the category for the main design goals. The design goals will vary for every design approach so that five sets of weightings are applied. Risk and Policy Analysts Ltd (2004) stresses that the most crucial part of an MCA is “… ensuring that the weights are credible and justifiable.”. Therefore, the team thoroughly considered the weighting of the criteria to assure it is appropriate. To do so, a swing weighting procedure is adopted. The procedure is explained in the following paragraph.

Firstly, the criteria were listed according to their importance with regard to the respective design approach. A weighting is then given to the most important criterium, say 100%. The second most important criterium will be given a share of this weighting, depending on how important it is in relation, say 50% if the second criterium is half as important. This follows for all the criteria. The resulting weightings for all the design approaches can be seen in chapter 8.1. As a result, all the options will be ranked for every design approach.

From the weightings, five different rankings of all the options were obtained. The rankings focus on maximum resistance, maximum resilience, stakeholder benefit, cost and good-for-all. From the rankings design approaches were developed that harmonize the single options to make the design approach feasible. The following section introduces the options.

7.2 Design Options

The design options are grouped in resistance options, resilience options and policy options. This is because they all approach flood risk minimization from a different point of view. Whereas resistance and resilience options mostly focus on constructive measures, policy options aim to change the behaviour of individuals.

7.2.1 Resistance Options

7.2.1.1 Dike Heightening

For a heightening of the dike additional soil is placed on the crest and slopes of the levee. As the slope angle should not be increased, a heightening commonly comes with a widening. The idea of dike heightening as a flood risk management strategy dates back multiple centuries (Ciria, 2013). But conventionally, the heightening was only undertaken after a flood event had shown that the available dike height was not sufficient. The dike was then heightened up to the measured flood level plus an arbitrarily chosen additional safety buffer. Dike heightening decreases overtopping, as the magnitude of run-up to overtop the dike is raised. An image of dike heightening can be seen in Figure 30.

Figure 30: Dike Heightening, www.news-press.com

7.2.1.2 Dike Widening

For a widening of the dike additional soil is placed on either the seaward or landward slope. Coastal dikes usually have a gentle seaward slope and rather steep landward slope (Ciria, 2013). Regarding overtopping, a widening of the landward slope would be advisable, as it reduces the slope angle of the landward slope. With a smaller slope angle the velocity of the overtopping water, that rushes down the landward slope, is reduced and the risk of erosion is decreased. The dike widening enlarges the cross-section of the dike. Therefore, the overall stability of the dike is increased. An example can be seen in Figure 31.

Figure 31: Dike Widening, www.ice-holland.com

7.2.1.3 Outer Berm

An outer berm is an earthen structure placed on the seaward side of the dike as can be seen in Figure 32. It reduces wave-run up and overtopping discharge which leads to "… a lower required crest level for the dike or embankment." (PULLEN AND VAN DER MEER, 2016). A side-effect is that it stabilises the dike against slip failure of the outer slope and seepage. It also provides a track for dike inspection (Ciria, 2013). Regarding its design, (PULLEN AND VAN DER MEER, 2016) conclude that it is most effective when lying on the still water line.

Figure 32: Berm in the area around Westkapelle, Nikos Sigalas

7.2.1.4 Surface Protection

Surface protection takes a variety of forms but can be divided up into soft and hard engineering solutions. For the soft solutions additional layers of sand, gravel, clay, grass or geotextiles are placed onto the surface. For the hard solutions armour stones, concrete blocks, tied block mattresses or a continuous paving made from concrete or asphalt are used. The main purpose is "… to reduce the threats of erosion and scour on levee projects." (Ciria, 2013) by guaranteeing stability of the dikes surface. The individual situation determines which type of surface protection is favourable. Grass is inherently linked to the concept of resilience, as it repairs itself. If the strength added through a strong grass cover is not sufficient, turf reinforcement mattresses can be used to strengthen the surface further. The two different types can be seen in Figure 33 and Figure 34.

Figure 33: Hard Surface Protection, www.tudelft.nl

Figure 34: Soft Surface Protection, www.lakeyinc.com

7.2.1.5 Flood Walls

Flood Walls are vertical barriers, typically made from concrete, mortared stone or brick. The purpose of the wall is to keep the water level behind it below a predefined damage criterion. There are two common types of design, the gravity and the cantilever design. It is possible to drive sheeting into the soil below the wall to minimise seepage. The walls "… are used when space does not allow increasing the levee cross-section, the right of way is not available, or the levee foundation cannot support the weight of the additional earth fill." (Ciria, 2013). Its most important benefit is that it does not require a lot of space but provides effective safety against overtopping. An example can be seen in Figure 35.

Figure 35: Flood Wall, www.odebrechtusa.com

7.2.2 Resilience Options

7.2.2.1 Raising Structures

In the case of a breach or severe overtopping of the dike, raising the existing structures above the Base Flood Elevation (BFE) would help avoid damage to the buildings in the flood prone areas. There are multiple methods to raising buildings, but for coastal flooding which is generally more severe compared to riverine due to the addition of waves, it is recommended to add freeboard. This means to elevate the building higher than the minimum BFE to a level called the Flood Protection Elevation (FPE). If the BFE is known, it is suggested to elevate the buildings 30 to 60cm above the BFE, whereas if the BFE is not accurate, it is suggested to raise the building 1m above the BFE (Hill, 2014).

The most common methods of raising buildings are Elevation on Fill, Extending Foundation Walls, Abandoning the Lower Enclosed Areas, and Elevating on an Open Foundation. Elevation on Fill and Extending Foundation Walls are not recommended for coastal zones since they involve solid construction under the building which would impede flood flow and lead to further damages. Abandoning the Lower Enclosed Areas means "removal of non-load-bearing walls from the lowest floor level of a multi-storey building in order to permit flood waters to flow through relatively unimpeded. The implication of this is that the lower floor level is 'abandoned' as a habitable space and the upper floor level becomes the new lowest floor level" (Hill, 2014). Elevation on an Open Foundation refers to "the jacking up of a building and the replacement of foundation walls with posts, piles or piers. In the first option, wood, concrete or steel posts are installed with new foundations in pre-dug holes. Piles driven into the ground may be stronger than posts in coastal and high velocity zones and may need cross-bracing, but steel piles will be subject to rust corrosion in coastal areas. Piers made from concrete blocks, poured concrete or brick are only suitable for areas with low flood velocity and minimal erosive force, and are thus not suitable for coastal areas" (Hill, 2014). An example of an Open Foundation can be seen in Figure 36.

Figure 36: Open foundation, pressherald.com

Overall there are multiple methods to raise existing structures to reduce flood damage. The appropriate method would have to be chosen for each building to optimize the damage reduction, and this method should only be used when the expected flooding is severe.

7.2.2.2 Floodproofing

If raising structures is not possible or not the right solution for a certain building, another option to reduce or eliminate the potential of flood damage is floodproofing the building. Floodproofing consists of methods called wet floodproofing, dry floodproofing, barrier measures and interior modifications. Wet floodproofing, as the name suggests, includes measures that work with the flood water to mitigate damages. It includes altering buildings to allow floodwaters to enter and exit without causing major damages. They are generally used in areas of the building that aren't used as living space, for example in a home they could be used in a crawlspace, basement or garage. Other than reducing flow forces on the building, allowing water in the building reduces the pressure difference inside and outside of the walls which helps prevent walls from caving in. This method also includes raising the building utility systems to protect them from damage or loss of function during a flood.

Dry floodproofing consists of sealing the building to prevent floodwaters from entering. Some examples are using waterproof coatings or coverings, installing waterproof shields and devices that prevent sewer and drain backup. An example can be seen in Figure 37.

Figure 37: Dry Floodproofing with waterproof shield, resilientdesign.org

Barrier measures include floodwalls and levees built around a building. These may be effective at controlling floodwaters up to a certain depth, but also require appropriate space to be constructed.

Lastly, interior modifications include making changes to an existing building to reduce flood damages. Examples are filling in the basement if it is located beneath the BFE, abandoning the lowest floor, and elevating the lowest interior floor. These modifications lead to minimizing damages by adapting the building to the flood environment, essentially relocating the living area to be above the BFD. These modifications lead to a loss of square footage of the property and are high in cost (FEMA, 2015).

7.2.2.3 Sand Nourishments

A natural remedy to aid flood defence and therefore increase the resilience of the system would be sand nourishments. In general, this means importing large quantities of sand to the coast to widen the beach seaward of the coastal dike. The coast is a dynamic system and fluctuates with the changing forces. “Sandy beaches for instance, respond to increased wave activity by flattening their profiles as the beach face becomes saturated with water and the net cross-shore transport of sediment becomes more biased towards offshore (Dean, 1991). Sand dragged offshore may form or augment sequences of submerged bars. This creates a system in which the bigger waves break more aggressively in the shallow water depths over these bars. A wider, dissipative surf zone develops, reducing the wave energy incident on the shoreline.” (Hanley et al., 2014). This would help protect the coastal dike by reducing wave energy before the wave reach the dike. This may be thought of as a ‘soft’ engineering solution compared to the more classic ‘hard’ solutions such as breakwaters and sea walls.

7.2.2.4 Multifunctionality of Buildings

An important option to consider for resilience includes multifunctional buildings. With climate change effects increasing, the understanding that Man cannot fully dominate the nature is becoming more apparent. Resilient systems must include urban modifications as well as flood defence structures. Urban resilience is emerging as a new approach in the flood risk management field (Cutter et al., 2008) which is leading change to current infrastructure and societal planning. A great example of multifunctional buildings comes from Rotterdam. Aiming to learn to live with the water instead of fight to keep it out, Rotterdam has invested in many urban resilient systems to reduce flood damage in the case of an extreme event. A major example is the multifunctional car parks. The car park near the Museumpark is equipped with an underground water storage facility touted to become the largest water storage facility in the Netherlands which can hold 10,000 m^3 of water which will be held here until it can be pumped out into the sewers and dealt with in a usual manner. The city has also constructed additional water plazas which are areas that will fill up in a controlled manner during heavy rainfall preventing the surrounding streets from flooding, as shown in Figure 38. When not being used for water storage they are open public spaces (Mackenzie, 2010).

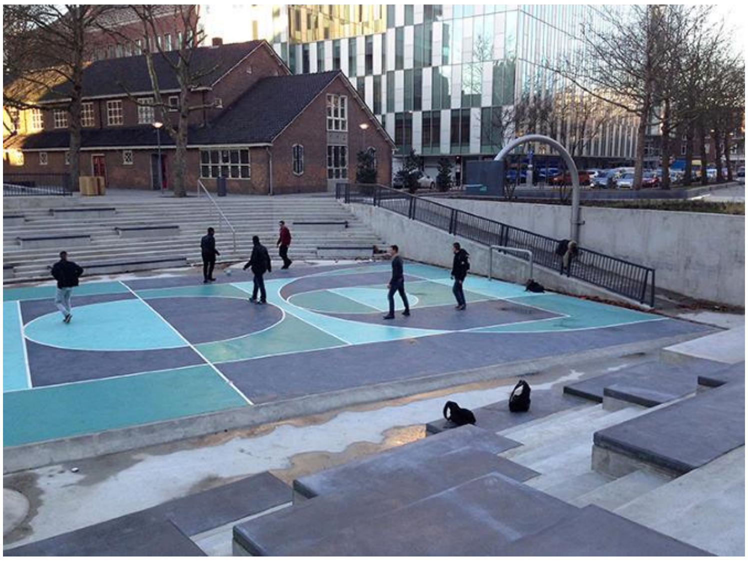


Figure 38: Multifunctional sports grounds, pinterest.com

7.2.2.5 Forelands, Breakwaters and living Shorelines

Living breakwaters are another option to increase the resilience of the flood defense system. These include engineered breakwaters or natural structures placed offshore to break waves and reduce the impact of wave run-up. They can be hard structures made of rock or stone or they can be soft and multifunctional such as artificial islands, reefs and floating facilities. Breakwaters are most effective in shallow waters and areas that suffer from large wave forces. An example is show in Figure 39.

Figure 39: Living shorelines, southernenvironment.org

On the other hand, living shorelines are a form of coastal protection based on the use of natural vegetation and soil to the foreshore to reduce wave and surge impact. They can be a combination of hard structures such as bulkheads and revetments with natural soils added to provide wave attenuation and reduce erosion. Living shorelines can be a good option when raising the coastal defence or adding a floodwall is difficult or expensive. They can also increase the ecological and recreational value of the area (Veelen, 2016).

7.2.2.6 Diversion canals

Increased wave overtopping rates results in wave transmission on the leeward side of the dike. If discharges are high a surge can form by flow accumulation resulting in difference of water depths. After reaching a critical level, if surge is not taken care of it starts propagating resulting in flooding. To avoid this surge propagation and flow accumulation diversion canals can be used starting from the heel of the dike towards the landward side. In case of a flood event, these canals would divert excessive flows and act as a source of damping in the system. Also according to RCI (2013) water management in the polder city improved the water quality and the water level management, and at the same time made the city more attractive.

7.2.2.7 Salt Marshes

Salt marshes are important coastal ecosystems. They are considered as one of the nature-based flood defences (Leonardi et al., 2017). Salt marches are seen as one of the soft sea defence engineering solution (Möller, Spencer, French, Leggett, & Dixon, 1999). This is due to the fact that salt marshes act as a buffer against the impact of storms (Leonardi et al., 2017). According to Van Loon-Steensma (2015), salt marshes form a vegetated transition zone between land and water where they break incoming waves, reduce wavelength and velocity, eventually dissipate wave energy via friction with vegetation and the marsh surface. (Leonardi et al., 2017). Salt marches are one of the soft sea defence engineering solutions. This is because salt marshes act as a buffer against the impact of storms. According to Van Loon-Steensma (2015), salt marshes form a vegetated transition zone between land and water where they break incoming waves, reduce wavelength and velocity, eventually dissipate wave energy via friction with vegetation and the marsh surface.

Figure 40: Artificially created salt marshes (Temmerman, Meire et al. 2013)

Numerical model experiments of Möller, Spencer et al. (1999) concluded that salt marshes reduced the wave height (average 60·96%) on approximate average of four times more effectively flat sand (average 15·29%). Delta programme suggested dike designs, including vegetated forelands (e.g. salt marshes), as one of the resilient and promising flood protection strategy for the Wadden region (Waddengebied, 2012). In the long run, coastal ecosystem flood defences, such as salt marshes, can be more cost effective compared to hard conventional engineering defence. According to a study conducted in the United Kingdom, 25 years of tidal marsh restoration on reclaimed land proved to be economically more beneficial than maintaining dykes (Temmerman et al., 2013). However, the implementation of ecosystem-based flood defences is complex. Nature-based flood defences require more space more than conventional structures systems (Temmerman et al., 2013), (Waddengebied, 2012). In the long run, coastal ecosystem flood defences, such as salt marshes, can be more cost effective compared to hard conventional engineering defence. According to a study conducted in the United Kingdom, 25 years of tidal marsh restoration on reclaimed land proved to be economically more beneficial than maintaining dykes. However, the implementation of ecosystem-based flood defences is complex. Nature-based flood defences require more space more than conventional structures systems.

7.2.2.8 Resilient Transport

Another critical option to consider for resilience is ensuring the resilience of infrastructure networks. Infrastructure networks are often considered to be the backbone of cities. Ensuring their resilience has become a vital aspect of governing and managing an economically-viable and livable city (Pregnolato, Ford, Wilkinson, & Dawson, 2017). According to Lee, Wong et al. (2010), most transportation infrastructure, particularly roadways and bridges, were designed to last 50 years or longer. Moreover, many were constructed without today's knowledge of climate change and the accelerated projections in sea level rise (Lee, Wong, & Woo, 2010). Studies have shown that roads are among the first cause of deaths in cities during flooding, due to vehicles being driven through flooded roadways. Therefore, resilient systems must include urban modifications as well as flood defence structures. Urban resilience is emerging as a new approach in the flood risk management field which is leading change to current infrastructure and societal planning. (Pregnolato, Ford, Wilkinson, & Dawson, 2017).

Serre, Barroca et al. (2018) came up with a conceptual DS3 model to study the resilience of urban networks. The model focuses on three specific capacity measures to analyze the urban network. These are the resistance capacity, absorption capacity, and recovery capacity. In the study, resilient transportation refers to creating multiple modes of transport or raising transport higher than the reference water level to reduce damages if a flood were to occur. These measures are reliable strategies for ensuring resilient transportation.

7.2.2.9 Critical Infrastructure Resilience

Critical infrastructure in a flood-prone area is more than just levees, bridges, and canals. It is a complex series of interdependent built and natural systems that keep the coastal city safe, productive, and healthy (Katsman et al., 2011). Part of a resilient approach to infrastructure is an accurate understanding of the full geographic and functional breadth of these systems and the connection between the built urban environment and the managed landscapes that surround it (Ouyang, 2014). Extreme events like storms might have cascading effects on the city's critical infrastructure systems (Ouyang, 2014). When one system is compromised, it negatively impacts the function of other critical systems (Ouyang, 2014). For example, after Hurricane Katrina, the supply of crude oil and refined petroleum products was interrupted because of loss of electric power at the pumping stations for three major transmission pipelines (Neal, 2014). Because of the loss of power, about 1.4 million barrels of crude oil were lost per day (Neal, 2014). This is an example of how the failure to understand the dynamics of these relationships especially in chaotic environments can lead to misuse of resources, personnel, limited supplies and relief efforts (Neal, 2014). Interdependency is a bidirectional relationship where the state of one system is directly influenced by the state of the other (Neal, 2014). Figure 41 highlights the layers of critical infrastructure.

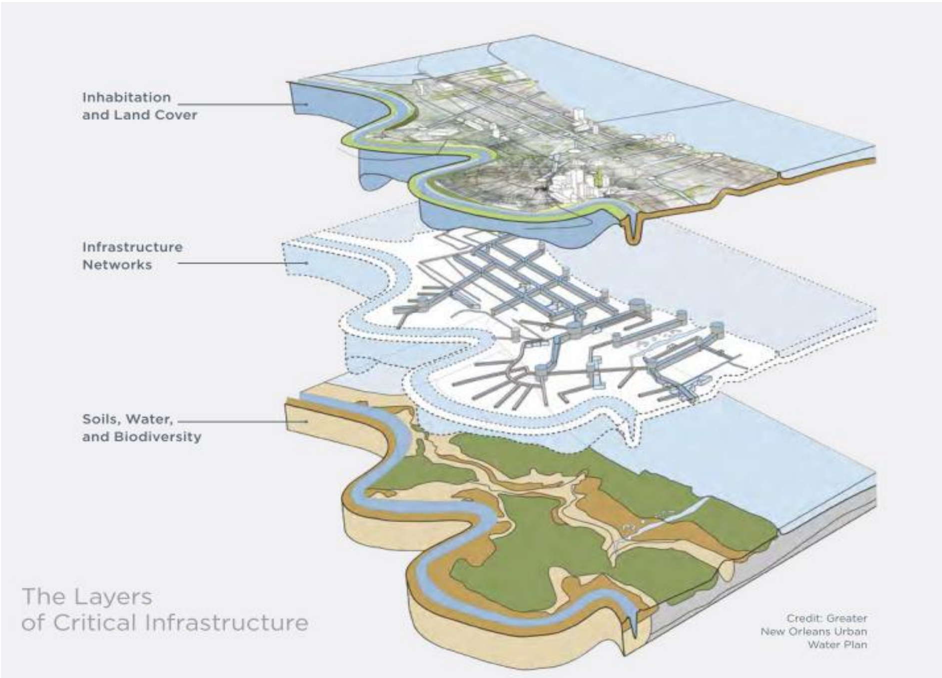


Figure 41: Layers of Critical Infrastructure, Great New Orleans Urban Water Plan

Therefore, the concept of Critical Infrastructure Resilience (CIR) is one of the mitigating strategies against flooding. Theoretically, CIR concept is to divide the low laying prone to flood cities into compartments that function independently to provide protection against floods and storm water. Each compartment comprises a physically discrete flood-protection zone that can be isolated from flooding in adjacent zones. At the same time, each compartment presents opportunities for integrated social and community planning. The compartments work in unison to protect and enhance the city, yet each compartment is designed to stand on its own (Bianchini, 2015).

7.2.2.10 Wetlands and Groundwater replenishment

Wetlands function as natural sponges that trap and slowly release surface water, rain, snowmelt, groundwater, and flood waters and distribute these waters more slowly over the floodplain, thereby lowering flood heights and dissipating storm surge (Cahoon et al., 2006; Costanza et al., 2008; Herbert et al., 2015). An example can be seen in Figure 42.

Figure 42: Coastal wetlands, commons.wikimedia.org

Coastal wetlands function as valuable, self-maintaining "horizontal levees" for storm protection. They provide a host of other ecosystem services that vertical levees do not (Cahoon et al., 2006; Costanza et al., 2008). Their restoration and preservation are an extremely cost-effective strategy for society (Costanza et al., 2008). According to Costanza, Pérez-Maqueo et al., (2008), coastal wetlands reduce the damaging effects of hurricanes on coastal communities.

However, coastal wetlands are vulnerable to climate change and mean sea level rise and can be affected by erosion, inundation, and saltwater intrusion. Sea level rise and land subsidence could greatly increase the risks of salinization of the coastal wetlands (van Dijk et al., 2015). Literature emphasizes that climate change and hydrological cycle alteration could lead to a further increase in the severity of wetland salination which in turn would result in a significant change in the wetland ecosystem function (Erwin, 2009). As a result, the coastal wetland resilience to sea level rise would be minimized. Although, effects of climate change are inevitable, various mitigation strategies have been suggested to reduce and prevent future negative affect of climate change. According to the Climate adaptation report (2004, the expected increase of salinity (seepage of salt water) along the Dutch coastal zones is mainly due to the sea level rise. The report suggested few adaptation strategies to the salinization of agricultural land (Nillesen & Van Ierland, 2006). These strategies could be applied to minimize the same effect on the coastal wetlands (Nillesen & Van Ierland, 2006). The adaptation strategies are:

- improving the efficiency of freshwater use in areas subject to salinization.
- the growing of halophyte cultures
- the growing of macro- and micro-algae
- the growing of bait for fish in saltwater basins on the land and fish
- use or design of salt tolerant crops
- changing land use.

7.2.3 Policy Options

7.2.3.1 Risk Zoning and Societal Planning

Coming up with a resistant system does not serve the purpose completely; the system must be adaptable to make it robust (RCI, 2013). Before coming up with adaptable strategies to counter any extreme event, one can start to analyze risk variability in the area and tendency of the system for risk accumulation or diversification. Practically, this would result in risk maps identifying areas with maximum and minimum risk. It could also be inferred from the analysis how certain measures can increase or decrease the risk of the system.

Risk zoning can be used for the basis of societal planning. Societal planning would include improving urban dynamics through land use plans, urban development plans, scattered residencies to counter population densification, building codes and many more. Such planning and policy measures would decrease vulnerability and eventually the risks.

Resilience indexing of areas in terms of ratio of response time to duration of perturbation is also part of this option. Indexing could be done through mock drills of, say, fire brigade or even running evacuations models. In flood dormant times, when there is no flooding, this resilience option would serve as a starting point for flood mitigation.

7.2.3.2 Awareness and preparedness

Resilience is not all about prevention, but it also accompanies preparedness. Beside structural measures one way to increase preparedness is by creating awareness among the stakeholders. The stakeholders who must be the most prepared are the ones who can get affected the quickest and the ones who can get affected the most.

Raising public awareness could be done through indirect education such as seminars, trainings, mock drills for homeowners, neighborhood organizations, and key professionals. etc. Awareness topics range from education about localized urban flood to immediate steps (evacuation measures) to be taken after hearing news about the flood. (City of New Orleans, 2015)

For resilience enhancement additional objectives like preparedness must be undertaken. Besides usual measures like creating emergency response centers, enabling every household to access of a basic flood response toolkit and having efficient evacuation plans. Also, adding response curves of areas, meaning identifying areas which will take the least time to restore its function to pre-flood condition will help add resilience to the system. It would also help to increase preparedness and prevent heavy impacts on the system from which recovery is extremely difficult without outside help (Gersonius et al., 2016).

7.2.3.3 Financial management

Finances play an important role in determining which options to implement, and what are the tradeoffs and opportunity costs. On average a Dutch citizen pays 200-300€ annually for flood defenses, varying due to the area and its flood risk. However, this money is mainly used for maintenance and construction of flood defence structures. Financial management includes efficient allocation of limited resources in terms of finances before and after the flood event like insurances or government compensations.

Such measure is a valuable addition for combatting flooding and is a source of positive externality for government and associated institutions. Proper fiscal management would not only help in crisis management, for example through insurance options, but will also mitigate the consequences of an event. In other words, it would prevent a low scale flood event from turning into a bigger catastrophe by reducing recovery time and eventually improving robustness.

7.2.3.4 Shared Initiatives

Every stakeholder has its own wishes, rights, and responsibilities which might overlap in some cases. In case it does not, efforts should be made for stakeholder consultations. For each structural or non-structural measure, either involving physical construction or policies and laws, every stakeholder should be on the same page. Sharing of knowledge, agreement, and practice to reduce risks and impacts collectively would not only be effective but also improve 'social climate'. (UNISDR, 2017)

Stakeholder consultations are not only a source of discussing solutions from different points of view but also can largely help in execution of planned solutions. Shared initiatives have an indirect relation with every other option in terms of ease of implementation. Once every stakeholder is on board, most of the execution and implementation issues are already solved and implementation becomes easy. On the contrary, issues like communication gaps, lack of shared responsibilities, lack of ownership etc. is a common sight in projects involving community. Consultations like Rijkwaterstaat meeting local farmers every 3 months to brief about flood defense works going on and initiatives such as community and capacity building schemes, collective gardens, etc. would add to robustness of the system.

The following chapter explains how the single options were combined to form design approaches.

Chapter **8**
Design Approaches

8 Design Approaches

Five different design approaches were formed and analyzed carefully. The first section of this chapter introduces them and explains their approach on risk reduction. One of the approaches was selected to be followed up and designed in more detail in the design phase. The second section describes the reasoning behind the selection. The third section highlights costing considerations, that were considered during the analysis of the design approaches. The importance of costing will increase in the next phase where the design will be subject to economic optimization.

8.1 Approaches

Every approach is introduced with a description of its main focus. The specific weighting is introduced in form of a table, where the bottom row shows the specific weighting percentage of every criterium. The row above gives the order of importance. The results from the MCA are compared to the expectations from the respective focuses.

8.1.1 Resistance Design Approach

Table 13: Weightings in resistance approach

Resistance		Resilience			Aesthetics/Environment			Economy						
Prone to breach/catastrophi	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
1	1	4	2	3	11	8	9	5	6	12	13	10	14	7
1	1	0,65	0,85	0,81	0,18	0,21	0,21	0,61	0,61	0,18	0,09	0,20	0,03	0,43

The resistance design approach reflects on the system's ability to withstand a disturbance. The most important criteria for the resistance design approach were Prone to breach/catastrophic failure, Resistance to overtopping, Resilience gain, Expected amount of overtopping, Prone to damages. Cost also played an important role. The highest weighted criteria for this design were prone to breach/catastrophic failure and resistance to overtopping. The weighting was done in this way, because the focus of the resistance design approach is, on the one hand, to decrease the failure probability of a dike under breaching. On the other hand, the level of damages, that overtopping causes, must be considered as well. This is done with the criterium Resistance to overtopping. The resilience gain, expected amount of overtopping and prone to damages, which are purely resilience criteria, were the second priority of the weighting. With the climate change uncertainties and sea level rise, a resistance system backed with a resilience one is one of the utmost solution in reducing the risk of an extreme event during breaching and failure of resistance system. Cost was also given a relatively high weighting, as resistance mostly requires constructive measures with a high cost.

The top 5 design options that match this criteria weighting were surface protection, flood wall, dike heightening, living breakwaters, and dike widening. Reinforcing dikes can increase their stability and resistance against dike breaching. For example, a surface protection such as grass or concrete could be added to the dike to reduce erosion rates and increase the friction of the water on the dike which will ultimately contribute to make a dike less susceptible to erosion induced by floodwaters and overtopping. Floodwalls are normally considered when high flow velocities may erode a levee/dike. However, with increasing the certainty of sea level rising, both dike and floodwall can be a parallel system where wave overtopping can be minimized enormously. An alternative for floodwall system would be dike heightening and widening. This alterative system would be an overtopping resistant one and can be, at the same time, multifunctional (for example recreation or transport). These options could all be combined to create a robust flood protection system.

8.1.2 Resilience Design Approach

Table 14: Weightings in resilience approach

Resistance		Resilience			Aesthetics/Environment			Economy						
Prone to breach/catastrophi	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
15	14	3	1	2	13	6	12	9	10	5	4	7	11	8
0,10	0,10	0,86	1,00	0,95	0,21	0,51	0,26	0,37	0,37	0,68	0,68	0,49	0,26	0,46

The resilience design approach deals with those options, ranked and weighted high, which help the system to be more responsive in case of failure i.e. excess overtopping rates. System responsiveness can be described in various dimensions, for instance it could be defined in terms of recovery time, or increased serviceability limit states or decreased vulnerability to damages etc. Keeping in mind the aforementioned description of resilience, the ranking of criteria is as follows: resilience gain, expected amount of overtopping, prone to damages, impacts on local businesses, and flexibility and adaptability to future. As a result, top five options from the MCA in the order of decreasing importance are: surface protection, awareness and preparedness, living breakwaters (reefs, oyster/mussel reefs), shared initiatives, and multifunctionality of buildings.

From the range of available options, one might argue that flood walls give more resilience gain to the system than surface protection but for this analysis, the flood wall is treated as a pure resistance option. The scoring and ranking is done keeping in mind the scenario, that the flood has already gone past the dike. With this definition of the resilience design approach, surface protection and living breakwater options in the MCA results are inconsistent.

8.1.3 Stakeholder Design Approach

Table 15: Weightings in stakeholder approach

Resistance		Resilience			Aesthetics/Environment			Economy						
Prone to breach/catastrophi	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
1	14	9	2	3	6	8	4	10	11	13	5	7	12	15
1,00	0,06	0,30	1,00	0,90	0,50	0,35	0,63	0,30	0,30	0,10	0,63	0,50	0,15	0,06

This stakeholder design approach focuses on the desires of the previously determined most important stakeholder, the local inhabitants. The highest weighted criteria for this design was therefore the prone to breach/catastrophic failure since it is the lives and property of the local inhabitants at risk if a breach were to occur. The remaining top five criteria for this design are (in order) resilience gain, expected amount of overtopping, cultural modification, and impacts on local business. The resilience gain and expected amount of overtopping add to the fact that the local inhabitants are most concerned with their lives and therefore reducing the risk of an extreme event causing extensive damage and putting them in danger is of utmost importance. The cultural modification and impacts on local business are the fourth and fifth most important criteria since after their lives, the local inhabitants would like to see that their ways of living and economy are not heavily impaired or in danger. The least important criteria for this design is the ease of implementation since the inhabitants will not care how difficult the construction process is, they simply want the best solution to create a safe living environment.

From the weighted multicriteria analysis, the top five design options are dike heightening, awareness and preparedness, living breakwaters, dike widening and surface protection. These options can all be combined to create a robust flood protection design. On the coast, at the locations of the highest waves during a storm, a natural breakwater could be placed just past the surf zone to cause the waves to break offshore and reduce wave forces on the coastal dike. The dike itself could then be heightened and widened (provided there is enough space) to cope with the climate change predictions of rising sea levels. Rising and widening the dike would create a stronger and more stable dike while at the same time reducing wave overtopping. On top of this, a surface protection such as grass could be added to the dike to reduce erosion rates and increase the friction of the water on the dike and in turn reduce overtopping. Lastly, apart from physical flood protection measures, a policy could be put in place to increase the awareness and preparedness of the inhabitants. This would include evacuation plans that provide safe routes for all inhabitants to high elevation points or nearby cities in a different dike ring. The policy would also inform the inhabitants of the current situation and all possible flood events and outcomes. Lastly, the policy would include a quick and effective warning system to alert the inhabitants that a flood event is possibly coming so that the citizens can act with sufficient time.

8.1.4 Cost Design Approach

Table 16: Weightings in cost approach

Resistance		Resilience			Aesthetics/Environment			Economy						
Prone to breach/catastrophi	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
11	12	8	9	10	15	13	14	1	2	6	5	3	7	4
0,17	0,16	0,26	0,23	0,21	0,06	0,10	0,08	1,00	1,00	0,32	0,36	0,80	0,26	0,40

The cost design approach focuses on determining the most economically efficient design. Based on the four design group criteria, the multi-criteria analysis ranked and weighted them from the most relevant to costing, Economy, to the least relevant to costing, Aesthetics/Environment criteria. The optimal standards followed in the cost analysis approach was based on basic research and personal judgment with existing flood protection standards. From the MCA, it was found that the most important criteria for the cost design approach were cost, maintenance, land use, ease of implementation, and impact on local business. The options that come out on top are Surface Protection, Shared initiatives, Awareness and preparedness, Financial management and Multifunctionality of buildings. This is because, they were less costly than the other flood protection measures. For instance, Zethof and Kolen (2015) estimated the costs for organizing a disaster exercise and a course to be approximately €5 million. Certain resistance measures such as surface protection and multifunctionality of buildings are estimated to be less expensive than other resistance measures such as flood wall or dike heightening. Interestingly, the top three design options have frequently occurred in most of the previous design approaches. This gives an indication of their importance in the optimum design approach.

8.1.5 Good-For-All Design Approach

The good-for-all design approach is analyzed as an option to see if there is an optimum solution, with regard to all the considered aspects. This optimum solution would be most resistant, resilient, cost effective and fulfill the interests of the most important stakeholder i.e. local inhabitants to the highest degree. Two sets of options are obtained by using two slightly different ways to come up with the top options for this design approach. Firstly, a conventional MCA is conducted with the weightings as given in Table 17.

This gives the following top six options are: surface protection, awareness and preparedness, dike heightening, living breakwaters (reefs, oyster/mussel reefs), shared initiatives, and dike widening. Secondly, the top two and the top three from all previous design approaches were compared. The more frequently an option comes up, the better it was deemed for the good-for-all approach.

Table 17: Weightings in good-for-all approach

Resistance		Resilience			Aesthetics/Environment			Economy						
Prone to breach/catastrophi	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
1	4	2	1	2	5	4	3	1	2	6	3	4	5	5
1.00	0.40	0.80	1.00	0.80	0.20	0.40	0.60	1.00	0.80	0.10	0.60	0.40	0.20	0.20

The two sets of options do not match entirely two irregularities appear. Firstly, when considering the top two options of all the design approaches living breakwaters does not appear. Secondly, when considering the top three options dike heightening has more importance than awareness and preparedness which seems more logical than the results from the MCA.

Anyhow, the results of these two different ways of obtaining the top design options were well in line. The combination of results from the previous design approaches gave options that will allow a better robustness maximization. Therefore, the following options were chosen for the good-for-all-design approach: surface protection, dike heightening, awareness and preparedness, living breakwaters (reefs, oyster/mussel reefs), and shared initiatives.

8.2 Chosen Approach

After completing the Multi Criteria Analysis (MCA) the team reviewed the results to compare the different design approaches. The design approaches were created with the combination of the top five appropriate options for each design focus. What was noticed is that even with the different weightings of criteria for the different design approaches, the selected design options only differed slightly. Table 18 below shows the options (rows) that were selected for the five different design approaches (columns).

Table 18: Selected design options for the design approaches

Design Approaches				
Resistance	**Stakeholder**	**Good for all**	**Resilience**	**Cost**
Dike Heightening	Dike Heightening	Dike Heightening	multifunctionality of buildings	multifunctionality of buildings
Dike Widening	Dike Widening	Shared initiatives	Shared initiatives	Shared initiatives
Surface Protection	Surface Protection	Surface Protection	Surface Protection	Surface Protection
living breakwaters	living breakwaters	living breakwaters	living breakwaters	Financial management
Awareness and preparedness	Awareness and preparedness	Awareness and preparedness	Awareness and preparedness	Awareness and preparedness

The same colors represent the same design option. As can be seen, the surface protection and awareness and preparedness options are part of every design approach. Also, the options dike heightening, shared initiatives and living breakwaters were a part of at least 3 of the design approaches. The only selected design option that appeared once was financial management, all other selected options appear in 2 or more of the approaches.

Only 8 of the 19 different design options were selected. This suggests that these options which were selected repeatedly heavily outweigh others. To move forward with one design approach, the original procedure was to do a cost benefit analysis of each design approach and from that determine which approach was the best. Since the approaches are quite similar it was decided that the cost benefit analysis wouldn't have as great of an impact as previously expected.

Observing the table above, what can be noticed is that the good-for-all approach contains options that appear the most frequently across the different approaches. The options that it does not include are the 3 that appear the least frequently. It makes sense that the good-for-all approach contains the options that are most common in the other approaches since it is intended to account for all aspects of the design. Due to its ability to meet the needs of each design, and that it contains the most commonly selected options across all approaches, the good-for-all design has been selected as the design approach to move forward with.

8.3 Costing

8.3.1 Cost figures from literature

The cost consideration is important for all design approaches for practical purposes. After literature review from Kok et al. (2008), AFPM (2006), Lenk et al. (2017), Bos (2008) and Hillen et al. (2010) the table below is produced. It is to be noted that the policy options, i.e. financial management, shared initiatives, awareness and preparedness, are considered negligible as compared to the costs to these options and are therefore not included in Table 19.

Table 19: Cost of design options

Design Option	Unit	Source/year	Base year	Base year cost	Cost in 2018
Dike heightening (rural)	M€/km/m	(Committee, 2008)	2008	10	11.54
Dike heightening (urban)	M€/km/m	(Committee, 2008)	2008	20	23.09
Maintenance	M€/km/year	(Waterkeringen, 2006)	2006	0.1	0.12
Surface protection	€/m2*	(Lenk, Rybski, Heidrich, Dawson, & Kropp, 2017)	2017	26	26.35
Living breakwaters	M€**	(Berger, 2017)	2016	61.75	63.45
Nourishments	€/m3	(Committee, 2008)	2008	3	3.46
Vertical Seawall	M€/km	(Bos, 2008)	2008	4	4.62
Storm Surge barriers	M€/km	(Marten M Hillen et al., 2010)	2010	2	2.25
***40 CAD/m2 converted at 1 CAD=0.64€ (2018)**					
****76 M$ USD converted at 1 USD=0.81€ (2018)**					

8.3.2 Inflation

Since the costing was from various sources which have different base years, the cost of the option today was to be determined. Yearly data for average consumer price index (CPI) was acquired from tradingeconomics.com and has been reproduced in Appendix C - Costing**.** Compound varying interest rate formula, as written below, has been applied to get the last column of the Table 19.

$$P_n = P(1+i)^n$$

where; P_n is total inflated estimated Cost, *P* is base estimated cost, *i* is inflation rate and *n* is the difference between base year and selected year. A plot of the inflation rate since 2008 can be seen in Figure 43.

Figure 43: Netherlands inflation rate, tradingeconomics.com

Chapter 9 Overtopping Discharge

9 Overtopping Discharges

In this chapter a thorough analysis of allowable overtopping values is conducted. In times of climate change it may be necessary to question the values that are given in standards. Different points of view are adapted to determine reasonable allowable overtopping values for lower bound and higher bound climate change scenarios.

9.1 Designing with Climate Change

As outlined in chapter 0, the loading at the project location is subject to significant levels of uncertainty. This is due to the long design life (until 2100) and the uncertainties of climate change, as given in chapter 0. Even though substantial research efforts have been undertaken, the spans of climate change are large, e.g. varying between 0.65 m and 1.3 m of SLR. In order to design constructive measures, the team had to assume deterministic loading conditions. Nevertheless, uncertainty about those deterministic values had to be taken into account as well. Therefore, the team developed a design philosophy that harmonizes both these contradicting inputs.

Robustness, as given in (Mens, 2015), requires the system to withstand certain levels of loading (resistance) while the damages from higher levels of loading can be controlled via resilience. Therefore, no water is permitted to enter the system under normal conditions. Under extreme conditions, certain amounts of water are permitted to enter the system. However, the entering amount of water is limited so that catastrophic failure of the system cannot ensue. This approach is at the core of the team's design philosophy.

Climate change can be expected to significantly change water levels through SLR and subsidence, and wave heights, through an increase of storm severity. For wave heights the highest projected value, an increase of 18%, was adopted. Due to scarcity of research projects, regarding the wave height increase, the team opts for this conservative estimate. The change in water levels was taken into account by adopting a lower bound of 0.65 m SLR as the design conditions to be withstood via resistance. It was ensured that higher SLR-scenarios could be controlled via resilience of the design.

The driving factor in the design process was the definition of "withstanding" the design conditions. From the final section of chapter 4.3 it became clear that the choice of the allowable overtopping values leaves room for subjective judgements and is not straightforward. The team therefore decided to analyze the acceptable level of overtopping more profoundly, as presented in the next section.

The allowable overtopping value will be employed as the design guideline. This means that the recommended construction will ensure no violation of the predefined allowable overtopping value under lower bound conditions. The idea behind the lower bound allowable overtopping value is that the system does not need its resilience capacity and does not take any damage that would require repairs. A separate allowable overtopping value is chosen for the scenarios where the lower bound of climate change is exceeded. The idea behind this exceedance-scenario allowable overtopping value is that the system's resilience capacity is large enough to prohibit catastrophic damages.

9.2 Allowable overtopping values from different points of view

Four different points of view were identified to determine allowable overtopping values. In this section, each one will be explained separately. Final design values will then be derived from the combination of the different points of view. Figure 44 illustrates the process.

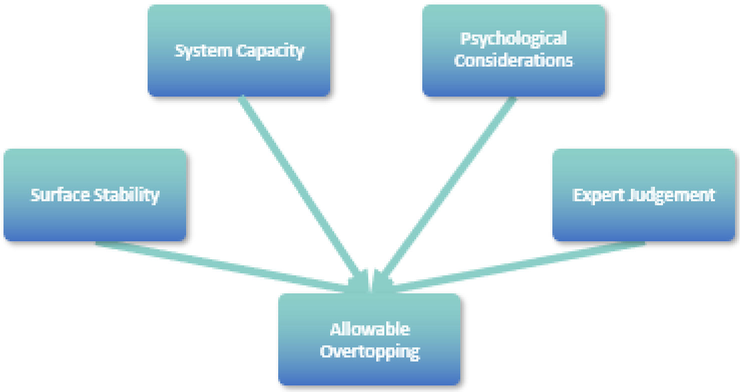


Figure 44: Flow chart for obtaining allowable overtopping values

As stated in chapter 4.3, research has cast doubt on the given values for maximum allowable overtopping with regard to stability of the dike surface. In accordance with the supervisor, the value of 10 l/m/s was chosen as representative for the local conditions. In light of van der Meer's (J. W. van der Meer et al., 2009b) research, where more than 30 l/m/s seemed acceptable for plain slopes with good grass cover, this seems conservative. However, experience shows that especially heterogeneities (e.g. stairs, other construction…) may lead to earlier onsets of failure and it is not only the stability of the dike surface that limits the allowable overtopping.

Additionally, the system itself (town of Westkapelle and surrounding area) can only take a certain amount of water before major damages are done. Agricultural areas, on the one hand, require only a small amount of water for the harvest to wither. The regional crops/plants in Westkapelle are prone to damage by even small discharge of salt inundation. The zero-salt damage is a wish for all farming stakeholders and is used to derive the salinity norms in the Netherlands (Bakel, Kselik, Roest, & Smit, 2009). Therefore, accepting some salinity damage during some extreme years would have a large effect on the salinity norms that are considered acceptable by the farming community (Bakel et al., 2009). However, in the coastal zones, the detrimental impact of the rising sea level on crops is not due to salt inundation but rather due to saline seepage (MNP, 2005). It will increase the brackishness of the groundwater and the surface water (Duan, 2016). The increased saline seepage to the upper groundwater will then harm existing agricultural crops with a low salt tolerance (Duan, 2016). In the Appendix an adaption strategy to salt intrusion is discussed briefly. Considering this argument, the amount of salt inundation due to overtopping is insignificant and hence negligible in calculating the overtopping threshold.

In urban areas such as Westkapelle, flooding and major damages are expected to occur when the water depth reaches values of 20 cm and above. This value originates from "… the experience that large-scale costs and damage only ensue when the water depths exceed 0.2m." (ENW, 2016). In order to obtain a threshold value for the allowable overtopping, the volume of water in the system is expressed in the following way:

$$V_{system} = V_{rain} + V_{overtopping} - V_{drainage} - V_{Exchange}$$

As volume is the product of area and water depth over the area and all the volumes refer to the area of the city of Westkapelle, the equation simplifies to:

$$h_{system} = h_{rain} + h_{overtopping} - h_{drainage} - h_{exchange}$$

Two assumptions are made to simplify this equation further. As the drainage system was designed for the well-known hydrological parameters of the region and will be updated with climate change, the terms regarding rain and drainage are assumed to cancel out. This corresponds to a practical situation where there is exactly so much rain as the drainage system can drain. Furthermore, the water exchange with other regions is neglected. When determining the area for flooding calculations, later in this section, an area surrounded by a dike will be assumed. Therefore, no substantial exchange with other regions can be expected on the short term. The equation now reads:

$$h_{system} = h_{overtopping}$$

The overtopping volume of water in the system can be expressed in the following two ways:

$$V_{overtopping} = q_{threshold} * t_{storm} * L_{dike} = A_{system} * h_{overtopping}$$

Rearranging yields:

$$h_{overtopping} = \frac{q_{threshold} * t_{storm} * L_{dike}}{A_{system}}$$

Inputting into the expression for the water volume in the system gives:

$$h_{system} = \frac{q_{threshold} * t_{storm} * L_{dike}}{A_{system}}$$

Rearranging yields:

$$q_{threshold} = \frac{h_{system} * A_{system}}{t_{storm} * L_{dike}}$$

As defined above, the critical height of water in the system is taken as 20 cm. The effect of the storm duration on the allowable overtopping was analyzed in Figure 45 by plotting the resulting values of allowable overtopping from storm durations ranging between 0 and 36 h to reach a depth of 20 cm in the system (whereas 36 h was chosen as the maximum because it corresponds to the duration of the 1953 flood disaster (De Kraker, 2006)). Physically, a high allowable overtopping value means that on average large amounts of overtopping can be allowed to enter the system per time step. Therefore, short storm durations will yield high allowable overtopping values – as the storm is short, large overtopping per time step is possible before the system capacity is reached. It becomes visible that the largest variations take place for storm durations between 0 and 10 h. From about 10 h onwards the variations are small. Therefore, a value of 12 h was chosen as the representative storm duration.

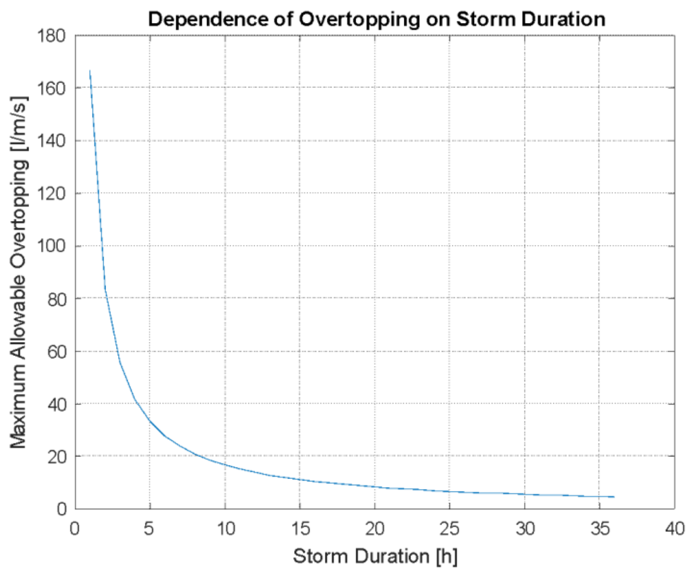


Figure 45: Maximum allowable overtopping from system point of view as function of storm duration

To obtain the area which is to be considered, flooding maps, refer to Figure 16, are employed. The maps show how water would distribute over dike ring 29 in case of a dike breach. It becomes visible that the area around Westkapelle is clearly separated from other parts of the dike ring. Additionally, water depths at the city of Westkapelle itself are shown to be lower than in the surrounding regions which indicates that Westkapelle is situated at a higher elevation so that water flows from Westkapelle to other regions. From these considerations the area as shown in Figure 46 is chosen for analyzing the amount of water that enters the system. The area amounts to roughly 37,5 km^2 and the dike length, where overtopping into the area will happen, amounts to 12,5 km. With these values the overtopping threshold from the system point of view is calculated as 13,9 l/m/s for a storm of duration 12 h.

Figure 46: Considered area for allowable overtopping from system point of view

Psychological impacts are another factor that need to be considered. Local inhabitants are living directly behind the dike and might not feel safe when the overtopping volumes are too large. Standards suggest that no one should be operating on the dike from a mean overtopping of 10 l/m/s. Anyhow, such an amount of overtopping still looks acceptable as the experiments of J. W. van der Meer (2011) show. 30 l/m/s, on the other hand, looks frightening and might cause panic among the local inhabitants.

Finally, expert judgment needs to be taken into account as well. To do so, the team arranged a meeting with hard coastal protection expert Bas Hofland (Hofland, 2018). The expert provided hands-on experience from engineering practice, in contrast to the mainly theoretical considerations of the team. In the meeting, the team proposed 20 l/m/s as the allowable overtopping value. This value was chosen because it was in line with psychological considerations, amounted to slightly more than the value that system capacity calculations recommended (where conservative assumptions were made) and because research showed that such high values may be possible with high-quality grass covers. The expert, in contrast, stressed that such a high value is not commonly encountered in practical engineering design. As overtopping can lead to erosion of the landward slope and finally a breach, designers are cautious not to allow too much of it and risk catastrophic failure of the dike. This is especially true in the situation of Westkapelle, where the city is located directly behind the dike. Additionally, the expert, in accordance with the supervisor, stressed the importance of transitions in the surface. Weak points like the connection between concrete stairs and the grass cover may lead to early onsets of failure. Table 20Table 1 summarizes the allowable overtopping values from the four different points of view.

Table 20: Allowable overtopping values from different points of view

<table>
<tr><th></th><th>Surface Stability</th><th>System Capacity</th><th>Psychological Considerations</th><th>Expert Judgment</th></tr>
<tr><td>Lower Value</td><td>10 l/m/s</td><td rowspan="2">14 l/m/s</td><td>10 l/m/s</td><td rowspan="2"><< 20 l/m/s</td></tr>
<tr><td>Higher Value</td><td>> 30 l/m/s</td><td>30 l/m/s</td></tr>
</table>

9.3 Finding lower bound and higher bound allowable overtopping values

The team aimed to determine two different allowable overtopping values. Firstly, the team required a lower bound allowable overtopping value. This value takes a lower estimate of climate change into account. Such a lower estimate is very likely to become true, so it was decided that the design needs to be entirely safe in these conditions. Therefore, the value of 5 l/m/s was chosen as the lower bound allowable overtopping. If the lower bound CC-scenario comes true, this value guarantees, besides people's safety, that no unnecessary damage is done to people's belongings via flooding (which only occurs at 14 l/m/s as determined in chapter 9.2). Furthermore, it contains a margin of safety for exceedance-scenarios. The proposed construction will mainly focus on guaranteeing the lower bound allowable overtopping.

If the lower bound prediction is exceeded, the lower bound allowable overtopping will also be exceeded. Adopting a low value for the lower bound allowable overtopping makes sure that very high levels of exceedance are required before substantial damage occurs. Additionally, the team put itself in the position of the local dike authorities, who carry the responsibility for people's lives. Local dike authorities tend to set their allowable overtopping to 1 l/m/s (TAW, 2003), and so the chosen value of 5l/m/s corresponds to a 400% increase. Convincing the local authorities of this increase seems possible, regarding the overall design philosophy. Convincing them of larger increases, as the 20 l/m/s that were proposed in the first place, seems less likely.

Secondly, the team required a higher bound allowable overtopping value. Such a higher estimate needs to make sure that the design does not allow life-threatening situations, even under the highest estimates of climate change. Breaches or catastrophic failures due to overtopping result from erosion of the landward slope and subsequently the dike core. The higher bound allowable overtopping value therefore results from stability of the slope and will amount to roughly 30 l/m/s, as will be shown in chapter 0. Since the resistance measures of the design focus on controlling the overtopping under lower bound conditions and capping it to 5 l/m/s, higher levels of overtopping is expected once the SLR is greater than the lower bound. The resilience aspect of the design takes this into account, but the higher levels of overtopping need to be smaller than the higher bound allowable overtopping value.

Chapter 10 Detailed Robust Design

10 Detailed Robust Design

As already discussed, the central idea of the proposed design is a robust design. The team decided that due to the uncertainty of sea level rise it would not be cost-efficient to design for the high bound climate change scenario (SLR=1.3m). Thus, the idea behind the proposed design is that only in the low bound scenario (SLR=0.65m) the discussed threshold of 5 l/m/s is achieved through resistance (and more costly) measures, which according to the MCA are a living breakwater and dike heightening. While at the high bound scenario, which is less probable to happen, more overtopping will be allowed into the system which will be alleviated by the resilience measures (surface protection, awareness and preparedness, shared initiatives). The proposed design is detailed in the following subsections.

10.1 Resistance Design

10.1.1 Living Breakwater

One of the proposed measures is the construction of a shore-parallel living breakwater which reduces coastal risk through decreasing exposure to wave action. Also, it enhances habitat functions and values by supporting local ecosystems including shellfish (mussels) through the creation and improvement of the nearshore coastal habitat. Living Breakwaters are designed to integrate micro-complexity for a diversity of species. “The living breakwaters provide habitat throughout the water column, from subtidal structure up to the crest level. Underwater, small scale pockets are incorporated into the breakwater that provide foraging and shelter for marine species. Above water, the breakwaters can host seals and nesting birds, providing habitat free from predators” (Parsons Brinckerhoff, 2013). After considering a variety of options (oyster reefs, floating structures etc.) a rubble mound breakwater, that allows mussel growth, was chosen. Naturally growing mussels were observed on the team’s site visit. Thus, Westkapelle seems to be a suitable natural habitat and the environmental parameters (temperature, wave climate, water salinity and pH) permit their growth. A concept idea of the considered type of living breakwater can be seen in Figure 47.

Figure 47: Living Breakwater concept (Parsons Brinckerhoff, 2013)

The next step was to choose the location and dimensions of the living breakwater. For the most efficient mussel growth, they should be submerged half the time of the day (EcoShape, Deltares, & Witteveen&Bos). The problem that arises is the large tidal amplitude in the region. The first iteration was a breakwater that would be completely submerged between mean water and high tide and partly emerged during low tide.

A breakwater like that however, during extreme conditions and considering climate change, would fail to provide a considerable wave height reduction. Through trial and error of different combinations of breakwater height and positions along the sea bed, it was decided that the most efficient would be one positioned near the toe of the structure, with a height of 2m. The concept can be seen in Figure 48 and Figure 49.

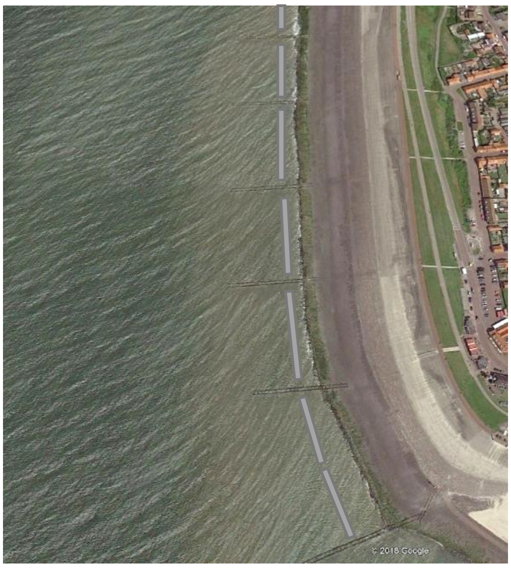


Figure 48: Sketch of living Breakwaters in front of Westkapelle, Google Maps

Figure 49: Detailed sketch of living breakwater in front of Westkapelle sea dike concept, Rijkswaterstaat

The breakwater will consist of multiple rock layers, have a slope of 1:2 and a crest width of 1m (CIRIA et al., 2007) as can be seen in Figure 50.

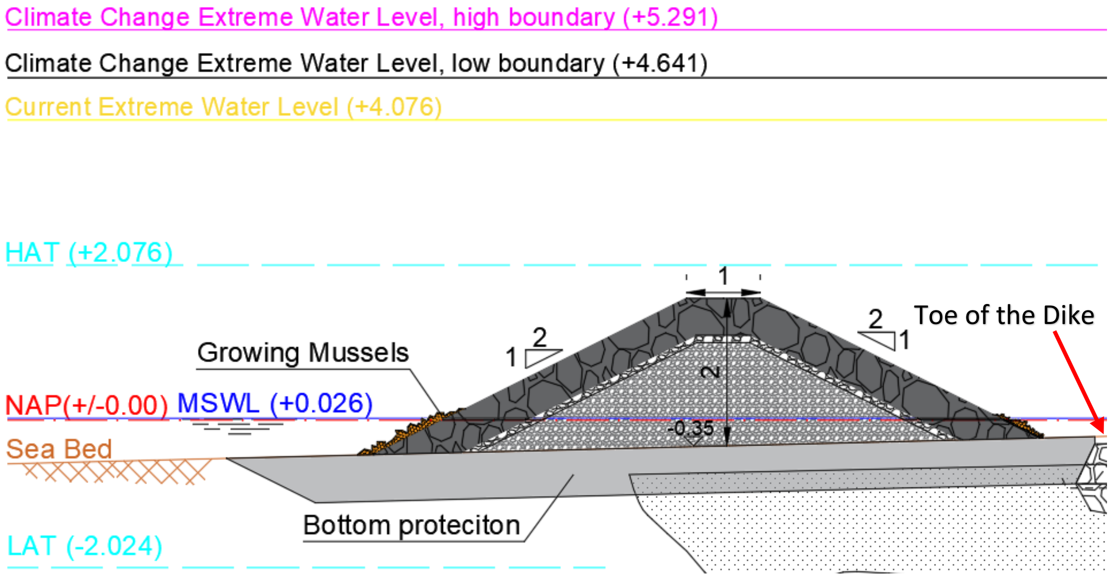


Figure 50: Drawing of the proposed living breakwater

The breakwater will incorporate ECOncrete© units, which are an innovative low pH concrete mix for maritime applications. It was shown that ECOncrete© improves habitat conditions for shellfish (amongst others) significantly (Parsons Brinckerhoff, 2013). An example of the units can be seen in Figure 51. On its base there will be tubes that allow water to flow through during flood and ebb tide. Mussels will not cover the whole breakwater but are expected to cover an area from the sea bed to above the mean sea water level. A more detailed design would require the calculation of weight and size of the armor stones based on different wave conditions, the choice of underlayers and core according to proper grading and filtering of the layers, more refined dimensioning according to geotechnical stability checks, as well as planning of the construction phase.

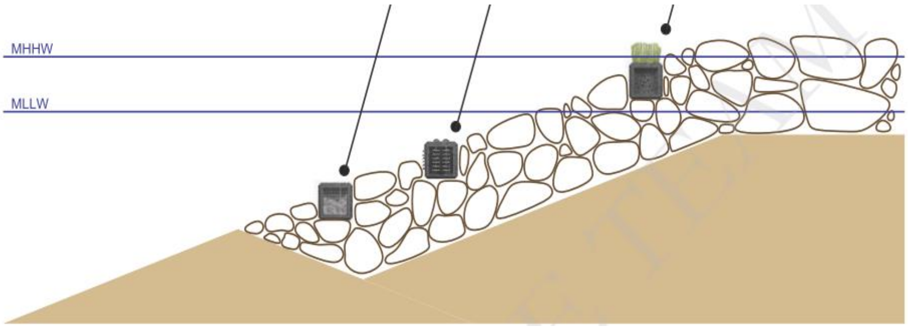


Figure 51: Side view of breakwater with ECOncrete© units, (Parsons Brinckerhoff, 2013)

A lot of thought was given to the hindrance that a breakwater like this might pose to people visiting the waterfront. During high tide the breakwater will be completely submerged and there will be no visual hindrance. There will be roughly 28m in the cross-shore direction between the breakwater and the waterline on the dike. Around mean water level, most of the breakwater will be emerged. Between the waterline on the landward side of the breakwater and the waterline on the dike there will be about 11.5m of shallow water between 0 and 25cm in depth, depending on the sedimentation behind the breakwater. This would be the time of the day where it would pose the most hindrance, as it would reduce the beach width. However, the considered (already existing) cross-section of the dike consists of asphalt penetrated rubble rock and it seems doubtful that people were using its slope for recreation. Between mean water level and low tide, the beach emerges on the seaward side of the breakwater creating a beach front that can reach up to 62m width. If stairs on top of the breakwater or crossings between breakwater sections are provided, the difference from the current situation will not be large for the visitors, as they already have to cross the 9m high dike in order to access the waterfront.

The proposed living breakwater will attenuate the incoming waves under normal and under heavy storm conditions (lower than the four-thousand-year return period). This will reduce damages and maintenance costs of the dike.

In case of both the lower bound (+0.65m water level) climate change and higher bound (+1.3m water level) climate change scenarios in combination with the worst case extreme conditions scenario (four thousand-year return period storm with highest tide and storm surge) the reduction of the incoming waves (and thus dike overtopping) through the breakwater is quite significant, refer to Table 21.

Table 21: Comparison of wave height at the toe of the structure, overtopping and run-up at the dike before and after the construction of the proposed breakwater

Scenario		Hm0 [m]	Hm_0 Reduction [m]	Q [l/s/m]	Q Reduction [l/s/m]	$Ru_{2\%}$ [m]	$Ru_{2\%}$ Reductio n [m]
No Breakwater	**Present**	3.16	-	2.28	-	4.89	-
	Climate change high bound	3.98	-	64.52	-	6.04	-
	Climate change low bound	3.57	-	15.42	-	5.47	-
Breakwater	**Present**	2.85	0.31	0.89	1.89	4.45	0.43
	Climate change high bound	3.67	0.31	43.57	20.95	5.59	0.42
	Climate change low bound	3.26	0.31	8.66	6.76	5.02	0.42

In Table 21, the $Ru_{2\%}$ reduction does not only refer to the difference between the calculated run-up before and after the construction of the breakwater, but also includes the introduced difference in water level at the toe of the structure. That is because the breakwater slightly increases the wave set-up values, and thus results in a higher water level for the scenario with a constructed breakwater.

The costs of the living breakwater are not easy to estimate. That is because the costs are quite different and must be examined in a case by case scenario. The dimensions, the used materials, the quarry or production site and also the transport distance and method play a large role in the cost estimation. Available literature mainly focuses on large scale breakwaters. For this case, where the breakwater is of a very small scale the tables from J. Vos (2016) are adopted in order to acquire a rough estimate. The material cost is around €270/m of length and the labor and equipment costs are around 20€/m of length adding up to €290.000/km plus €10,000 for mobilization costs. According to J. Vos (2016), the European Climate Adaption Platform (2015) and the Scottish Natural Heritage , breakwaters have relatively high construction costs but very low maintenance costs. As the proposed breakwater is further protected by the growing mussels, the maintenance cost is deemed so low (when compared to the magnitude of other costs) that it should not be included.

The construction of the breakwater may cause morphological changes, but a long series of semi-permeable wooden groynes to reduce longshore transport is already built in the region. In comparison to the groynes, the proposed breakwater, or series of breakwater sections, is not expected to have significant effects on the longshore transport.

However, any possible effects would cause sediment transport gradients that foster sedimentation, so that dike safety would be increased. Although, coastal morphodynamic changes are complicated to calculate and foresee in detail, especially because of complicated 3-D effects. Thus, a thorough analysis would be required to gain a full understanding of the impacts that the breakwater might have.

10.1.2 Dike Heightening

Another proposed measure is dike heightening. As previously discussed, the chosen threshold for overtopping is 5 l/m/s. Even with the construction of the living breakwater the expected overtopping is over the threshold value, as can be seen in Table 21. The additional overtopping will be mitigated through dike heightening. For the considered cross-section it is calculated that a dike heightening of 0.31m is required in addition to the living breakwater. To account for inaccuracies during construction, it is decided to increase the dike height by 0.35m. This is done by placing soil on top of the crest. The slopes are decided to remain the same, because steeper slopes might affect the geotechnical stability negatively. Thus, the dike will also need to be widened. In the cross-section under investigation the area in the inner side of the dike is capped with soil to create a straight surface as seen in Figure 52, thus widening is not an issue. For other cross-sections where the road and houses are close to the dike, as visible in Figure 53, a case by case analysis should be carried out, identifying if there is enough space for widening (around 2m at the base of the dike). In most of the sections there is enough space to accommodate the widening.

Figure 52: Considered dike cross-section top view, Google Earth

Figure 53: Area where the dike is close to the road and houses, Google Earth

The proposed design will have an additional height of 0.35m whereas inner and outer slopes as well as crest width do not change due to the reinforcement. Additionally, the dike will be covered by smart grass reinforcement (SGR) as will be discussed in chapter 10.2.1. The design can be seen in Figure 54.

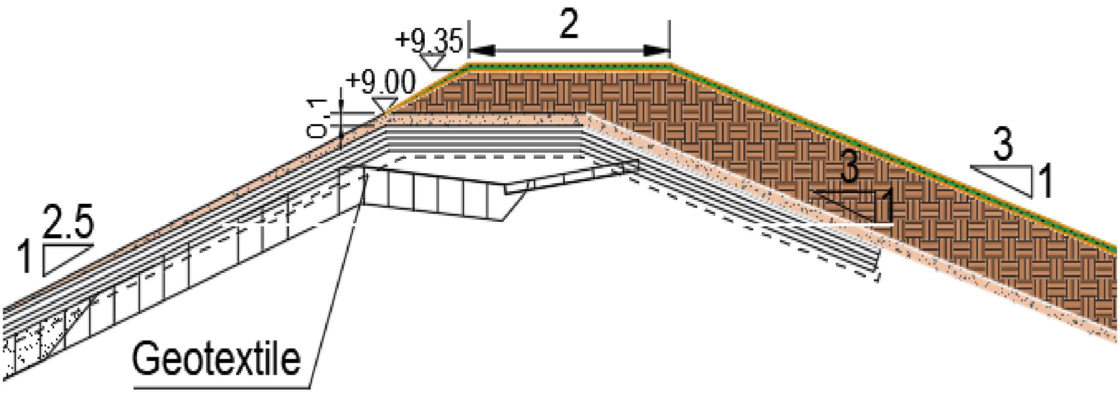


Figure 54: Drawing of the crest of the dike after dike heightening

One can observe in Table 22 that the dike heightening significantly reduces the overtopping and run up. With the dike heightening the overtopping in the lower bound climate change scenario (+0.65m WL) is under the threshold of 5 l/m/s and even in the extreme high bound climate change scenario, it is reduced to 26.77 l/m/s. The surface protection layer is expected to withstand this value, as will be discussed in chapter 10.2.1. Therefore, the possibility of a dike breach is ruled out.

Table 22: Comparison of wave height at the toe of the structure, overtopping and run-up at the dike in the cases of an existing proposed breakwater and with or without the proposed dike heightening

Scenario		$Ru_{2\%}$ [m]	Reduction [m]	Q [l/s/m]	Reduction [l/s/m]
Breakwater & No Heightening	**Present**	4.45	-	0.89	-
	Climate change high bound	5.59	-	43.57	-
	Climate change low bound	5.02	-	8.66	-
Breakwater & Heightening	**Present**	4.43	0.03	0.42	0.47
	Climate change high bound	5.58	0.01	26.77	16.8
	Climate change low bound	5.01	0.01	4.77	3.89

The construction cost for the dike heightening is estimated to be around €4,200,000/km of length (excluding the surface protection) (Committee, 2008). Maintenance costs under normal circumstances are estimated to amount to roughly €42,000/km of length (Waterkeringen, 2006). In this case, however, the breakwater will take most of the damage related to storms. Furthermore, a special surface protection is applied, so that maintenance costs can be expected to reduce to a third, thus €14,000/km of length.

10.1.3 Combination of resistance measures

The proposed reinforcement (resistance measures) is considered the best combination. From Table 21 one can observe that with only the proposed living breakwater, the overtopping is exceeding the requirements. If the threshold overtopping was to be achieved only by the construction of a breakwater, it would require a considerably large structure which would be more expensive than other solutions (e.g only dike heightening or sand nourishments). Furthermore, it would not be accepted by the local inhabitants, as it would cause a hindrance for reaching the waterfront as well as the view. Additionally, it would have to be a largely emerged structure which would inhibit the growth of marine life (the mussels and other organisms need to be frequently submerged).

If only dike heightening was done, one can observe in Table 23 that the overtopping exceeds the requirements. If a larger dike heightening was to be done, one that would be able to reduce the overtopping to the required threshold, it would need to be around 0.7m for the low boundary (threshold 5 l/s/m) and 2.20m for the high boundary (threshold 30 l/s/m). Other than being very costly, this would also require a large widening (3,75m and 11,90m) if the dike slopes were to remain the same, which would lead to constructability issues. It is certain that, for the sections near the Westkapelle city area, there would be not enough space to accommodate such a widening.

Table 23: Run-up and Overtopping comparison between the case of no construction, and dike heightening only (no breakwater)

Scenario	Hm_0 [m]	$Ru_{2\%}$ No Construction [m]	$Ru_{2\%}$ Heightening [m]	q No Construction [l/s/m]	q Heightening [l/s/m]
present	3.16	4.89	4.88	2.28	1.18
climate change (high bound)	3.98	6.04	6.03	64.52	41.46
climate change (low bound)	3.57	5.47	5.45	15.42	9.03

10.1.4 Safety in Lower and Higher Bound Scenarios via Resistance

It is important to examine the response of the resistance measures to different loadings other than the ones designed for, in order to validate its safety. As the proposed design was focused on the threshold of the lower bound climate change, it is important to measure the response to higher loadings, up to the high bound climate change scenario. The run-up and overtopping were calculated for the proposed design, for different percentages of SLR in the range between the two bounds and compared with the case of no dike heightening.

As can be seen in Table 24, before the dike heightening (with a breakwater as proposed) the overtopping threshold would be violated under low bound conditions. Under high bound conditions overtopping of 43.63 l/m/s would occur, which is deemed too large for the surface protection. Therefore, a breach could occur. The design is meant to guarantee that a catastrophic breach will not happen even under the high bound scenario. Thus, with the dike heightening, as seen in Table 24, not only the low bound threshold is attained, but in the high bound the overtopping is less than 30 l/m/s as well. A number that the team is confident that the smart grass reinforcement will be able to withstand.

Table 24: Run-up and Overtopping results for present scenario and for percentages of the climate change projection range

Scenario	No Dike Heightening		Proposed Dike Heightening	
	$Ru_{2\%}$ [m]	q [l/m/s]	$Ru_{2\%}$ [m]	q [l/m/s]
Present	4.45	0.89	4.43	0.42
Climate change (low bound)	5.024	8.66	5.01	4.77
Cl.ch. 70% of bound	5.415	26.42	5.4	15.68
Cl.ch. 80% of bound	5.484	31.18	5.47	18.73
Cl.ch. 90% of bound	5.381	35.97	5.53	21.82
Climate change (high bound)	5.59	43.63	5.58	26.77

Another issue for safety is answering the question, how does the system respond in serviceability limit state (SLS). In other words, if the design is safe for loading scenarios that will occur more frequently.

As can be seen in Table 25, the cases of storms with return period of 10 and 100 years were examined under the present water level and high and low projections of climate change for the cases of no construction, construction of only the breakwater and the proposed design of both the breakwater and dike heightening.

In Table 25 it can be observed that in the present and low bound climate change scenario the overtopping remains under the threshold of 5 l/m/s, while only for the high bound climate change scenario higher values of overtopping are met, as expected (still far from the threshold of 30l/m/s). Concerning the usability of the dike and the local inhabitant's psychology (as discussed in chapter 0), for the present conditions the overtopping values will remain quite low even during intense storms no matter the construction. As sea level rises, and after 2050 when intense storms are met, there will be roughly 2-3.2 l/m/s overtopping. This is even considered safe for people walking on the dike as according to (J.W. Van der Meer et al., 2016) where the safety limit for persons on the dike crest is around 5 l/s per m.

Only in the case of high bound climate change, which is quite uncertain to occur, higher overtopping values are calculated, which are meant to be handled by the system via resilience.

Table 25: Run-up and Overtopping for storms with return period of 10 and 100 years (SLS) with and without climate change for various construction phases

Scenario		No construction		Breakwater only		Full reinforcement	
	Storm Return period [y]	$Ru_{2\%}$ [m]	q [l/s/m]	$Ru_{2\%}$ [m]	q [l/s/m]	$Ru_{2\%}$ [m]	q [l/s/m]
Present	100	4.87	1.57	4.38	0.59	4.37	0.28
	10	4.84	1.3	4.35	0.47	4.33	0.21
High Bound climate change	100	6.00	48.41	5.54	32.3	5.53	19.52
	10	5.93	40.89	5.5	27.55	5.49	16.5
Low bound climate change	100	5.41	10.7	4.96	5.86	4.49	3.16
	10	5.39	9.13	4.92	4.8	4.91	2.56

From this it can be concluded that for the lower bound climate change scenario the resilience measures are not required. Still they are included in the proposed design to mitigate the overtopping in the case of higher bound scenarios. Thus, achieving a robust solution.

10.2 Resilience Design

10.2.1 Surface Protection

Grass is a common solution in protecting the landward slope, forming a beautiful landscape and being environmentally friendly (CIRIA, 2013). As an addition to natural grass, an SGR was incorporated into the proposed design. It mainly aims at strengthening the present grass cover at the crests and landward slope of the dike.

Field testing (J. W. van der Meer et al., 2009b) shows that the adopted measure enables an optimum root penetration and intertwinement of the grass roots and, at the same time, provides sufficient shelter to the under-lying substrate when uncovered. In addition, the SGR also contributes to the mitigation of other failure mechanisms such as shallow slip failure and internal erosion (Van Gerven, Van der Meer, Van Heerenveld, & Akkerman, 2006).

10.2.1.1 Smart Grass Reinforcement

As stated previously, the determined threshold for the overtopping is 5 l/m/s. The living breakwater and dike heightening are sufficient to reduce overtopping to 5 l/m/s for the low boundary climate change scenario, and although the climate change upper bound scenario is not as likely to happen, a constructive measure such as SGR should be incorporated to prepare for the higher scenarios.

This concept implies the deployment of a machine that cuts the grass at some distance below the surface and lifts the upper grass cover temporarily while placing a geosynthetic at the same time underneath, as shown in Figure 55 (Akkerman, Bernardini, van der Meer, Verheij, & van Hoven, 2009).

Figure 55: Schematic sketch of placement of the geosynthetic grass cover on the sea dike in Groningen, (Akkerman et al., 2009)

The Geocell system (Figure 56) is a feasible example of an SGR. In the figure it was penetrated through the existing grass revetment, without lifting the grass. This system can be applied at uneven surfaces and very poor grass covers (Van Gerven et al., 2006).

Figure 56: Geocell system (Van Gerven et al., 2006)

10.2.1.2 Overtopping and Erosion Resistance

Under the attack of overtopping flows, a grass covered slope can be eroded, thus possibly resulting in breaching (Hai & Verhagen, 2014). From 2009 to 2011, destructive wave overtopping tests have been conducted for more than 19 sections in the Netherlands including different slope specifications (Steendam, Provoost, & van der Meer, 2012; Trung, Verhagen, van der Meer, & Cat, 2012). Among those tests, a dike in Delfzijl, Groningen with three different slope specifications was tested against wave overtopping by the wave overtopping simulator. These include a normal grass cover, a section reinforced with geosynthetic grass cover and a bare clay slope with no grass cover (Trung et al., 2012). The major goal of the tests in Groningen was to check the performance of the SGR, as compared to the natural grassed slope (Akkerman et al., 2009).

After a week of testing, up to the overtopping discharge of 30 l/s/m, the grass covers of both the natural grass section (a) and SGR section (b), see Figure 57, were still intact (J. W. van der Meer et al., 2009a)). It should be noted here that both sections were treated and maintained properly prior to testing. Afterwards, different types of manual damages were applied to the inner slope such as small holes, major bare spots and placement of poles and pickets as shown in Figure 57 (Akkerman et al., 2009).

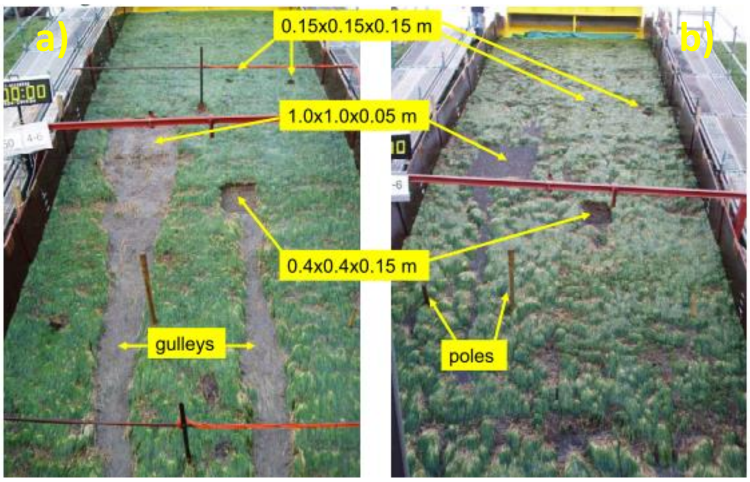


Figure 57: Final result Delfzijl, Groningen, sections (a) Natura grass and (b) SGR section

After testing, a marked difference could be observed between the natural grass section and the SGR section. This is illustrated in Figure 58, which shows the damage patterns after completion of the 50 l/m/s test (Akkerman et al., 2009).

Figure 58: Damage pattern at the natural grass section and the SGR section

At the non-reinforced section, long gullies developed and extended to the toe of the dike. At the SGR section, in contrast, no gullies developed (Akkerman et al., 2009)). Thus, The SGR system appeared to be stronger than a normal grass cover (J. W. van der Meer et al., 2009a)).

Additionally, Steendam et al. (2012) argues that the erosional resistance against wave overtopping of a grass cover is mainly determined by the root system rather than the soil characteristics (Trung et al., 2012). From the destructive test at the SGR section (Figure 57 and Figure 58) gully formation was virtually absent. This can be explained from the presence of the SGR, by which the grass layer is well reinforced with the geosynthetic. Figure 59 illustrates the grass roots that 'anchored' very well to the geosynthetic.

Figure 59: Anchoring of the grass roots to the geosynthetic (Akkerman et al., 2009)

10.2.1.3 Transitions

The overall strength of a dike body depends on its weakest link. Transitions in a landward slope are the weak spots in the dike system and large objects result in concentrated damage (e.g. staircases, trees, roads) (J. W. van der Meer, 2008). At weak spots, erosion was already observed for overtopping discharges less than 10 l/s/m. At a mean overtopping discharge of 1 l/s/m strong erosion started alongside the stairs, while for an undisturbed slope hardly any erosion occurred for discharges of 30 l/m/s (Steendam et al., 2012). Any existing dike in the Netherlands is a mix of disturbed (staircase, holes, transitions, etc) and undisturbed slope and any damage in one part will propagate to another part, possibly resulting in a breach. As the SGR can be embedded in or connected to the abovementioned disturbances, it would improve the connection between grass and disturbance. Thus, weak spots are strengthened.

The expected overtopping for both low and high climate change scenarios is proved to be withstood by the SGR with an extra margin for the uncertainties within climate change projection. At the higher climate change scenario, the expected wave overtopping is almost 27 l/s per m. The SGR was proved to remain intact after testing up to the overtopping discharge of 30 l/s per m (J. W. van der Meer et al., 2009a).

10.2.1.4 Costs

The estimated cost of Geocells amounts to €310/m of defence (ComCoast, 2005). Including 40% for contractor's surcharges, the cost for every kilometer strengthened sea defence will be about €280,500 for the Geocell system. Additionally, roughly €10,000 should be added for mobilization costs. These cost figures do not include design costs. It is expected that maintenance is restricted to occasional and local repair of the grass cover, if Geocells should become exposed. The material itself, however, is assumed not to become damaged. Hence, an annual budget of €1,000 per year per km length is reserved for small repairs, additional to the inspection cost of €5,000. As regards the life time expectancy of 50 years, the capitalized maintenance cost remains modest (ComCoast, 2005).

10.2.2 Awareness and Preparedness

Awareness and preparedness as a design option was introduced to cater with the situations when there is flood. However, a great deal of planning and procedural measures must be taken in times when there is no flood. Components of the awareness and preparedness plan are seminars, trainings, mock drills, and evacuation plans.

10.2.2.1 Emergency preparedness

This proposes a compulsory emergency kit to be owned by every individual for themselves, their family and livestock. This kit should have enough supplies to survive for 72 hours after a disaster. It is advisable to store these kits in more than one location, preferably where one spends the largest part of the day e.g. home, office, vehicle etc. Major items for the emergency kit are non-perishable food, drinking water, battery powered radio, flashlight, first aid kit, whistle, local map with evacuation routes, and a cell phone (FEMA).

Table 26: Emergency kit supplies as recommended plan for preparedness Source: (FEMA) and (Red Cross, 2009)

Item	Quantity (per person)	Function
Non-perishable food	Three-day supply	Food to maintain body energy levels to think and act proactively when in crisis.
Drinking water	Three-day supply – 6 liters	Water for drinking to avoid dehydration.
Battery powered radio	One radio with two extra batteries.	To stay updated to developing situations and announcements.
Flashlight	One flashlight with two extra batteries.	It is very likely that electricity is down in flood times. Evacuations are continuous and at nighttime lights are required. Also, flashlights could be used as a distress signal.
First aid kit	Seven-day supply Includes small manual for common diseases and typical epidemics.	Should contain basic bandage aids and medicine for common waterborne diseases. It would be disaster within a disaster if there is a disease. Better safe than sorry: In case of spread of an epidemic protective measure should be taken beyond first aid e.g. household chlorine bleach and medicine dropper to be used as disinfectant.
Whistle	One	To be used as a distress signal
Dusk masks and plastic bags	Five	Situations get worse if the environment gets polluted. Masks should be used for protection against contaminated air and plastic bags to avoid littering.
Local map with evacuation routes	One	It is common to use electronic maps these days, but one should have a hard copy of local maps with mentioned major shelter places, major roads and marked evacuation routes.
Cell phone	One with two extra batteries	To be used to inform about position and call for help in case of loss of contact
Waterproof container	One	Items: prescriptions, insurance policy, cash, matches, emergency contact information, documents providing personal information etc.

Since Westkapelle is a rural area, livestock might be a source of income for people and resilience measures need to include livestock. A livestock emergency kit must include: pet and livestock food, medications and vaccinations, extra leash, microchip and tattoo number (Pennsylvania Government).

If evacuation is impossible, then enough food and water supplies for the livestock and pets should be stockpiled. There must be a sign high on the building enabling rescuers to recognize it (PEMA, 2018).

Another important consideration is people with disabilities (PEMA, 2018). They take more time or in many situations need external support. Deaf and blind must be informed through means other than just an announcement. People with mobility disabilities should be put on specially designed exit routes. For people with similarly severe disabilities means of communication and methods to help must be improvised.

Businesses can also play an integral role in reducing flood risk by taking part in preparedness and mitigation plans. If businesses shut after flooding than it has a trickle-down effect on the whole market both regionally and nationally. 40% of small businesses cannot reopen and 75% of small businesses would fail within three years after a flooding event if they have no preparedness and mitigation plan (FEMA). Major steps in developing such a plan are:

1. Identify potential flooding risks by assessing the readiness of staff, surroundings, space, systems, structure and services.
2. Develop a plan by identifying critical measures to reduce the identified risks in the previous step.
   a. Staff: crisis communication plans could be developed, or flood drills could be carried out.
   b. Surroundings: Engineering company could be hired to conduct flood risk assessment and propose flood mitigation measures like flood wall, multifunctionality of buildings etc.
   c. Space: Relocation of critical contents, electrical sockets and chemicals at higher elevations
   d. Systems: elevating or anchoring of mechanical, electrical water systems.
   e. Structure: Raising of structures, wet and dry flood proofing
   f. Services: Engage with emergency management services provided in the area
3. Take action to implement the decisions that were made in previous steps.

10.2.2.2 Flood awareness framework

The commonly used Delta program strategies are considered a traditional flood risk management scheme which is mainly focused on the hard flood defense. Although, the Dutch government is aware of the other measures, such as mitigation and evacuation, there is a large awareness gap relating to flood risks among the Dutch citizens. This awareness gap is due to the complete trust Dutch citizens have in their government and competent authority. The citizens do not perceive flooding as a real risk. Thus, many people are unaware of the water risk, the basics about evacuation and whether their house was built on a flood plain (OECD, 2014)

If people are to become more self-reliant in the case of flooding, governments need to understand the need of people to receive information (Middelburg (NL), 2013). In this case, a flood awareness framework is necessary – a framework on which the development of an awareness campaign can be based in order to raise public awareness and self-efficacy (Middelburg (NL), 2013). An Interreg IVA 2Seas project suggested a framework that is comprised of the following five stages:

Stage 1: identifying the target group and level of awareness

Identifying the target groups and their level of awareness is important. The following figure shows the elements that need to be understood from the target groups' perspective.

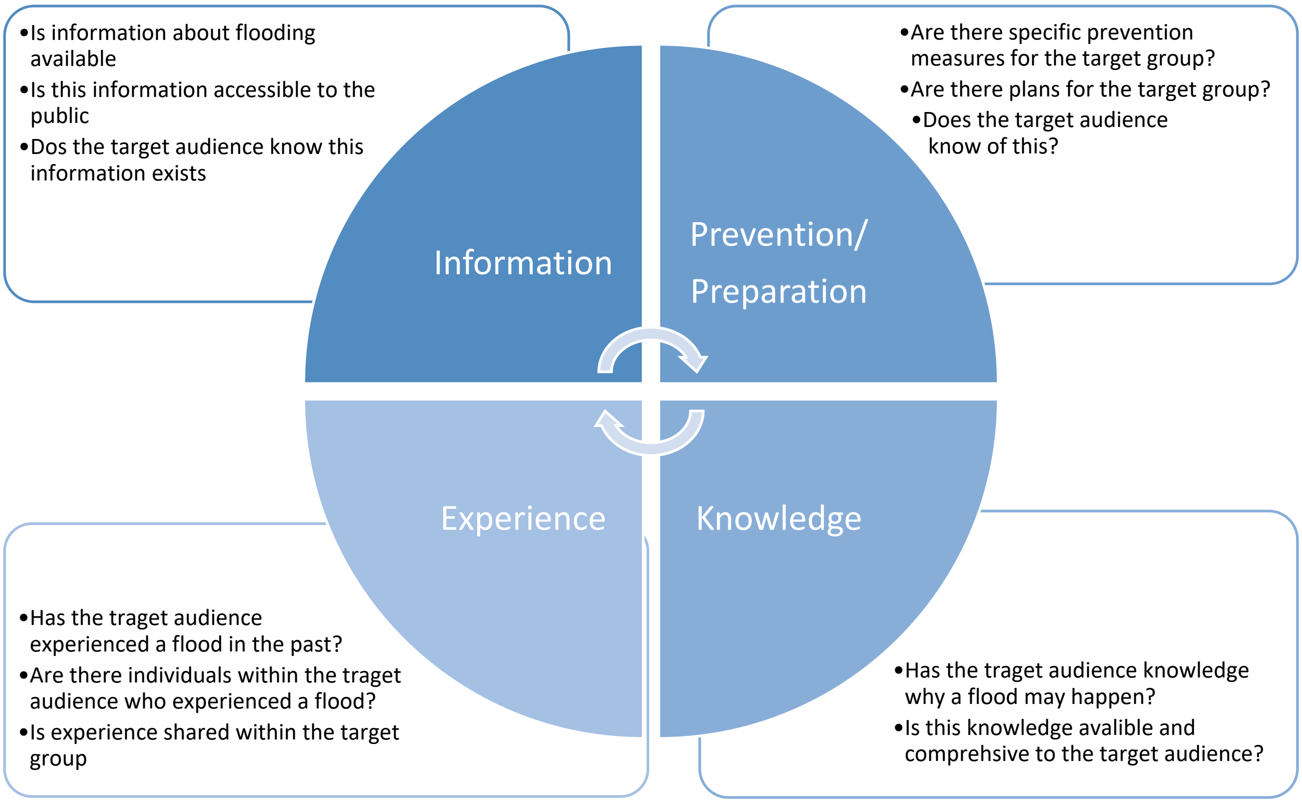


Figure 60: Elements defining the target groups' awareness

Stage 2: Finding out the needs of the target groups

In this stage more information about the group's level of awareness is obtained. One can investigate what the target audience already knows, what information they have and what prevention measures they know of. This can lead to find out the lacking information. Figure 61 shows the level of awareness indicator.

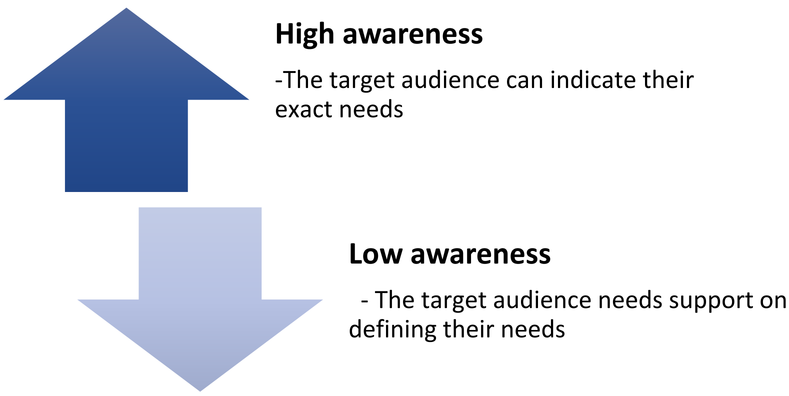


Figure 61: Indicators of Level of awareness

Stage 3: Finding out the best way of distributing information

Different audience groups need different ways and channels of effective communication. By identifying these ways of distributing information on flooding, the level of awareness could be increased among the audience.

Stage 4: Starting the campaign

Campaign should be launched addressing the needs of the target group. This could be focused on one element or combination of elements presented in stage 1. Which elements needs to be addressed in what depth is determined from stage 2. Figure 62 below shows the impact of campaign on the level of awareness among the target audience

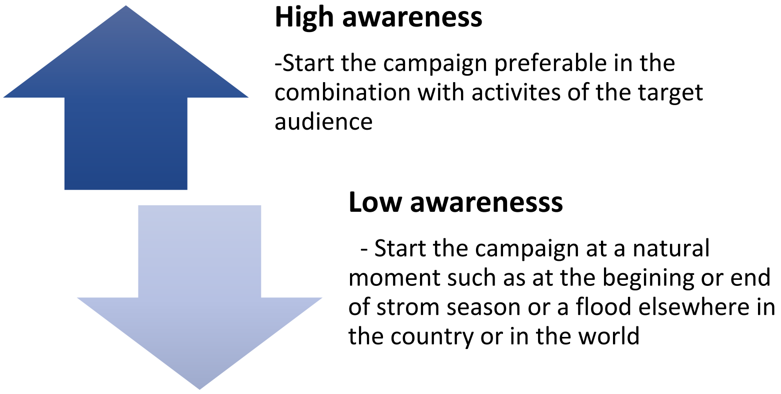


Figure 62: Impact of campaign depending on awareness level of target audience.

Stage 5 Evaluate and moving on

While the campaign is ongoing/running, it should be evaluated together with the target audience. This method will result in identifying the awareness weaknesses and gaps. Figure 63 illustrates the stages of evaluation.

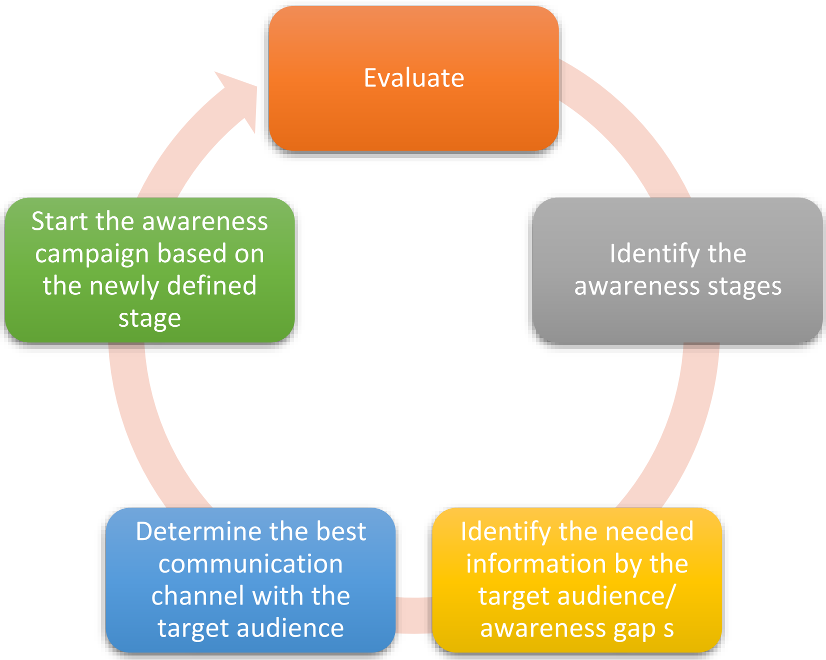


Figure 63: Stages of evaluation of campaign (Middelburg (NL), 2013).

10.2.2.3 Seminars

As a part of the flood awareness framework seminars could be conducted on the following topics:

- Information on surrounding property and safety evaluation
- Flood history in Netherlands
- Flood plain management
- Geography and topography

10.2.2.4 Evacuation plans

Early warning systems can reduce risks but only if evacuation plans are in place. Figure 64 shows expected percentages of evacuation, if a 70-hour warning is provided. Note that Westkapelle has one of the lowest percentages. The main reason is that once the flood comes, the road links to other cities would also be flooded. Therefore, the team recommends an efficient flood resilient transport system with workable evacuation plans marking major roads with color coding. Evacuation models must be generated and research must be carried out to optimize them. Generally, in a flood situation it is a global practice to take care of livestock first because it takes more time to be evacuated.

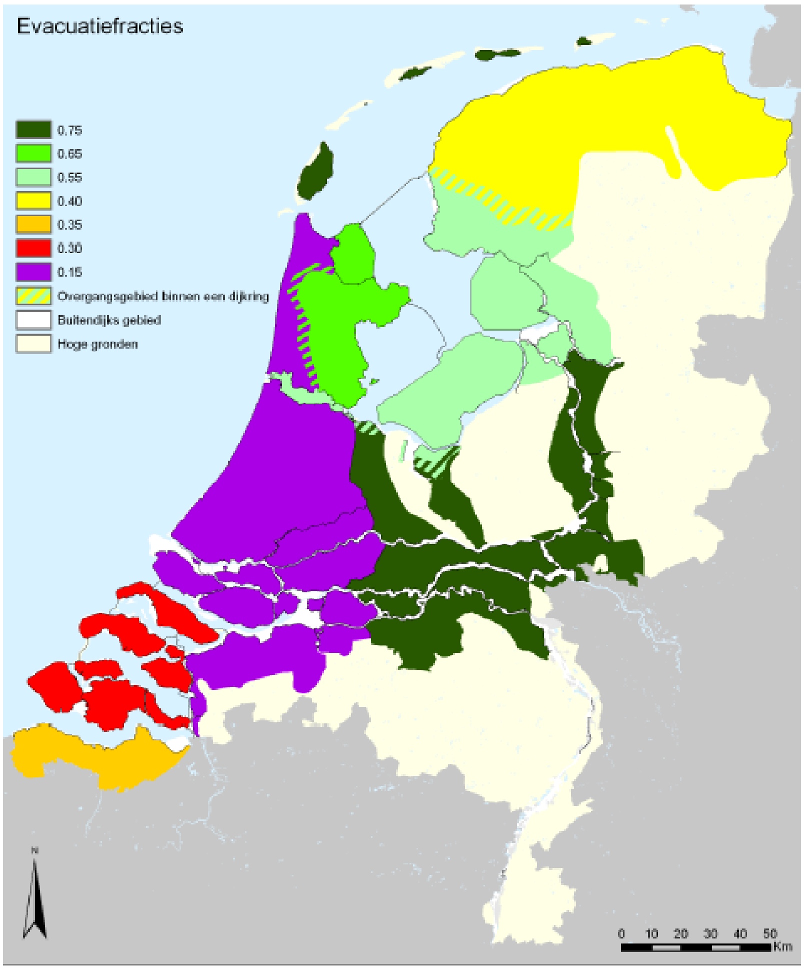


Figure 64: Evacuation percentages Source: (Kolen, Maaskant, & Terpstra, 2013)

The table below shows evaluation of having better evacuation plans.

Table 27: Evaluation of better evacuation plan. Source: (UNESCO)

Measure/desired outcome	Reduction in loss of life	Reduction in property loss	Protection of critical infrastructure	Costs	Social challenges	Other factors
Evacuation	Reduces to near zero	Minimal impact structure loss; some reduction in personal property loss	Minimal impact	Relocation process; temporary lodging; structure rebuilding; individual compensation	Can only be used infrequently; high social disruption	Minimizes damage to the natural environment

1950-Free Consultants

10.2.3 Shared Initiatives

Shared initiatives involve stakeholder consultations and an integrated approach to elevate the resilience of the system. Different measures can add to the robustness of the system. On one hand, consultations between professionals and local inhabitants, e.g. Rijkswaterstaat meeting local farmers every 3 months to discuss about ongoing flood defense works. On the other hand, initiatives such as community and capacity building schemes, collective gardens, etc. A set of shared initiative measures is proposed hereunder.

10.2.3.1 Shared Initiative Pro forma

Stakeholders involved are to be consulted before any project starts in the region. This is ensured by introducing a policy level change for construction or maintenance projects in the area. Whenever a new project is announced by the government, it has go to through a bidding phase. The new policy would require a 'Shared Initiative Pro forma' to be included in the tender documents at the time of the bidding for the project. This Pro forma would include a thorough stakeholder analysis to make sure that the contractor or the consultant, depending on the type of the project, understands the needs and concerns of all the stakeholders associated with the project. It should also include a detailed strategy to cater for the concerns of stakeholders as good as possible.

10.2.3.2 Detailed strategy and Workplan

Shared initiatives with stakeholders should also include a Cost Benefit Analysis (CBA), Cost Effectiveness Analysis (CEA) and Multi-criteria Analysis (MCA). Amongst these techniques MCA is the most effective and recommended technique as it takes into account multiple evaluation techniques. First, objectives and related indicators have to be identified. In this way stakeholders would be involved at an early stage in decision making as their development objectives are identified and criteria is weighed so the stakeholder participation is maximum. This is more suitable in countries where climate changes have multidimensional impact and data is not easily available (Mirza, 2003).

10.2.3.3 Task force

Another aspect of this plan is to incorporate team work with a minimal control lying with the municipality to have some check and balance. A team should be established who works towards planning, implementing and monitoring the shared initiative program. The team would also be responsible for the regular vulnerability assessment in which all stakeholders give their suggestions and the most practical options are shortlisted. Then a rapid participatory integrated assessment should be conducted in which key stakeholder concerns should be addressed and adaptation measures should be devised. At the end periodic review of risks and prioritization of activities should be done and constant monitoring should be done

Many potential adaptation processes could be made with collaboration of stakeholders even though for most of the time the government would have to be involved to a great extent. A Disaster Management Bureau should be established which focuses on coastal areas and other areas subjected to floods that make relief camps beforehand. Enhancing the emergency responses should be their main priority in case if flood hits the country.

10.3 Recommended construction summary

The proposed design implements both resistance and resilience measures to form a robust design. The resistance measures incorporate two "hard" reinforcements, the living breakwater and dike heightening. The resilience part is devised by one "hard" and two "soft" measures, the surface protection (SGR), awareness and preparedness and shared initiatives. In Table 28, a summary of the proposed measures is presented.

Table 28: Summary of the proposed measures

Proposed Measure	Type	Dimensions	Implementation
Living Breakwater	Rubble mount breakwater with ECOncrete© units and growing mussels	H: 2m W: 1/9m Slope: 1:2	-
Dike Heightening	Soil heightening (and widening) with surface protection	H: 0.35m W: 2/3.9m Slope: 1:2.5 /1:3	-
Surface Protection	Smart grass reinforcement (SGR)	L: ~40m Thickness: 0.1m	-
Awareness and Preparedness	Emergency preparedness, flood awareness framework, seminars and evacuation plans	-	Emergency Kit Evacuation Plans Business Preparedness Awareness campaign Seminars
Shared Initiatives	Share Initiative Pro Forma, Detailed strategy and workplan, Task Force	-	Stakeholder Consultation CBA, CEA and MCA Analysis Task Force organization Disaster Management Bureau establishment

The detailed drawings of the considered cross-section before and after the reinforcement can be found at the end of the Appendix.

It is important to note that although the specific cross-section was considered in this analysis, the choices and design are made so that it can be implemented along the Westkapelle waterfront. The loading is subject to changes as the direction and bathymetry changes along the coast, as well as the different dike dimensions and materials. Thus, the required dimensions of the reinforcement are subject to change as well. However, the idea behind the design remains the same and with these changes the proposed design could be implemented along the whole waterfront. The different cross-sections won't affect the resilience measures which are the same for the whole system.

Chapter 11
Sensitivity to Failure Probability

11 Sensitivity to Failure Probability

One of the major aims of the project was to understand the sensitivity of the failure probability of the dike in response to sea level rise. In this chapter a failure probability model has been proposed based on the failure mechanism of overtopping. Various approaches are made to address the problem and resulting failure probabilities are discussed in line with the detailed design.

11.1 Failure Probability model

A detailed hydrodynamic analysis was conducted to obtain the loading conditions under a 1/4000-year storm up until the year 2100. These conditions were used to calculate overtopping discharges and conduct a sensitivity analysis for the parameters, as presented in earlier chapters. Deterministic values need to be assumed for the analysis, even though the real world is far from deterministic and parameters are subject to uncertainties. Therefore, a probabilistic approach is needed. The description of terms in the overtopping formula are presented in previous chapters. The structure of the failure probability model is presented in Figure 65 and explained in the following paragraphs:

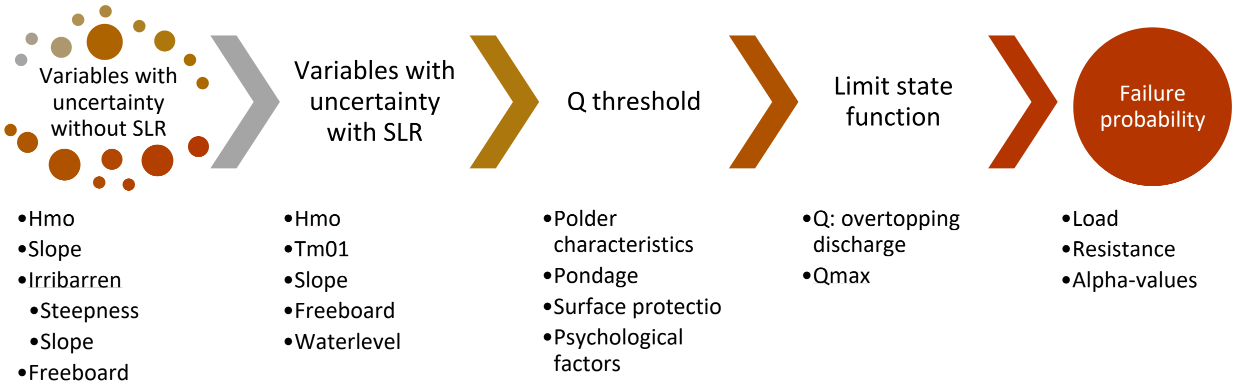


Figure 65: Failure probability model

In the model, variables were studied that had uncertainty. This was done for conditions both with and without climate change. Climate change affects the following variables: water level, wave height, wave steepness, wave period, average slope of the dike, and freeboard. Therefore, these variables changed in different scenarios of climate change and design conditions.

For failure probability calculations a limit state needs to be defined. A limit state is a condition of a system beyond which it no longer fulfils the relevant design criteria ("Eurocode EN 1990: Basis of Structural Design," 2001). It is determined by assessing the resistance R provided by the system and load S imposed on the system. Therefore, the limit state function Z is defined as loads subtracted from resistance. For this case, resistance was the threshold overtopping discharge and load was the overtopping discharge calculated for different hydrodynamic and design conditions. The limit state function is presented below:

$$Z = R - S = q_{threshold} - q$$

Critical overtopping discharge is threshold $q_{threshold}$ and overtopping discharge q (supposedly Weibull distributed) is:

$$q = \sqrt{g \times H_{m0}^{3}} \times \left\{ \frac{0.026}{\sqrt{tan\alpha}} \times \gamma_{b} \times \xi_{m-1,0} \times \exp\left[-\left(2.5 \times \frac{R_c}{\xi_{m-1,0} \times H_{m0} \times \gamma_{b} \times \gamma_{f} \times \gamma_{\beta} \times \gamma_{\nu}} \right)^{1.3} \right] \right\}$$

Probability of failure is then calculated as:

$$P_f = P(Z < 0) = \int_{0}^{\infty} f_H(h) \times F_R(h) dh$$

$f_H(h)$ - probability density function of hydraulic load levels; $F_R(h)$- cumulative distribution of resistance given a certain hydraulic load level (Jonkman, Steenbergen, Morales-Napoles, Vrouwenvelder, & Vrijling, 2015).

11.1.1 Scenarios

As stated above, loads were calculated for different climate change scenarios, which affected offshore conditions, and different design scenarios, which affected onshore conditions. In the probabilistic assessment three cases in terms of climate change were considered:

1. Current case: without taking into account effects of climate change
2. High bound: with taking into account higher bound of climate change
3. Low bound: with taking into account lower bound of climate change

Along with that, four design conditions were considered.

1. Do nothing: without taking any measure to counter overtopping
2. Breakwater: countering overtopping by building a living breakwater
3. Dike heightening: countering overtopping by raising the crest level of the dike
4. Robust design: countering overtopping by both building a living breakwater and raising the crest level of the dike as a part of robust design approach

Different hydrodynamic conditions were used to come up with offshore extremes of water levels, wave heights, periods etc. for different climate change scenarios. The SWANOne model was used to transform these offshore conditions to nearshore conditions. Those were then input in a MATLAB script to give overtopping values. For calculations, bathymetry was changed in SWANOne for each of the design conditions and combined with climate change scenarios, which gave twelve different model cases to study, as highlighted in the following figure.

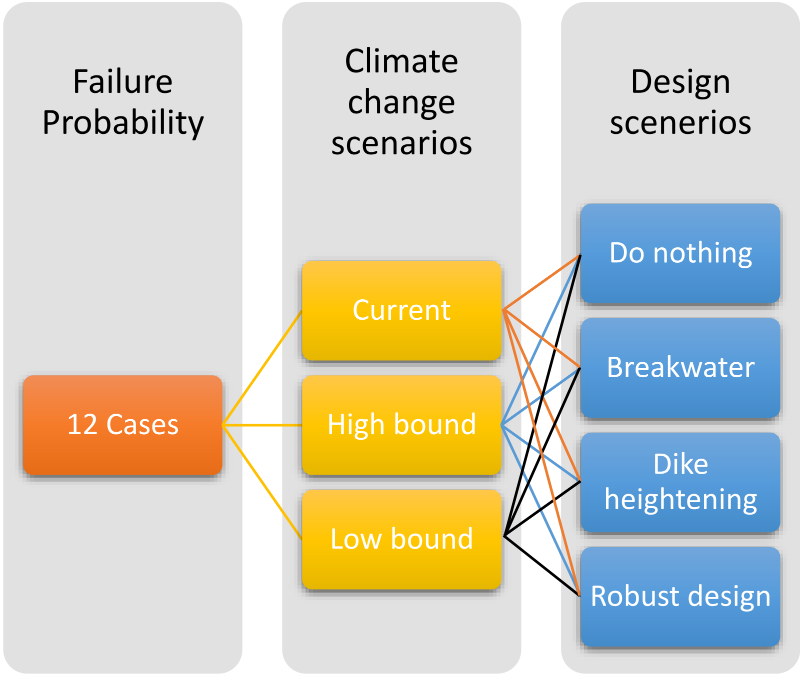


Figure 66: Cases for failure probability calculations

11.2 Approaches

Different approaches were employed to assess the failure probability. They mainly differ in the way variables are treated in the model. Additionally, the model cases may vary depending on the type of approach.

11.2.1 Semi-deterministic approach

In the semi-deterministic approach variables were calculated and 12 cases were generated, as explained above. Deterministic values of wave height H_{m0}, wave period T_{m01}, average slope angle α and freeboard R_c were used in the model. Stochasticity was only introduced in the coefficients $C_1 = 0.026$ and $C_2 = 2.5$ in the overtopping formula. However deterministic values were different in all 12 cases that made it possible to account for climate change even in this approach.

11.2.2 Stochastic approach

In practice, variables show variations and even for one specific case, calculated deterministically, variables would contain uncertainty. For instance, if the variable is measured then the uncertainty could be introduced from measurement apparatus or if it is calculated then this uncertainty could be introduced though the independent variables in the formula. Either way there is some 'stochasticity' in parameters which makes it necessary to conduct a probabilistic analysis with a stochastic approach for our project. Therefore, the case variables, that are directly affected by changes in water level, were considered stochastic and distributions were applied to them. For most of the variables a normal distribution was assumed, however the statistical parameters are case specific.

11.2.3 Water level functions approach

This approach was introduced to account for climate change directly. It was hypothesized that water level changes are the direct impact of the climate change. In climate change scenarios sea level rise was observed and sea level rise changed the water levels on the toe of the dike. Now with this change in water level all the variables that had uncertainty in it (wave height H_{m0}, wave period T_{m01}, average slope angle α and freeboard R_c) were also supposed to change. To account for this change in variable all these variables were made dependent on the water level as a function of it. This was done by running SWANOne models for different water levels and a dataset was prepared. Subsequently, curve fitting to the dataset yielded functions for all the variables in terms of water level.

11.3 Model Parameters

For the probabilistic modelling all the parameters in the limit state function are to be determined. Usually universal constants like gravitational acceleration, g, are kept as deterministic and all other variables that can vary with surroundings are made stochastic. In our project there are some variables which remain constant like gamma correction factors in all approaches and models but some change. The complete list of variables is presented in each approach with its values and an explanation.

11.3.1 Semi-deterministic approach

The variables in this approach were calculated through SWANOne runs for different hydrodynamic and design conditions. Summary of the values used in the model for different design scenarios is presented in following sections.

11.3.1.1 Do nothing

The do nothing scenario represents the flooding scenario when a 1/4000 storm occurs but no measures were taken to counter it. The corresponding hydrodynamic parameters for the failure probability model are tabulated and presented below:

29: Model inputs for semi-deterministic approach - do nothing scenario

Scenario Table	WL+setup	Hm0	Tm01	alpha	Rc
Current Extreme	4.468	3.16	8.51	0.191	4.531
Climate Change (high bound)	5.698	3.98	8.65	0.2063	3.302
Climate Change (low bound)	5.094	3.57	8.54	0.1998	3.906

11.3.1.2 Breakwater

The breakwater scenario represents the flooding scenario when a 1/4000 storm occurs and a living breakwater was built to counter it. This results in the following parameters:

Table 30: Model inputs for semi-deterministic approach - breakwater scenario

Scenario	WL+setup	Hm0	Tm01	alpha	Rc
Current Extreme	4.474	2.85	8.33	0.1868	4.526
Climate Change (high bound)	5.727	3.67	8.61	0.1999	3.273
Climate Change (low bound)	5.124	3.26	8.45	0.1914	3.876

11.3.1.3 Dike heightening

The dike heightening scenario represents the flooding scenario when a 1/4000 storm occurs and a dike heightening was undertaken to counter it. The following parameters are obtained:

Table 31: Model inputs for semi-deterministic approach - dike heightening scenario

Scenario	WL+setup	Hm0	Tm01	alpha	Rc
Current Extreme	4.468	3.16	8.51	0.1905	4.881
Climate Change (high bound)	5.698	3.98	8.65	0.206	3.652
Climate Change (low bound)	5.094	3.57	8.54	0.1994	4.256

11.3.1.4 Robust design

The robust design scenario is the combination of breakwater and dike heightening. These mainly account for the resistance part of robustness. It represents the flooding scenario when a 1/4000 storm occurs and it would be countered by both the living breakwater and the raised dike simultaneously. This results in following values from SWANOne:

Table 32: Model inputs for semi-deterministic approach - robust design scenario

Scenario	WL+setup	Hm0	Tm01	alpha	Rc
Current Extreme	4.474	2.85	8.33	0.1862	4.876
Climate Change (high bound)	5.727	3.67	8.61	0.1995	3.773
Climate Change (low bound)	5.124	3.26	8.45	0.1991	3.873

11.3.2 Stochastic approach

Care needs to be taken in the determination of parameters in stochastic approach, as the uncertainty of the individual parameters could greatly affect the reliability of the model. The distribution of each parameter was determined by taking mean and standard deviations from the dataset that was developed with SWANOne runs. For instance, all the water level values for the do nothing scenario were averaged and the standard deviation was assumed to amount to 10% of the mean which resulted in water level distribution to be used in calculation of failure probability for the do nothing scenario. Similarly, other distributions were determined and are presented below.

Table 33: Model input for stochastic approach - all scenarios

		WL+setup	Hm0	Tm01	alpha	Rc
	Distribution	Normal	Normal	Normal	Normal	Normal
	Standard deviation, σ	0.5	0.35	0.8	0.02	0.4
Mean μ	Do nothing	5.08667	3.57	8.5667	0.199033	3.913
	Breakwater	5.10833	3.26	8.4633	0.192713	3.8917
	Dike heightening	5.08667	3.57	8.5667	0.198633	4.263
	Robust design	5.10833	3.26	8.4633	0.194933	4.174

The reason why all distributions were assumed to be normal because its general nature. We have the data for offshore conditions and we transformed it to nearshore for different scenarios. Enough runs were conducted to come up with a representative mean and eventually a distribution. Ideally, water level should be Gumbel distributed, critical overtopping should be lognormally distributed, and crest height should be normally distributed to name a few. Distributions of Robust design scenario are presented below as an example.

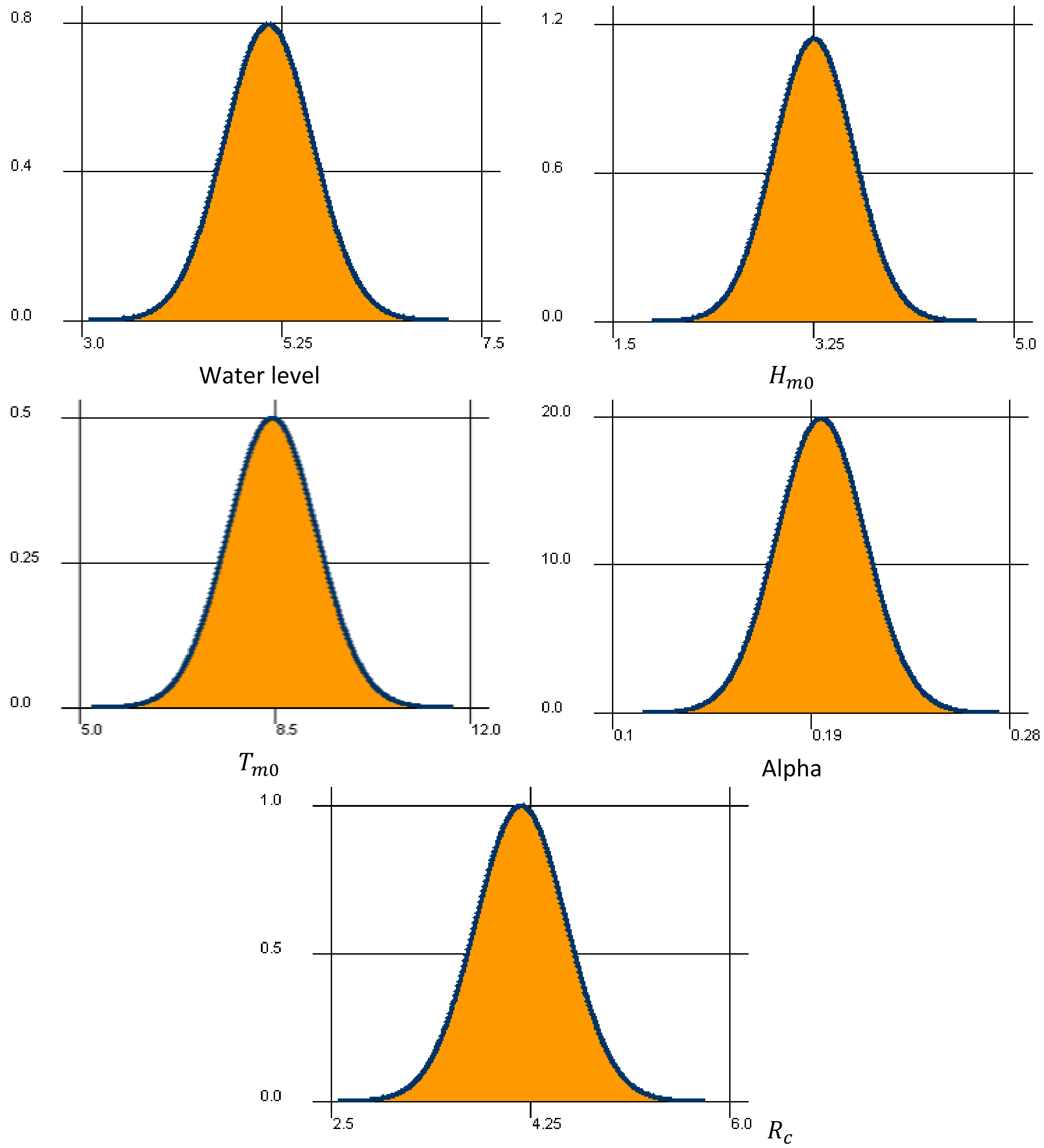


Figure 67: Distributions of stochastic variables

11.3.3 Water level functions approach

In this approach two scenarios were considered i.e. do-nothing vs robust design scenario. Failure probabilities of breakwater only and dike heightening only scenarios could be used to make a comparison between each other. However, in robust design scenario failure probability would always be lesser than these two scenarios. Therefore, only necessary comparison of do-nothing vs robust design scenarios was considered.

11.3.3.1 Water level distribution

The water level distribution governs this approach as wave height H_{m0}, wave period T_{m01}, average slope angle α and freeboard R_c are functions of the water level. The distribution as presented below was assumed to be normal and the statistical parameters were calculated as mean $\mu = 4.631$ and standard deviation $\sigma = 0.45$.

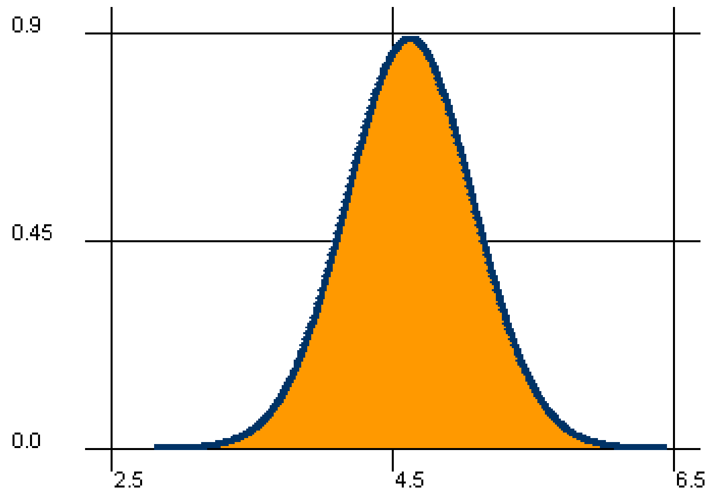


Figure 68: Water level distribution for failure probability calculation

11.3.3.2 Do nothing scenario

The following table represents the range of water level values used for preparing the dataset to obtain functions. Different water levels represent different real life possible situations which were first identified and then modelled in SWANOne. Offshore values resulted in nearshore values which were then used to calculate run up and overtopping through the MATLAB script. The data can be found in the Appendix F.

The data set is then plotted and by curve fitting techniques its functions are generated for different variables. These functions are then used in the model and are given in the plots below.

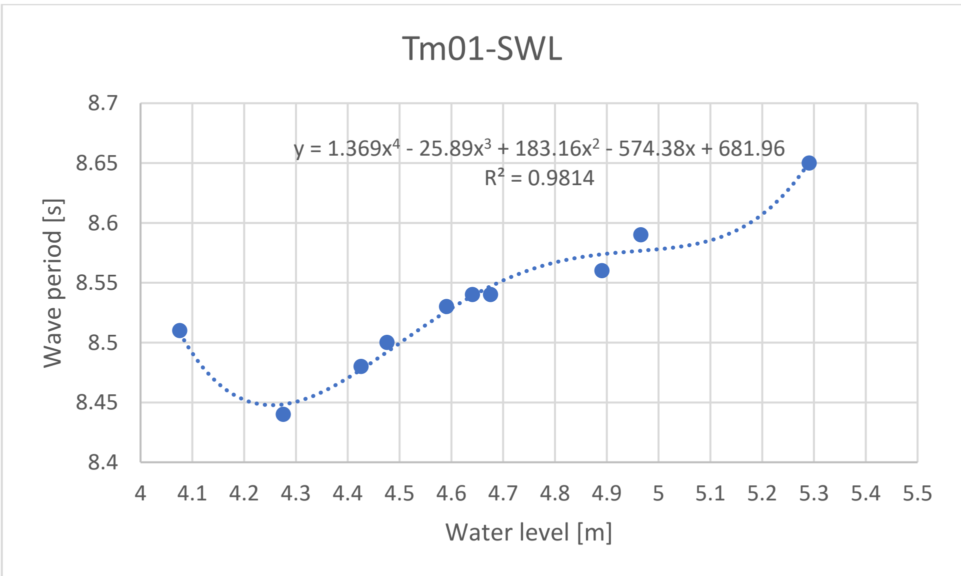


Figure 69: Wave period water level function - do nothing scenario

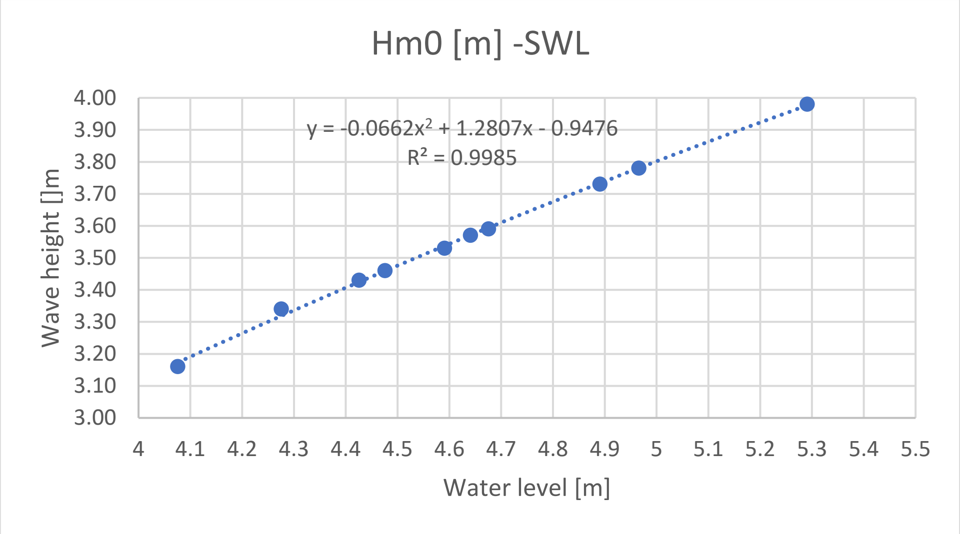


Figure 70: Wave height water level function - do nothing scenario

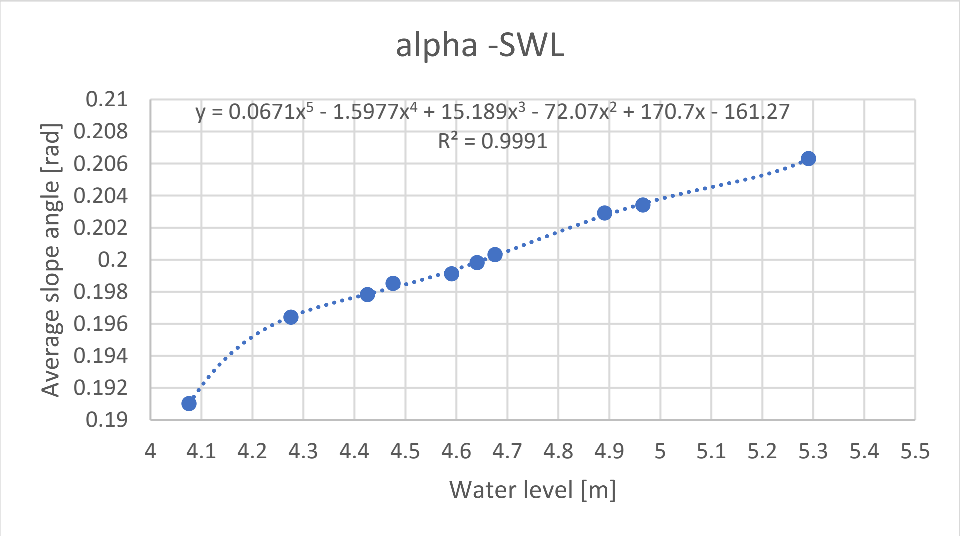


Figure 71: Average slope water level function - do nothing scenario

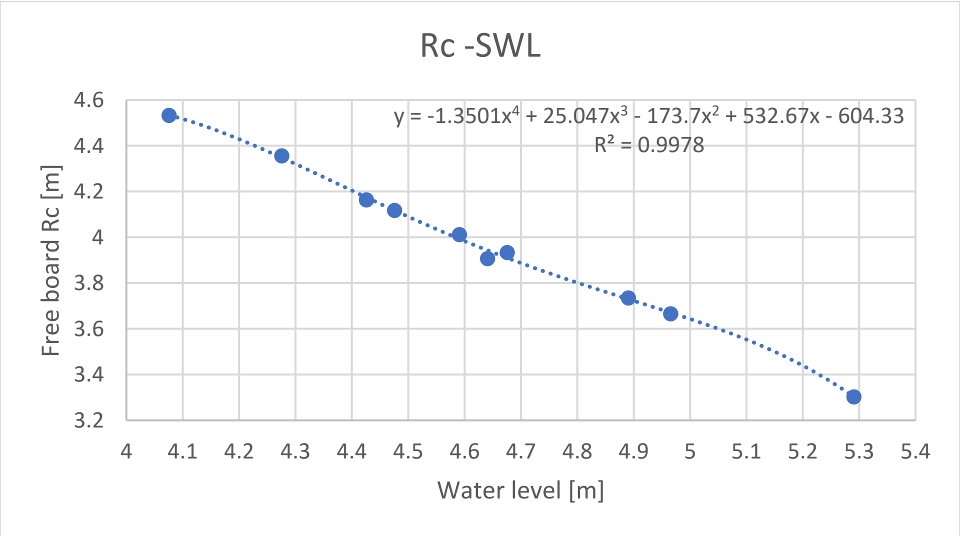


Figure 72: Freeboard water level function - do nothing scenario

11.3.3.3 Robust design scenario

A similar procedure, as was used for the do nothing scenario, was adopted but this time with both breakwater and heightening dike in place. The data can be found in Table F 2 in Appendix F and the functions are below.

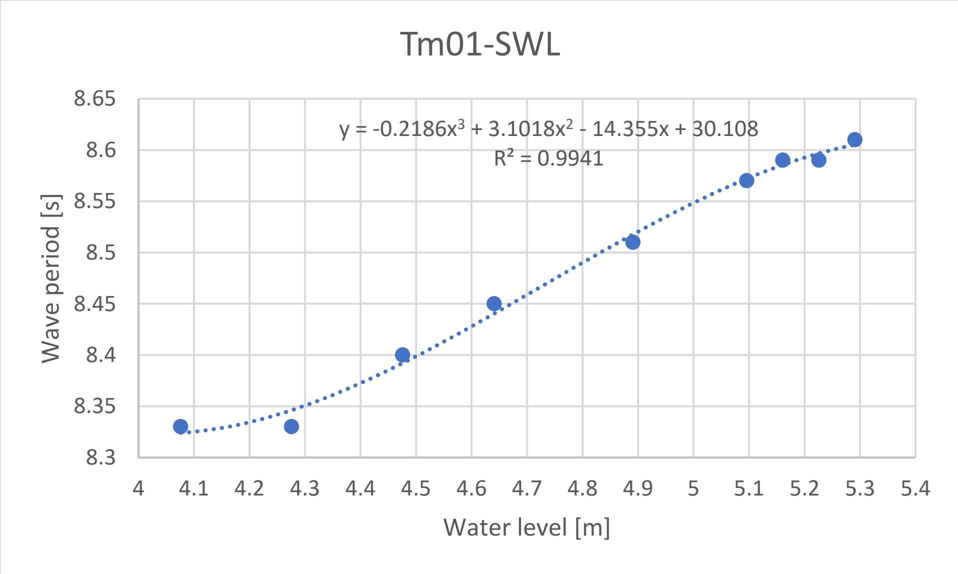


Figure 73: Wave period water level function - robust design scenario

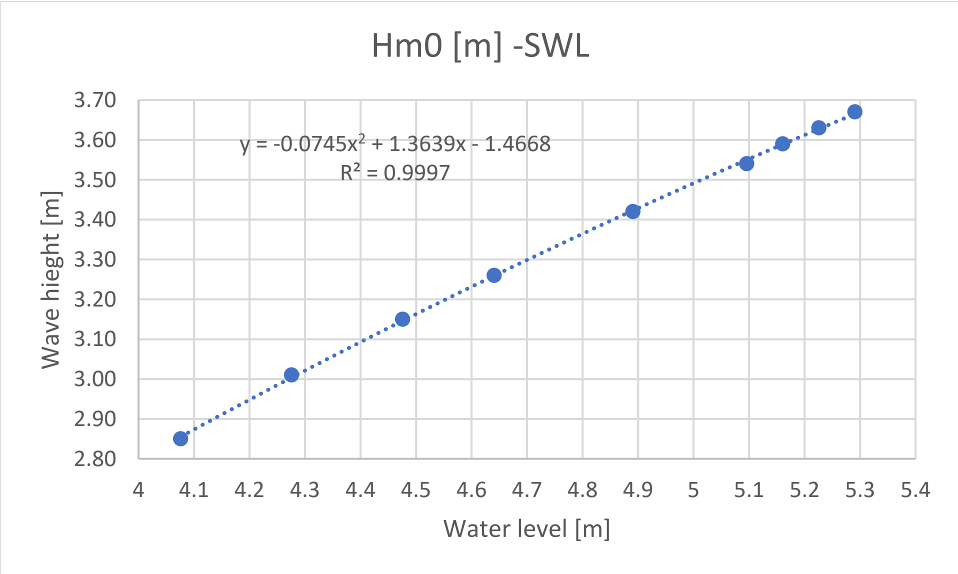


Figure 74: Wave height water level function - robust design scenario

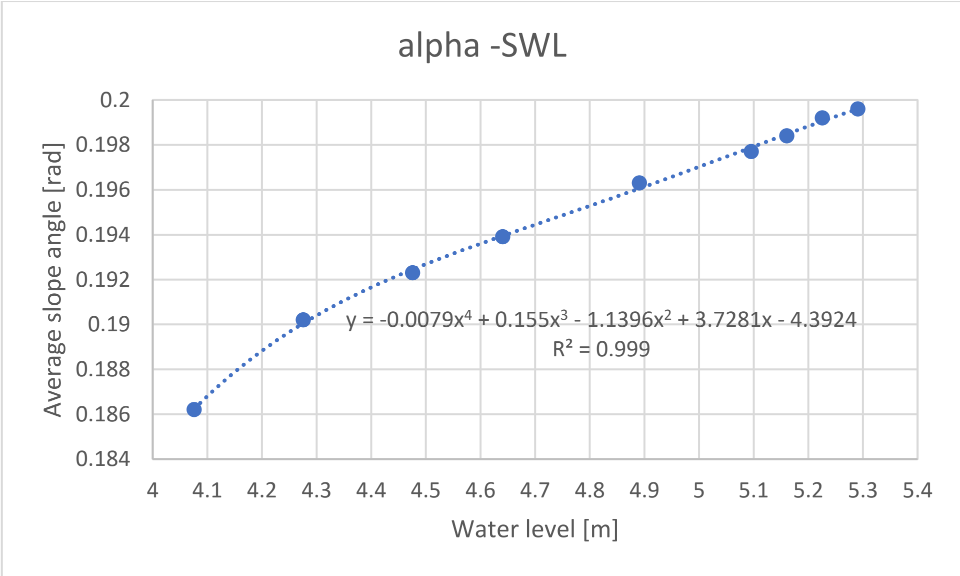


Figure 75: Average slope water level function - robust design scenario

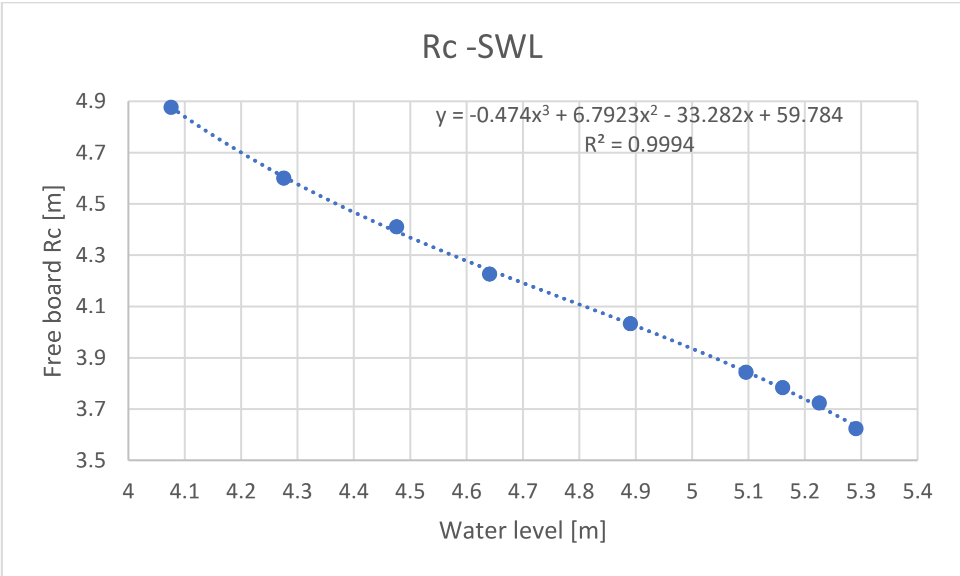


Figure 76: Freeboard water level function - robust design scenario

11.4 Results and Discussion

The failure probability model was implemented in Prob2B model which enables probabilistic analyses and to find the reliability of predictions. It gives results in the form of failure probabilities, alfa values, and reliability indices. During the simulations, if the limit state function Z was found to be negative, meaning the actual overtopping exceeded the critical overtopping discharge, then the respective iteration represented dike failure. The Prob2B program divided the number of dike failures by the total number of simulations to determine the total probability of failure of the dike section due to overtopping. All the calculations were done based on the FORM (First Order Reliability Method) analysis which is a Level II method.

11.4.1 Failure probabilities

11.4.1.1 Semi-deterministic approach

The results from the model for semi-deterministic approach are tabulated and plotted below.

Table 34: Failure probability results - semi probabilistic approach

Scenario	Do nothing (BN)	Breakwater (BW)	Dike Heightening (H)	Robust design (RD)
Current Extreme	0.0084	0.00018	0.0007	0.0000079
Climate Change (high bound)	1	1	1	0.975
Climate Change (low bound)	0.865	0.354	0.471	0.58

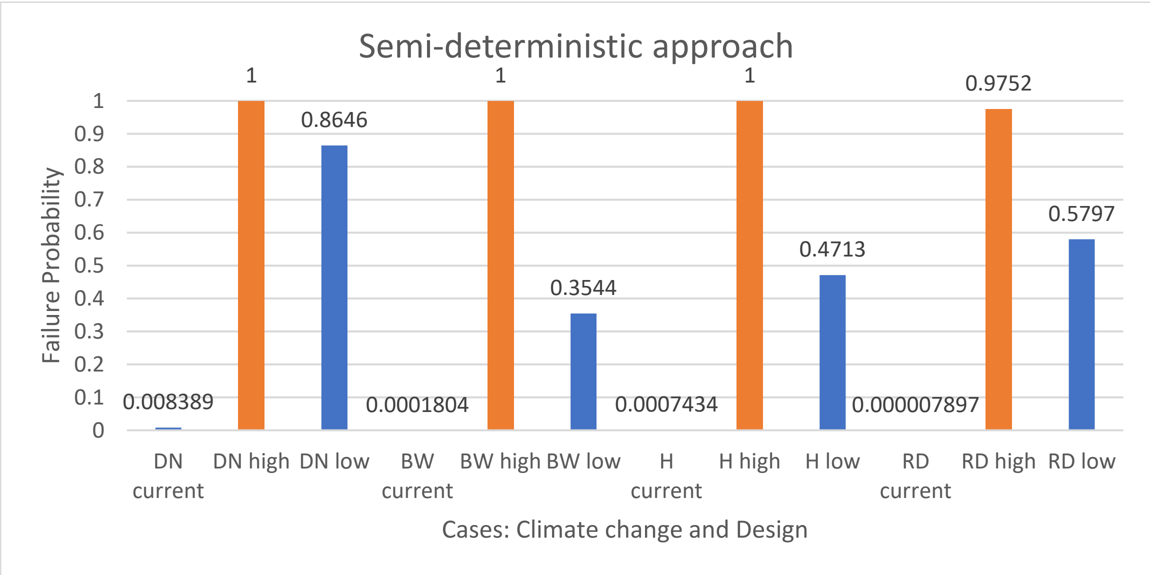


Figure 77: Failure probability bar chart - semi probabilistic approach

It can be observed from the results that failure probabilities are generally lower for the robust design as compared to do nothing scenario. For the lower climate change bound the robust design provided nearly half of the failure probability as compared to the do-nothing scenario.

For current climate conditions failure probability is decreasing when going from do-nothing to breakwater then dike heightening and the robust design scenario. This seems a reasonable outcome as more measures are meant to give more reduction in failure probability, if no climate change is taken into account.

For the high climate change bound all three design scenarios are failing except for the robust design approach. This depicts that even if the highest level of climate change occurs and sea level rises by 1.3m, the robust design would offer some resistance. The reduction in failure probability may not be grave but compared to other design approaches it does stress the benefits of the robust approach. If the robust approach were optimized further, it could also be used for higher bounds of climate change. Note, that the probabilistic calculations do not take the resilience aspects of the design into account. Including them, would decrease system risk even further.

11.4.1.2 Stochastic approach

By computing all the stochastic parameters with their corresponding distribution, Prob2B provided the probability of failure for a defined limit state function Z. The results of the probability of failure are summarized in the table below.

Table 35: Failure probability results - stochastic approach

Case	Failure probability
Do nothing	0.5638
Breakwater	0.4559
Dike Heightening	0.4012
Robust Design	0.3488

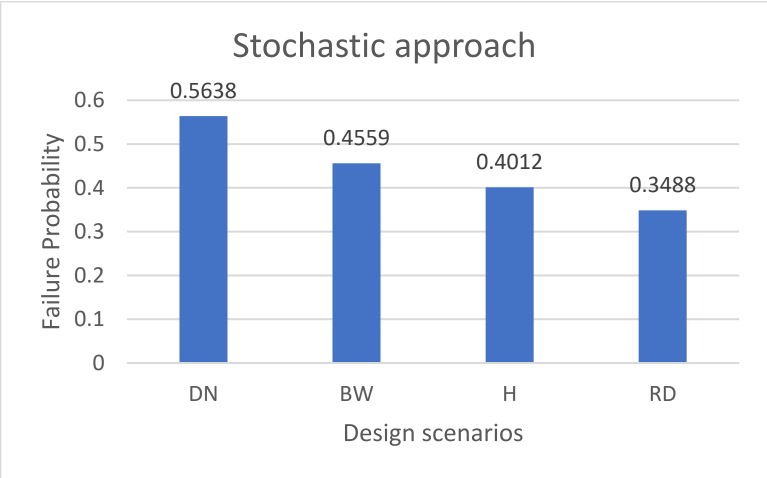


Figure 78: Failure probability bar chart - stochastic approach

One can observe from the figure that failure probability decreases from the do-nothing scenario to the scenarios where some counter measures are taken. Breakwater construction and dike heightening both reduced failure probability. The robust design approach has the minimum failure probability, which is very much in line with the hypothesis of the stochastic approach. The results show that the robust design approach is reliable to be applied as a flood defence approach in general.

11.4.1.3 Water level functions approach

Two overtopping discharges were used as a threshold because in this approach. Overtopping threshold discharge $q_t = 5$ is for the low boundary and $q_t = 30$ is for the higher bound of climate change. In high boundary we know that the system will surely fail with that threshold ($P_f = 1$) so we want to see the probability of failure for the threshold of 30 as well i.e. the threshold we have for the high bound.

Table 36: Failure probability results - water level functions approach

Scenarios	Do nothing	Robust design
$q_t = 5$ **[l/m/s]**	0.7821	0.2849
$q_t = 30$ **[l/m/s]**	0.3021	0.02562

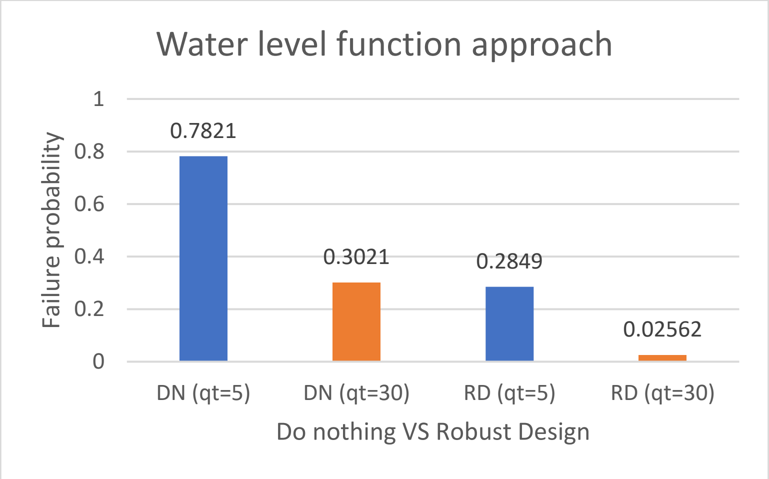


Figure 79: Failure probability bar chart - water level functions approach

The results are convincing as failure probability decreases when threshold discharge increases. This is analogous to increasing the resilience of the system because if we increase resilience of the system we can allow more water in the polder. When there is bigger threshold discharge, the capacity of the system to sustain extreme condition associated floods because of the overtopping also increases.

Furthermore, failure probability for robust design considerably decreases in comparison to the do-nothing scenario. This trend is observed for both threshold discharges but it turns out to have nonlinear relation with higher threshold discharges. One might infer that robust design approach works even better when higher threshold discharges are allowed. Higher threshold discharges can only be allowed if resilience of the system is increased which is the virtue of robust design approach.

In water level function approach, both of these findings of reduction in failure probability strengthen the idea presented in this project, including all assumptions and all steps taken to come to the results of the robust design approach.

11.4.2 $\alpha -$ value analysis

The Prob2B model outputs alpha (α) values, which are representative of how much each variable contributes to the probability of failure. These influence factors reveal important information about the sensitivity of the failure probability model to uncertainties in different variables. The bigger the absolute α-value the bigger is its influence and contribution to the uncertainty in the probability of failure. Therefore, the influence factors provided by Prob2B, give the relative importance of each variable on the final result. This allows the determination of the most important variables in each scenario and action plans, that directly target the respective variables, can be generated.

11.4.2.1 Stochastic approach

Alpha value analysis was carried for the stochastic approach because it had the most parameters (the parameters were input in the model as distributions). The alpha values for this approach are presented in Table F 3 in Appendix F and Figure 80 below.

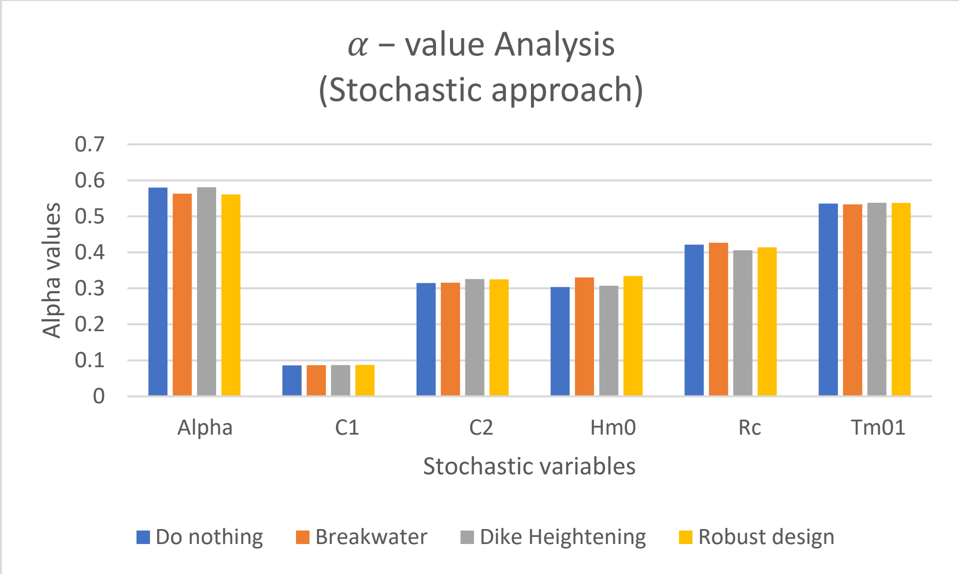


Figure 80: α- value analysis - stochastic approach

It can be observed that for all scenarios the average slope angle α has the biggest uncertainty contribution to the model. To improve the model, its uncertainty must be reduced. One way of doing so is reducing the standard deviation of it. It seems reasonable that the slope angle has the maximum uncertainty because it results from an iterative process as explained in the hydrodynamic analysis and sensitivity chapters. The iterative process increases its dependencies on other variables therefore adding to the overall uncertainty in the model.

The least important parameter was the C1 coefficient. It is low because of its very low standard deviation value as compared to the other parameters. Another reason could be that it was directly taken from the EurOtop manual, took a large amount of data into account to come up with this coefficient. The large amount of data makes in nearly an independent variable which doesn't contribute much to the uncertainty of the failure probability models.

11.4.2.2 Water level functions approach

Similarly, alpha value analysis was carried for the water level function approach as well. The table below shows the alpha values for four different cases. As expected water level has the maximum value and contributes most to the failure probability. This makes sense as most of the parameters, which had a considerable amount of uncertainty in the stochastic approach, are now functions of the water level. When a variable is entered in Prob2B as an expression, all of its uncertainty becomes dependent on the independent variable of the expression. Here the independent variable was the water level. The results are presented in Table F 4 and Figure 81 below.

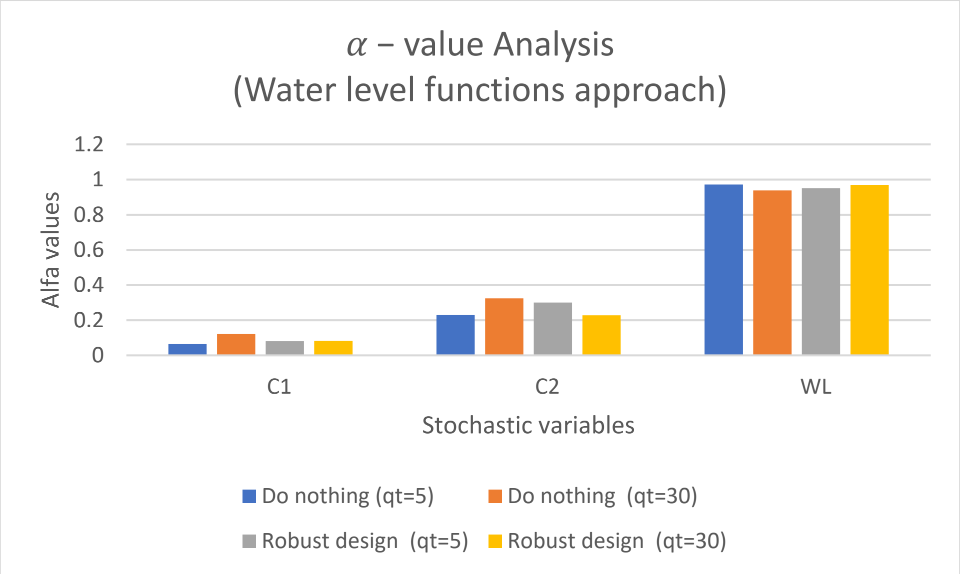


Figure 81: α- value analysis – water level functions approach

Chapter 12
Reference Case: Sand Nourishments

12 Reference Case: Sand Nourishments

The Water Board of Zeeland developed a coastal defence plan for Westkapelle in 2006. The plan includes the idea generation, decision-making process, and the preferred solution for the different regions along the coast. An illustration of the overview of the design can be seen in the figure below.

Figure 82: Province of Zeeland nourishment design (Sannen, 2006)

The yellow line represents areas where a nourishment is recommended, and the arrows detail if that is to be landwards or seawards. As seen in the image, at the cross section of consideration (within area 2b), a seaward nourishment is recommended.

12.1 Definition of Reference Case

The purpose behind the reference case design was high-safety and future spatial development of the coastal zone (Sannen, 2006). They split the region into different areas based on the shore type and created solutions accordingly. For the whole project area, the total amount of sand to be dumped along the shore is 2.45 million cubic meters, and of this, 0.25 million cubic meters are reserved for dune reinforcement. For the cross section of interest, within section 2b, the preferred solution was a seaward extension of the dune by 70m and a height increase of 2.5m, plus a foreshore nourishment to provide an extra 10m of dry beach (Sannen, 2006). The section is 400m long, and the total cost for this section is estimated to be €1.2 million (Sannen, 2006). To complete the construction, the sand will be transported from sea and sprayed to shore.

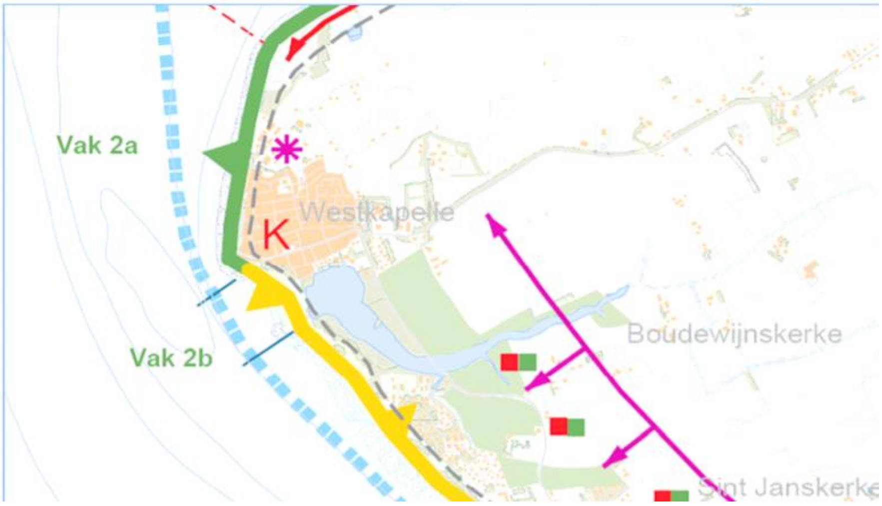


Figure 83: Section 2b, area of interest (Sannen, 2006)

The seaward nourishment is enticing for this region because first of all it matches the defined goals of increasing coastal safety and increasing spatial development, but also because of the unique wave conditions. For Dutch standards, this region can provide good surfing waves which was a factor in the design for creating a wider and more stable beach.

12.2 Cost Analysis

The total cost of the project has been reported to be about €26.9 million, €14.1 million dedicated to the sand nourishment. A plan of the Weak Links project details the expected cost for each section of the coast. Section 2b is the area that has been designed for and is being used for comparison. The report specifies that section 2b is 400m long, and the total cost to this section is expected to be €1.2 million (Sannen, 2006). To justify this cost, it is assumed that roughly 300,000 m^3 of sand is needed for the dune advancement and growth, and 25,000 m^3 of sand is needed for the foreshore nourishment. The sediment costs roughly €3/m^3, so therefore the cost of the sediment for this stretch is about €975,000. The remaining costs would include construction and land use costs. To put this cost in terms that can be comparable to the other design, it is equal to €3,000,000/ km. The maintenance of this zone is assumed to be about €100,000/km/year, as mentioned in a TU Delft paper as the approximate maintenance cost for all coastal defences in the Netherlands (Marten M Hillen et al., 2010).

The cost of beach nourishments has risen significantly in the last decade due to the amount of nourishment projects along the Dutch coast (M.M. Hillen et al., 2010). Therefore, it is safe to assume that maintenance costs will increase in the future as more nourishments will surely be needed along the coast to cope with sea level rise. Sand nourishments would have to be monitored to observe how much sand is leaving the system each year and how often maintenance nourishments would be needed.

Chapter 13
Design Comparison

13 Design Comparison

To quantify the effectiveness of the created design, it was compared to the reference case. The comparison is done by analyzing the costs, additional benefits, flexibility and adaptability, stakeholder satisfaction and performance under climate change scenarios. First, the aspects of the robust design will be discussed.

13.1 Dike Heightening, Living Breakwater and Surface Protection Design

Dike Heightening was selected to be part of the total design to reduce the expected amount of overtopping. With the consideration of sea level rise and more severe storms, it was calculated that the dikes in this region should be heightened by 0.35m to a total height of 9.35m above NAP. The living breakwater will have an integrated purpose that includes reducing coastal risk through decreasing exposure to wave action and enhancing habitat functions and values supporting local ecosystems including shellfish (mussels) through the creation and improvement of nearshore and coastal habitat. Lastly, grass is a common solution in protecting the landward slope because it is environmentally friendly and forms a beautiful landscape. The unique concept used for this design includes the deployment of a machine that cuts the grass at some distance below the surface and lifts the upper grass cover temporarily while placing a geosynthetic at the same time underneath (Akkerman et al., 2009). Below the effectiveness of this design is evaluated.

13.1.1 Cost

The cost of the dike heightening is roughly €11.54 M/km/m (Delta Committee, 2008). Since the dike is being heightened 0.35m and assuming linear extrapolation, the cost will be €4.04 M/km. This cost includes the land use, inner and outer slope costs as well. The cost of the living breakwater is €290,000/km plus €10,000 for a mobilization cost. The cost estimation is not easy because of the relatively small size of the breakwater compared to what is found in literature but this cost is assumed to be accurate. The construction cost for the reinforced grass cover is roughly €310/m (ComCoast, 2005). Including 40% for the contractor’s surcharges, the cost of strengthened sea defence will be about €280,500/km using the Geocell system (ComCoast, 2005). An amount of roughly €10,000 is to be added for mobilization costs.

The maintenance costs for the design are expected to be quite low. The yearly costs for management and maintenance for primary flood defences in the Netherlands is estimated to be approximately €100,000 per km per year (M.M. Hillen et al., 2010), but as mentioned in Section 10, the surface protection and the living breakwater should greatly reduce the wave damage to the dike. The estimated maintenance costs are therefore only €14,000/km/year. Regarding the grass cover, an annual budget of €1,000/year/km length is reserved for small repairs (ComCoast, 2005). The breakwater is also expected to receive very little damage, so little that the maintenance costs are negligible.

Therefore, the total cost of the design is roughly €4.6 M/km for the initial construction costs, and maintenance until 2100 is expected to cost roughly €1.23 M/km. The total cost until the year 2100 is estimated to be €5.86M/km and can be visualized in the pie chart below.

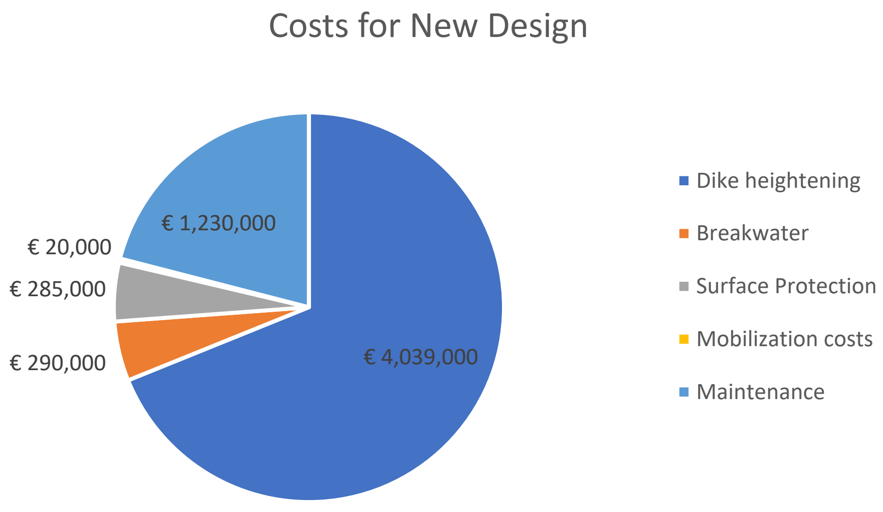


Figure 84: Costs for New Design until 2100

13.1.2 Additional Benefits

The additional benefits that dike heightening provides are very slim. It mainly serves its primary function, but the dike itself also serves as a bike and walk pathway. This was observed during the site investigation.

A Living Breakwater however has many additional benefits. It is a submerged rubble mound breakwater that is not only designed to dissipate destructive wave energy but also to host shellfish (mussels) and other maritime species through designed "reef street" micro-pockets as shown in Figure 85 (Parsons, 2013).

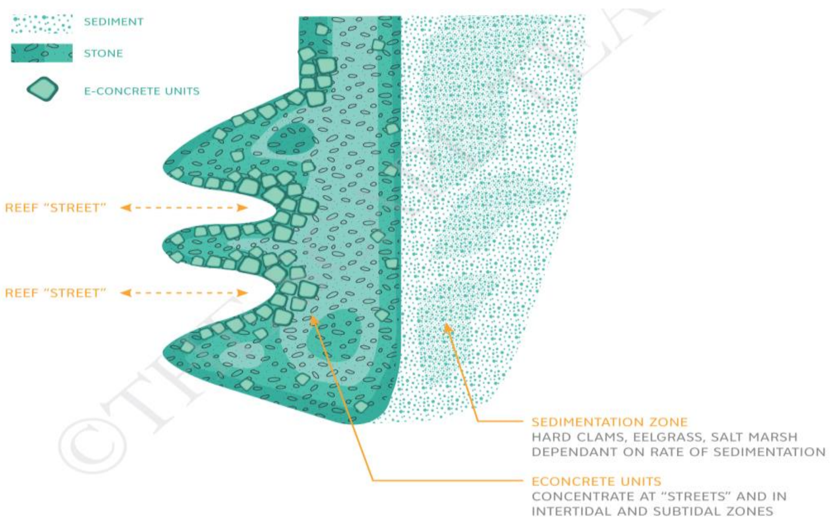


Figure 85: Artificial reef streets creating a habitable maritime ecosystem (Parsons Brinckerhoff, 2013)

ECOncrete® units (which are part of the design) are an innovative low pH concrete mix for maritime projects which are proven to increase complexity and habitat for a many species including shellfish (mussels). Above water, the breakwaters can host harbor seals and nesting birds, providing habitat free from predators (Parsons, 2013).

Further benefits include their ability to capture sediment, resulting in sedimentation in their rear which protects the coast from erosion. They also provide ecosystem services such as improving the water quality through filtration, they augment biodiversity as wave attenuation, substrate and water treatment have a positive effect on flora and fauna populations, and they provide income from shellfish farming and fishing (EcoShape, Deltares, & Bos).

Benefits of the SGR include that it is a significant reinforcement of the grass cover, highly cost effective, easy to install with minimum disturbance of the existing grass cover, hidden under the grass and thus is invisible, durable, and has no adverse environmental impacts. These benefits make the SGR a great option for dike protection. The most important additional benefit is that the landscape aesthetic is enhanced. A nice green grass cover is much more desirable than an asphalt cover that other dikes use. This will benefit the local inhabitants and the tourism industry.

13.1.3 Flexibility and Adaptability

Dike heightening is not a very flexible design. The construction process is quite disruptive and therefore isn't meant to be done frequently. That is why the heightening design was based on the climate change calculations until 2100. Therefore, this design has a design life of about 80 years. Of course, more construction can be done before then, and maintenance will have to be done, but the ability to adapt with this design option is limited.
The living breakwater is flexible because since it is a living system it will build up in parallel with the future sea level rise (Parsons, 2013). As the water level rises, the designed living system builds up biogenically (agglomeration of mussels and multiple other marine species on the face of the underwater breakwater (Parsons Brinckerhoff, 2013)) in parallel with the sea level rise. Thus, the breakwater is adapting to sea level rise while increasing its strength, stability, and longevity and reducing any maintenance costs.

The SGR is also a flexible design because it has a long lifetime, can withstand high levels of wave overtopping, and is easy to install. As mentioned earlier in the report, due to sea level rise the loading and the wave overtopping rates will increase substantially at this dike cross section. The hydrodynamic loading has been calculated for both the low and high climate change scenarios and SGR has been proven to be able to withstand the overtopping with an extra margin for the uncertainties within the climate change projection. Also, the product is said to have a lifetime of at least 50 years with no further maintenance of the reinforcement systems (ComCoast, 2005).

Since the major innovative aspect of the Smart Grass Reinforcement is in the installation concept, the risk during construction will be considerably less than in the present reference situation of raising the crest level (ComCoast, 2005). This is attributed to the short recovery time of the SGR (1 to 2 months) where the existing grass cover will not be removed and seeded but just lifted or penetrated. On the other hand, the recovery time of a newly seeded grass cover may last 5 years (ComCoast, 2005). Therefore, the SGR is adaptable because of the quick recovery time and flexible because of its high overtopping resilience.

13.1.4 Stakeholder Satisfaction

The main influence of dike heightening is to reduce the probability of flooding. This benefits the majority of involved stakeholders since it reduces damage and fatalities. Dike heightening is especially in the interest of the local inhabitants, farmers, and local business owners. Dike heightening may have a negative impact on the stakeholders concerned about the aesthetics of the environment, but it is only adding to what is already present so this impact would be minimal compared to something completely different.
The living breakwater is assumed to have mixed reviews by the stakeholders. It is great for the local aquaculture, environmental organizations and water board since it protects the coast from flooding and provides ecosystem benefits. The stakeholders who may find this aspect of the design slightly unpleasant are the local inhabitants and tourists because it interferes with the recreational beach space.

The newly adopted SGR would assure the safety of the Westkapelle polder satisfying the main stakeholder interest. In addition, the smart reinforcement system is proven to strongly reinforce the stability of the crest and inner slope, is highly cost-effective as compared to raising the dike crest and finally is a flexible application with minimum disturbance of existing grass revetment. These design criteria ensure that the surface protection has met all the requirements, guidelines and safety standards not only for Rijkwaterstaat and the waterboard but all other stakeholders involved.

13.1.5 Performance of Design

As mentioned in Chapter 10, the team's created design is capable of withstanding the lower bound of climate change predictions using the resistance measures, and the extra overtopping that occurs if the higher bound climate change occurs is dealt with by the resilience measures. This means the overtopping from a severe storm when SLR from the lower bound climate change results in only 4.77l/m/s and during a storm with SLR matching the upper bound climate change projection the overtopping severely increases to 26.77 l/m/s. A table outlining the performance of the design with both SLR projections can be seen below.

Table 37: Performance of team's created design

Breakwater and Dike Heightening	Ru2% [m]	q [l/s/m]
climate change high bound	5.58	26.77
climate change low bound	5.01	4.77

13.2 Reference Case

The reference case includes a seaward extension of the dune by 70m and a height increase of 2.5m, plus a foreshore nourishment to provide an extra 10m of dry beach. The cost, additional benefits, flexibility and adaptability and the satisfaction of the stakeholders regarding this design are discussed.

13.2.1 Cost

The total cost of this design is roughly €3 M/km. This includes the cost of sediment and the installation cost. The maintenance costs are assumed to be roughly €100,000/km but assumed to increase in the future due to the heavy demand of nourishments along the Dutch coast. Therefore, the cost until the year 2100 is estimated to be €11.2 M/km, and these costs can be seen in the pie chart below.

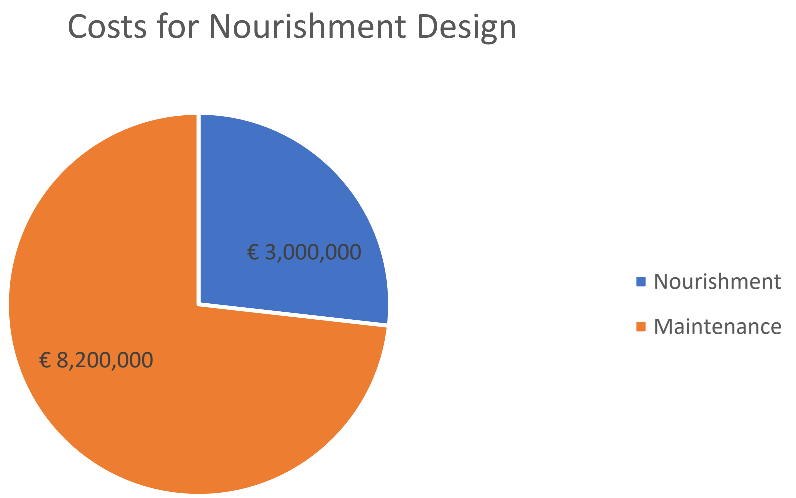


Figure 86: Costs for Nourishment Design until 2100

13.2.2 Additional benefits

Sand nourishments prove to come with a few additional benefits. These are mainly recreational, since they involve creating a greater beach area for locals and tourists to enjoy. This may then also benefit the tourism industry of the region if the beach becomes a destination for travelers. Another benefit is that when the sand is transported out of the region it was intended for due to coastal processes, it may end up further alongshore reinforcing other coastal areas. Although this does not provide a benefit to the intended location, it is interesting that even when the design fails (due to transport) another section of coast may find that beneficial.

13.2.3 Flexibility

Sand nourishments are meant to be a flexible design. Since they are dynamic they work with nature instead of trying to fight it. Regarding sea level rise, extra nourishments can be done to provide enough sediment to the coast to rise with the sea. This process is relatively simple compared to dike construction. Also, after monitoring how the sediment behaves along the coast, the next nourishments can be adjusted to have a greater impact in terms of flood protection.

13.2.4 Stakeholder Interests

Stakeholders will most likely be happy with this design. It seems to meet most interests of the stakeholders because it increases the safety of the system while not disturbing the current environment and not being overly costly. Local inhabitants can be safer and enjoy the recreational benefits that a nourishment provides. Nourishments are becoming a popular choice for coastal touch ups and so the waterboard wouldn’t find this design difficult to comprehend or approve. The main disadvantage is the ecosystem impacts that a nourishment can have. The seaward reinforcement will cause about 7 hectares of habitat to be permanently lost. The waterboard argues that in total this is a very limited loss since it is only about 0.1% of the total (Sannen, 2006). The deterioration is also not taking place in an area with exceptional environmental values such as shells banks. There are other protected habitats close to the project but they are said to not be affected (Zeeland Islands Water

Board, 2007). Therefore, the sand nourishment seems to meet the stakeholder requirements quite nicely.

13.2.5 Performance of Design

The performance of the nourishment design was evaluated by using a combination of Xbeach and SWANOne. First, Xbeach was used to determine how the profile would reshape during the 1/4000 year storm. This was done for the upper and lower bound SLR predictions. Then, the newly shaped profile was input into SWANOne to determine the wave heights at the toe of the dike, and the overtopping was calculated as previously done for the team's design. In Xbeach, the dike was set such that it cannot erode, and therefore all the changes to the bathymetry can be seen to occur below the MWL (0m elevation) where the toe of the dike ends and the nourishment would be applied. The nourishment was estimated to look something like the profile seen in Figure 87.

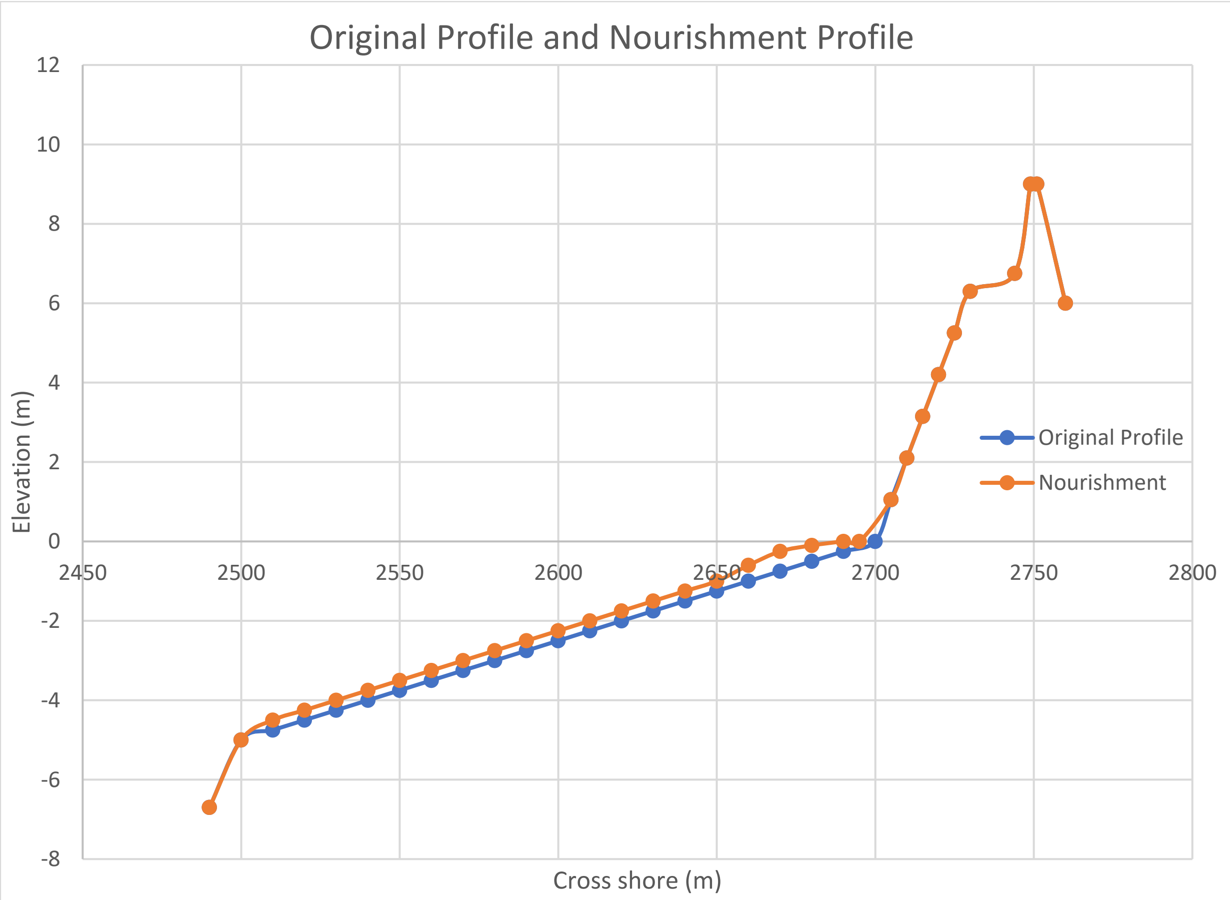


Figure 87: Nourishment profile compared to original profile

The XBeach analysis provided profile changes that match the expected outcome. The high waves from the severe storm would cause sediment to move away from the beach, further offshore, forming more of a shallow foreshore. The profile changes due to the upper bound and lower bound simulations can be seen below in Figure 88 and Figure 89. The profile changes from the upper and lower bound SLR are very similar. This is most likely due to the fact that during both scenarios, the water level was much higher than the nourishment, and therefore the waves would act similarly on the profile in both scenarios.

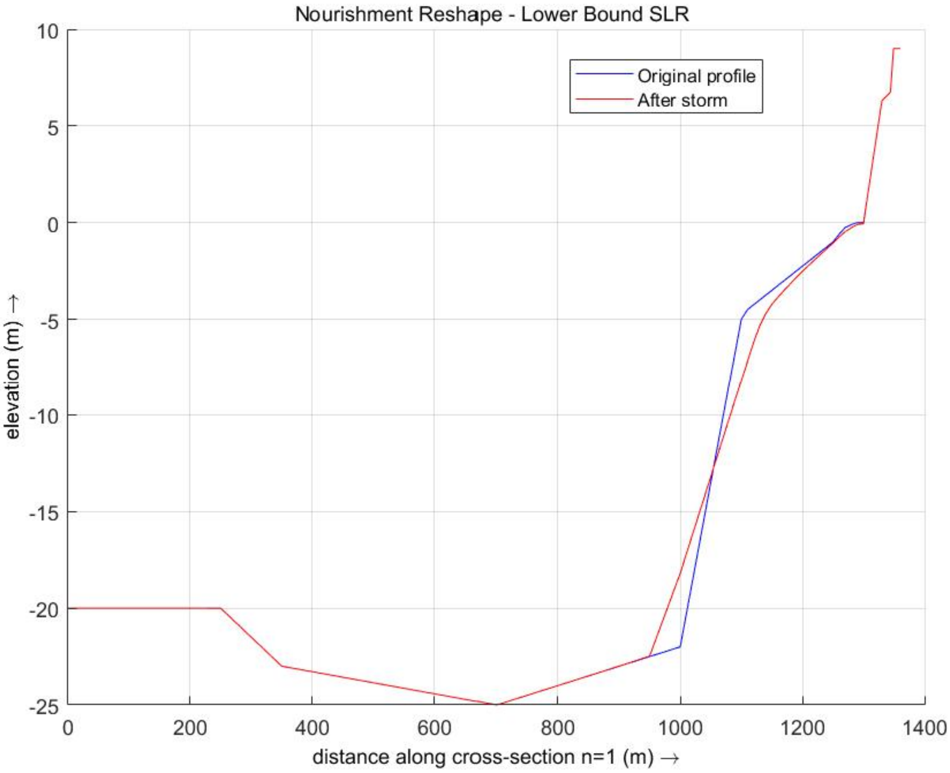


Figure 88: Nourishment reshape after storm - lower bound SLR

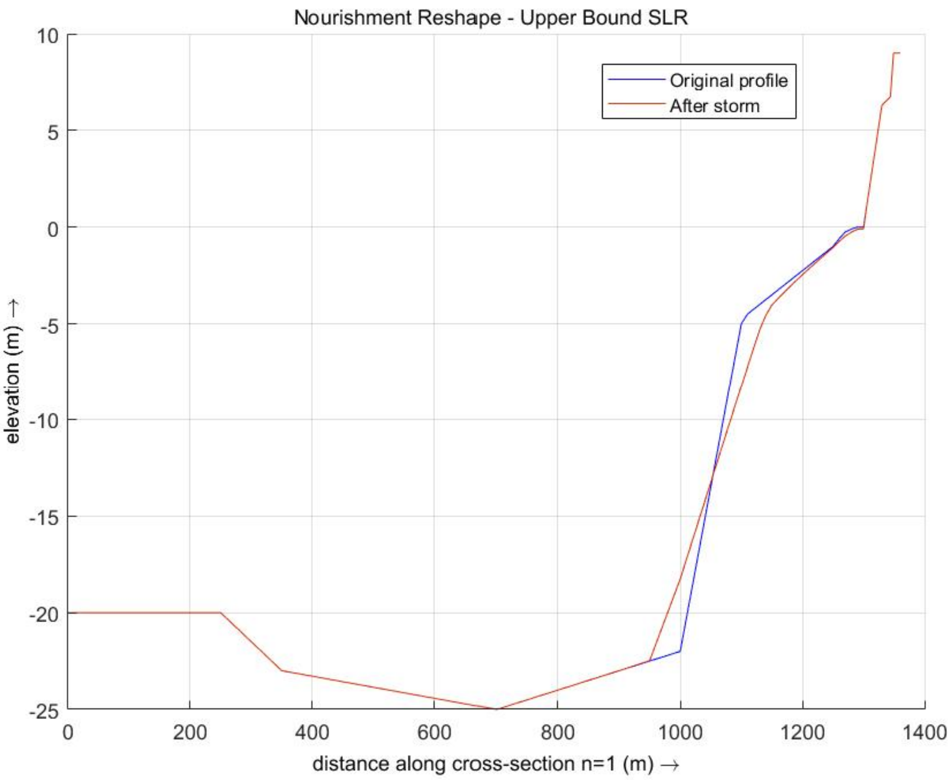


Figure 89: Nourishment reshape after storm - upper bound SLR

These profiles were then input into SWANOne for the wave height and overtopping calculations. The values obtained from this analysis are in Table 38.

Table 38: Performance of nourishment design

Nourishment design	Ru2% [m]	q [l/s/m]
Climate change high bound	5.7	44.64
Climate change low bound	5.09	8.67

13.3 Discussion

Overall, each design has advantages and disadvantages. Of course, the nourishment design was selected by the water board to be the actual design for the region and years of design experience make it a good option. It is relatively cheap, is proven to be successful along other areas of the coast, is very flexible, and satisfies stakeholder interests. The design that the team created is also proven to be an excellent option. It was created with robustness as the main driver and fulfills that criteria well. It also provides many additional benefits through the living breakwater and the grass cover, and is a stakeholder pleaser. Table 39 shows a chart comparing the two designs using the text from the above sections.

Table 39: Comparison of Designs

	Cost	Additional Benefits	Flexibility	Stakeholder Satisfaction	Performance
New Design	++	++	-	++	++
Nourishment Design	+	+	++	++	+

The flexibility is the only criteria for the new design that is not marked positively, and that is because it is a design that is already created with the future in mind and is therefore able to withstand a varying level of climate change scenarios.

In terms of the performance of each design, the team's design is expected to have less overtopping under a design storm with both the upper and lower SLR projections. Under the lower bound SLR the team's design gives about 4 l/m/s less than the nourishment, and under the higher bound SLR, the team's design gives roughly 18 l/m/s less overtopping. It is safe to assume that the amount of overtopping that the nourishment design allows under the higher bound SLR is far too much at 44.64 l/m/s, but that is where the flexibility of the design comes in, so that if the sea level is rising to the higher bound, which is a slow process, changes can be made to accommodate and ensure that the amount of overtopping is adequate. Nonetheless, the team's design is already capable to handling the upper and lower SLR scenarios and that is why it is marked higher than the nourishment design.

Another interesting aspect is the cost of the two designs. Although the cost of the nourishment is originally cheaper, the maintenance costs make it cost almost double the team's design by the year 2100. This is due to the robustness of the team's design which not only makes it less susceptible to damage, but also lowers the maintenance costs.

Overall, both designs can be seen to be good ones and satisfy their intended requirements well. The team's design accomplishes the task of flood safety and robustness well while the nourishment design also protects the coast and provides more area for spatial development. The team's design seems to be more appropriate for long term planning, and the nourishment seems to be the cheaper design that is preproperate for the shorter term and can be easily altered to accommodate the expected hydrodynamic loading.

Chapter **14**
Multicriteria Optimization

14 Multicriteria Optimization

Decision-making processes are dominated by limited resources and the unlimited want for them, the economic problem of scarcity. Most often the processes break down to a give and take between criteria and options. Negative externalities of one option are tried to be balanced out by the positive externalities of another option. Another concern in such situations are opportunity costs. Here the loss of benefits from one option is quantitatively studied in comparison with the additional benefits that are gained from another. This process goes on and on like a loop until an optimal point is reached. This chapter explains how the team proceeded in finding the optimal point of the robust design.

14.1 Linear Programming and Optimization

Flood risk problems include multiple criteria that must to be fulfilled at the same time. Therefore, an optimization study is required. Ideally, nonlinear optimization should be conducted to find an optimal point between design options, costs and reduction of flood risk. However, linear programming was adopted for this project because of the lack of detailed on site information and the multitude of design options (constructive measures).

In order to explain the general concept, an example of non-linear optimization is shown in the figure below. It relates the increasing investment for raising the dikes to the reduction in flood risk. The minimum of the total costs curve is the optimal point for decision makers to consider. Beyond this point there would be more investment and less reduction in flood risk. On the left side of the point, the flood risk would be too high and national safety standards violated. On the right side of the point, the design would overshoot the required safety standards, which is not cost-efficient.

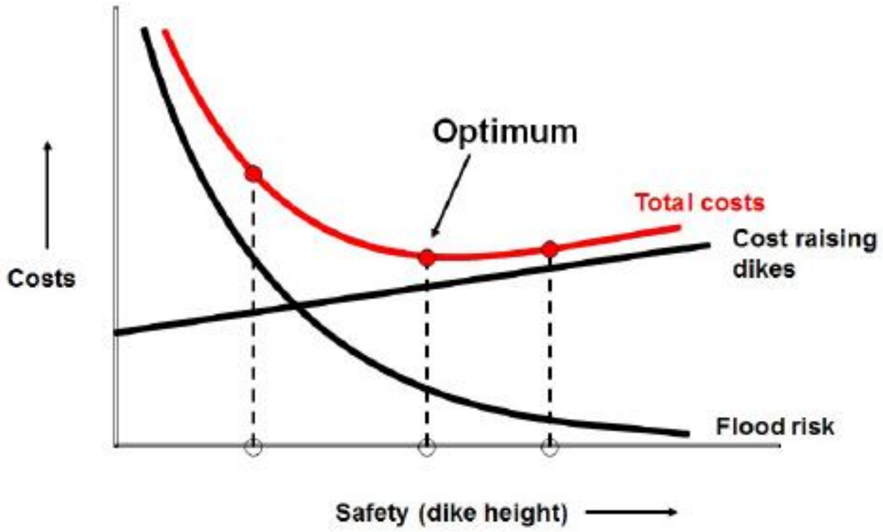


Figure 90: Example of Cost and Safety Optimization

Linear programming is a technique of optimization for simple situations. It is governed by decision variables, constraints and an objective function. It results in a feasible region and a frontier line, which shows optimal solutions.

Let X_i ; $i = 1,2,3 \ldots n$ be decision variables:

- Decision variables in this project are the design options of the robust design approach.

Let O be the objective function

- The objective function governs the entity that needs to be maximized or minimized. In this project the robustness of the design needs to be maximized.

Let C_i ; $i = 1,2,3, \ldots n$ be constraints

- Constraints are the physical barriers that the system cannot go beyond. They are the maximum and minimum values of design options and budget available for the project.

14.2 Decision variables

The decision variables that resulted from the MCA are a dike heightening and an offshore living breakwater.

$$X_1 = Dike\ heightening\ [m]$$

$$X_2 = Breakwater\ height\ [m]$$

14.3 Objective function

The aim of the process is to optimize the objective function by optimizing the proportions of design variables. This can be done by evaluating the reduction in overtopping via the design options under consideration. Reduction in overtopping is key because it increases safety of the system. The following equation shows the relation of design options to the required design overtopping discharge.

$$Q_{Design} = Q_{\frac{DH}{m}} * X_1 + Q_{\frac{BW}{m}} * X_2$$

Q_{Design} is the amount of overtopping that is allowed in the case of lower bound climate change. Unit reductions of overtopping discharges $Q_{\frac{DH}{m}}$ and $Q_{\frac{BW}{m}}$ were calculated based on the calculations in the section of hydrodynamic analysis and are presented in the section below.

14.3.1 Unit Reduction in Overtopping

14.3.1.1 Dike heightening

For dike heightening, different overtopping values were obtained for different hydrodynamic conditions. As can be seen in Table 40, only climate change scenarios are considered for the optimization study not the present situation.

Table 40: Overtopping values for different scenarios for dike heightening

	Heightening	0	0.35	0.5
Overtopping	High bound	43.63	26.77	21.61
	90%	35.97	21.82	17.52
	80%	31.18	18.73	14.98
	70%	26.42	15.68	12.48
	Low bound	8.66	4.77	3.67
	Average	29.172	17.554	14.052
	Reduction		11.618	15.12

Overtopping reduction is plotted against dike heightening. The curve is extended to obtain overtopping values for up to 1m of dike heightening and is presented in the figure below.

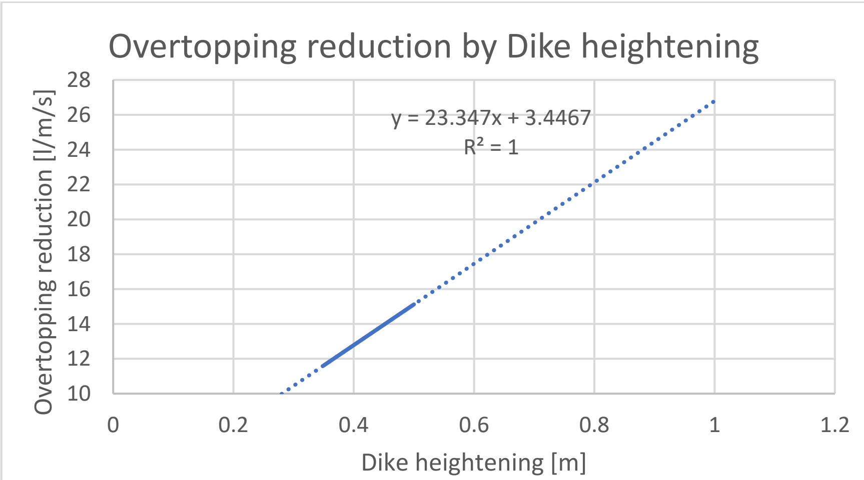


Figure 91: Overtopping reduction by Dike Heightening

It can be concluded from the figure that for every 1m of dike heightening, q reduces by 26.79 l/m/s. Therefore, $Q_{\frac{DH}{m}} = 26.79$ is input into the optimization model.

14.3.1.2 Breakwater

For the breakwater, overtopping was calculated for the case of a 2m high breakwater and the case of no breakwater. The table below summarizes the reduction in overtopping. The values are plotted to obtain the unit overtopping reduction by unit meter of breakwater freeboard.

Table 41: Overtopping values for different scenarios for breakwater

Scenario	**Without BW**	**With BW**	**Reduction**
Present	2.28	0.89	1.39
Climate change high bound	64.52	43.57	20.95
Climate change low bound	15.42	8.66	6.76
	Average reduction =		9.7

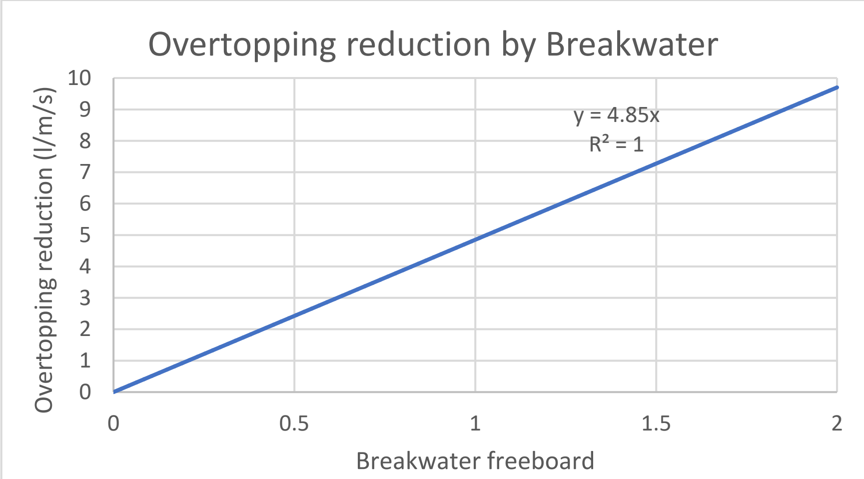


Figure 92: Overtopping reduction by breakwater

It can be concluded from the figure that for every 1m of breakwater, q reduces by 4.85 l/m/s Therefore, $Q_{\frac{BW}{m}} = 4.85$ could be used in the optimization model.

The optimization must focus on a single target overtopping discharge so that Q_{design} was taken as 18 l/m/s. The value was chosen because it is roughly in the middle of the lower bound and higher bound allowable overtopping values, that were chosen in chapter 9. It is assumed to represent the conditions in 2100 fairly well. The final equation reads as follows:

$$18 = 26.79 \times X_1 + 4.85 \times X_2$$

14.4 Constraints

Spatial and economic conditions introduce constraints for dike heightening, breakwater height and costs. The maximum limit for dike heightening and breakwater freeboard was set to 3m and the minimum limit was zero. Costs were controlled by the budget available for flood defence measures for the region of Westkapelle. The constraint functions are given below:

$$X_1 < 3$$

$$X_2 < 3$$

$$C_{\frac{DH}{m}} * X_1 + C_{\frac{BW}{m}} * X_2 < Budget_{Design}$$

where $Budget_{Design}$ is the amount of financing available to counter overtopping in the case of the lower bound of climate change, and $C_{\frac{DH}{m}}$ and $C_{\frac{BW}{m}}$ were determined from literature and are presented in the section below.

14.4.1 Unit costs

To obtain the unit cost for breakwaters, data was obtained from G. J. Vos (2016) and plotted. The results are shown in the figure below (J. Vos, 2016)

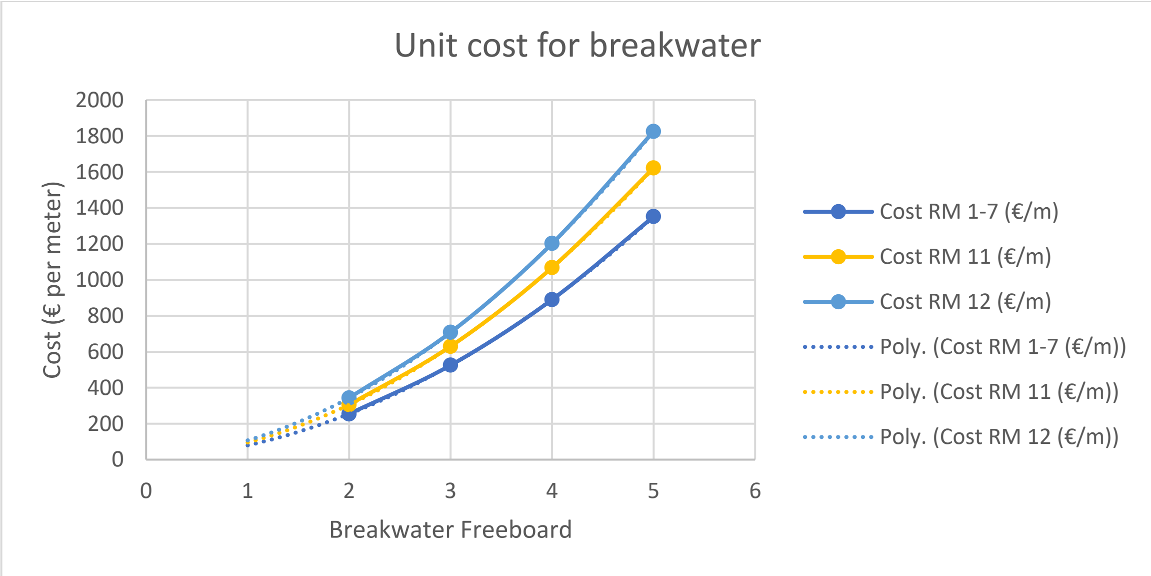


Figure 93: Unit cost for breakwater (Vos, J 2016)

As it can be seen that the material cost is around 170€/m of length and the labor and equipment costs around 20€/m of length adding up to 190.000€/m of length. The cost of dike heightening per meter is explained in chapter 8.3. The results are tabulated below.

Table 42: Unit costs for dike heightening and breakwater

Design Option	Unit	Cost
Dike heightening (rural)	M€/km/m	11.54
Breakwater	M€/km/m	0.19

The total budget available for the Delta program, as of 2007, was 743 M€. For Westkapelle, however, it was 3.9 M€. Therefore, $Budget_{Design} = 3.9\ M€$ (NL Government, 2009) and the cost equation reads as follows:

$$11.54 \times X_1 + 0.19 \times X_2 < 3.9$$

14.5 Results and Discussion

The functions and constraints were coded in MATLAB and the following plots were obtained. The first plot shows the complete design space including all limits of constraints for both decision variables. The blue region enclosed by all constraint functions shows the feasible region.

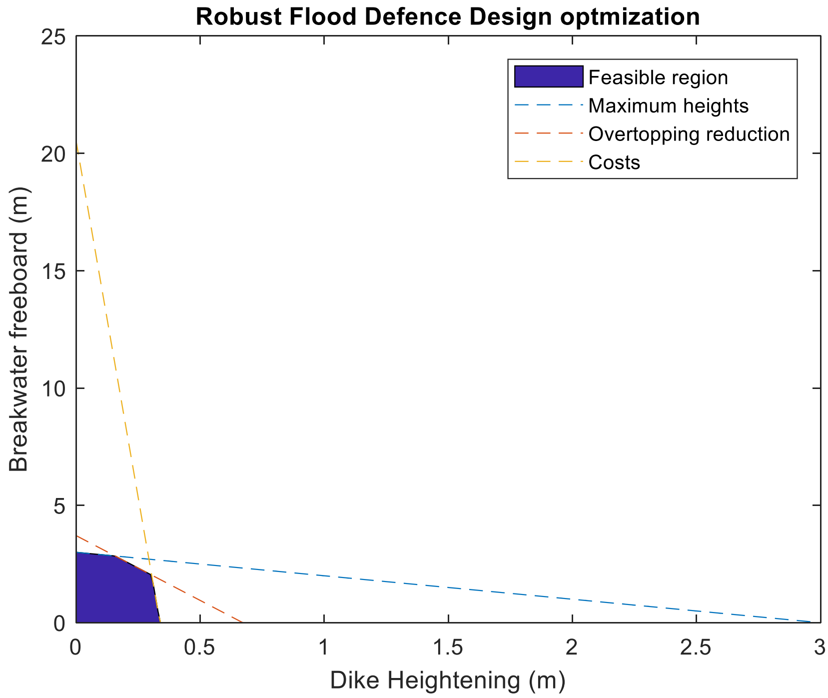


Figure 94: Multicriteria Optimized Design

The enlarged version of the plot is shown below, highlighting the feasible region. By looking at the feasible region it becomes visible that any combination of dike heightening and breakwater freeboard satisfies the design needs. In other words, the feasible region could be referred to as a robust design region. For an optimal solution, however, only those ratios of design options should be picked, which are on the frontier enclosing the region.

It is important to notice, that the frontier results from all three criteria. Furthermore, in different parts of the region different criteria become dominant. For a dike heightening lower than 0.15m, for instance the decision is dominated by the maximum heights limit. For the region between 0.15m to 0.3m of dike heightening, the overtopping reduction starts to govern the optimal solution. Beyond the dike heightening of 0.3m, the costs restrict the decision space. These relations are the true essence of a multicriteria optimization as the dominating criteria of each region can be identified clearly.

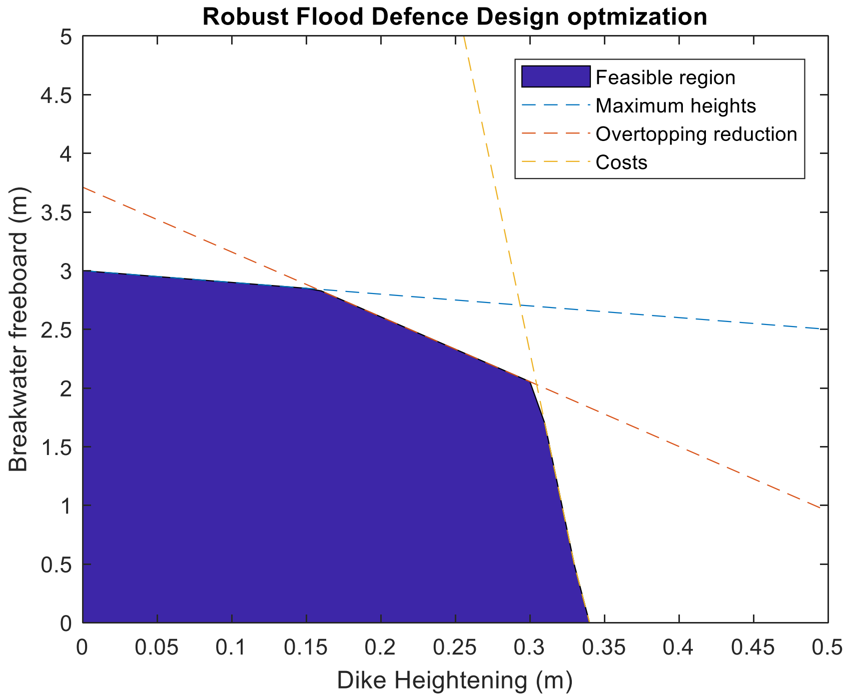


Figure 95: Multicriteria Optimized Design - Feasible region

As seen in chapter 10, a breakwater freeboard of 2m and dike heightening of 0.31m were calculated. The results are in line with the optimization study as this exact ratio exists on the optimized frontier. If that was not the case, another design iteration would need to be carried out.

Chapter **15** Conclusion

15 Conclusion

In this chapter the conclusions, that result from roughly four months of project work, are presented. The chapter is split in two, because conclusions regarding the team's methodology and conclusions regarding the recommendations for the case need to be drawn.

15.1 Regarding the Methodology

A robust design methodology for flood risk management was developed. It is robust in two ways. On the one hand, it makes use of measures that are inherently robust, e.g. the living breakwater that grows its own mussel armor. Furthermore, multiple options are combined to create one robust design approach – if one measure fails, there are still other measures that prohibit catastrophic failure. On the other hand, it puts uncertainties at the core of the design process. The design calculations do not assume single deterministic inputs but account for uncertainties. The design performs well under a variety of climate change scenarios without risking catastrophic failure or disproportionate damages.

A conventional design methodology strives to adopt a single loading scenario for design. It follows standards and experience and direct costs are mostly the decisive factor. For direct costs, tangible aspects of the design matter: e.g. a wall that keeps the water out is good because it prohibits any flooding. But what if the wall breaks or the water level exceeds the wall? A shortcoming of conventional designs is that they are narrow and neglect the system's resilience. A robust design, on the other hand, makes use of the system and its resilience. Flooding may be permitted, if it can be limited so that no substantial damages occur. The resulting additional benefits, which conventional designs lack, are various.

Firstly, socio-economic factors can be accounted for more precisely, e.g. by setting the allowable overtopping for a region according to the economic damages that occur if that overtopping were to happen. Secondly, stakeholder needs can be satisfied more effectively. This is because stakeholders are not being exposed to a decision between life and death, as in heightening the dike and being safe or saving the money for heightening and risking catastrophic failure and death, anymore. Instead, they are rather being exposed to the question "How often do you want to risk getting your feet wet and what are you willing to pay for it?". Thirdly, allowing certain levels of overtopping allows to cut down on the very costly constructive measures that provide resistance. Resilience mostly comes at a cheaper price and is effective in any extreme scenario. A flood wall, for example, might not be necessary in most of the smaller storms whereas it might be too low in a very extreme storm.

A robust design methodology may prove helpful beyond the realm of flood risk management. Climate change affects the loading conditions in many civil engineering disciplines, from wind loads in structural engineering to temperature boundary conditions in fire engineering, and uncertainties grow. Robustness was shown to be an effective tool in accounting for uncertainties and integrating them into the design process.

A number of tools were found to be essential for the methodology. Firstly, the Multi Criteria Analysis (MCA) enabled the team to gauge the robustness of design measures and select the most effective ones. Secondly, the SWANOne software was crucial for understanding the hydrodynamic loading and how effectively single measures are countering it. Thirdly, the MATLAB software simplified the overtopping calculation process significantly and allowed the execution of many design iterations in a short period of time. Fourthly, the Prob2B software proved to be a very helpful tool in comparing the effectiveness of constructive measures in terms of reduction in failure probability more profoundly. Fifthly, the XBeach software allowed an estimation of the conventional design's effectiveness so that a meaningful comparison with the robust design could be made. Lastly, the honest and direct group discussions were an essential tool at every stage of the project. Due to the large number of options, finding a design can be a very confusing process – especially so, when departing from the well-trodden path of conventional design and taking up new concepts like robustness. The team was lucky enough to feature various backgrounds and cultures. This made it possible to analyse problems from many different perspectives, to derive highly-differing solutions and to make reasonable decisions.

15.2 Regarding the Practical Recommendations for Westkapelle

The hydrodynamics of the region and climate change's impact on them were analyzed. High levels of uncertainty were identified and lead to high levels of uncertainty in the expected overtopping. Despite the uncertainties, it became clear that constructive measures are required to ensure safety of the region until 2100.

A combination of living breakwater, dike heightening, surface protection and two policy measures was identified to provide the required flood risk reduction. The proportions, that every measure takes in the combination, were optimized for robustness. The robust design was then compared to the conventional solution in a variety of aspects, notably stakeholder satisfaction amongst others, and came out slightly on top.

The design was generated by assuming that one cross-section is representative for all cross-sections, as all the given cross-sections were quite similar. Nevertheless, adjustment of the design parameters would be necessary to apply the design all along Westkapelle's waterfront. Similarly, it may be possible to apply the robust design in other regions along the Dutch coast as well. Furthermore, the robust design methodology may prove useful in other design projects regarding flood risk management in general.

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17 Appendices

17.1 Appendix A - Scheduling

Table A 1: Gant Chart Data

	Start	End	Duration	Milestones	
Research for Inception Report	12-Feb	20-Feb	9.0		Activity
Writing Inception Report	19-Feb	26-Feb	8.0		Activity
Inception Report Due	26-Feb	26-Feb	0.0	1.0	Milestone
Literature Review	12-Feb	26-Mar	43.0		Activity
Stakholder Analysis	27-Feb	03-Mar	5.0		Activity
Climate and Loading Analysis	27-Feb	13-Mar	15.0		Activity
Create MCA	06-Mar	13-Mar	8.0		Activity
Generation of Design Approaches	13-Mar	20-Mar	8.0		Activity
Site Visit	24-Mar	25-Mar	2.0		Activity
Grading of Design Approaches	20-Mar	27-Mar	8.0		Activity
Writing Interim Report/Presentation	27-Mar	09-Apr	14.0		Activity
Exam Break	09-Apr	20-Apr	12.0		Activity
Interim Report Due	16-Apr	16-Apr	0.0	1.0	Milestone
Interim Report Presentation	23-Apr	23-Apr	0.0	1.0	Milestone
Literature Review	23-Apr	07-May	15.0		Activity
Indepth Design of Chosen Approach	23-Apr	08-May	16.0		Activity
Develop Reference Case	08-May	15-May	8.0		Activity
Generalization of Approach	15-May	22-May	8.0		Activity
Writing Final Report	22-May	08-Jun	18.0		Activity
Final Report Due	08-Jun	08-Jun	0.0	1.0	Milestone
Creating Final Presentation	08-Jun	15-Jun	8.0		Activity
Final Presentation Due	15-Jun	15-Jun	0.0	1.0	Milestone

17.2 Appendix B- Hydrodynamic Loading

The following relates to the current hydraulic boundary conditions.

The Dike

In the next two Figures show the top view and cross-section of the selected dike section in Westkapelle. These design drawings were taken from Rijkswaterstaat Zeeland, Sea Defense Project Office.

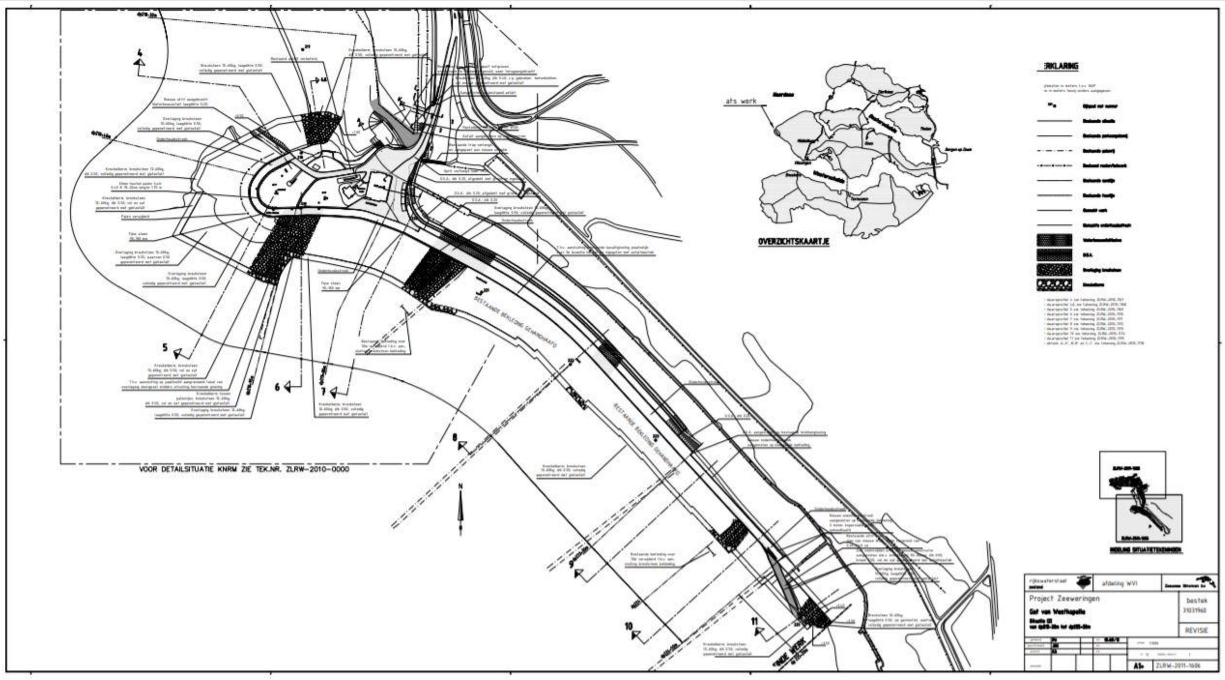


Figure B 2: Dike drawing plan view

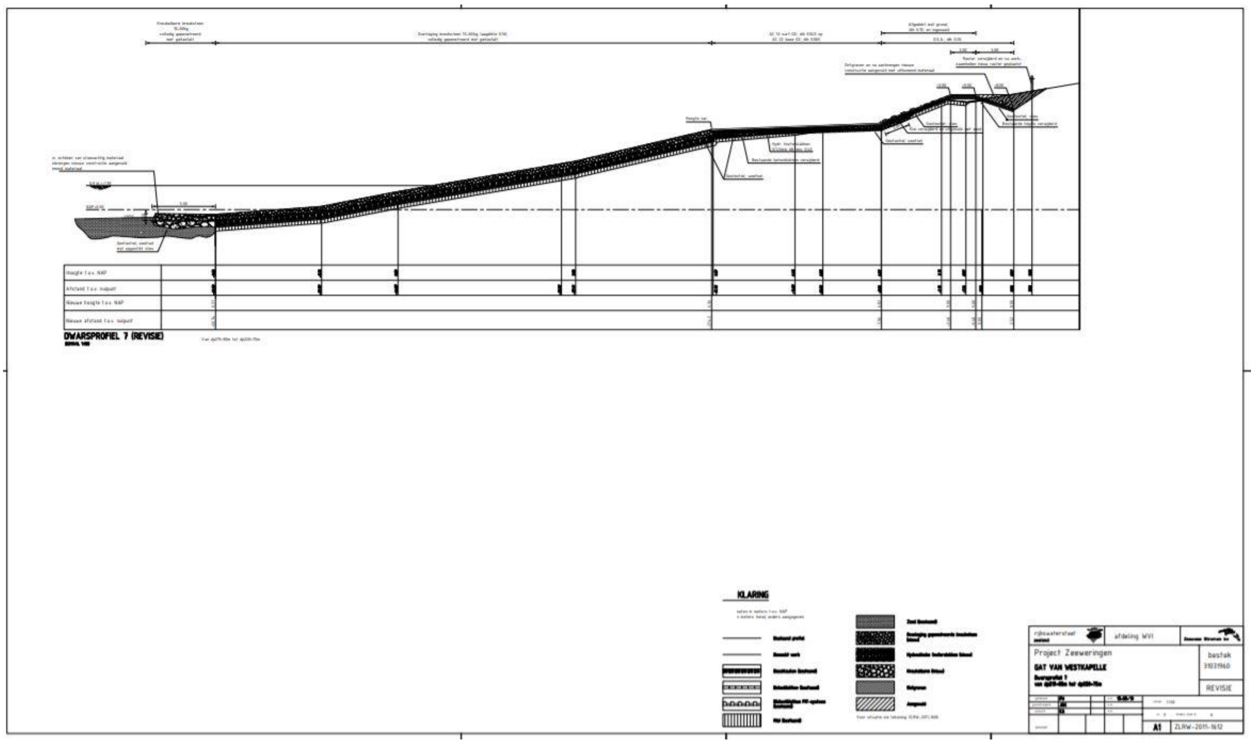


Figure B 1: Dike drawing section view

<u>Mean Sea Level</u>

The present mean sea level in the region around Westkapelle as acquired from PSMSL is around 2.63 cm above NAP. Specifically, as the mean sea level at our exact location was not readily available, two data points were selected. One in Roompot Buiten and one in Vlissingen as shown in Figure B 3.

Figure B 3: Data Points locations for a) Roompot Buiten and b) Vlissingen, (PSMSL, 2018)

For these two points we retrieve the following data charts (Figure B 4and Figure B 5) from which we can observe the annual mean sea level (in mm) through the years and take the value corresponding for 2016, which we can assume that roughly holds until the present day.

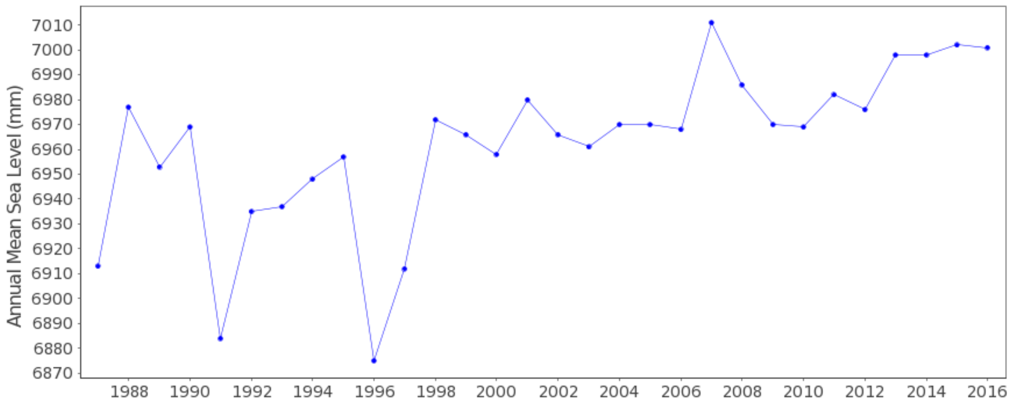


Figure B 4: Annual Mean Sea Level at Roompot Buiten, (PSMSL, 2018)

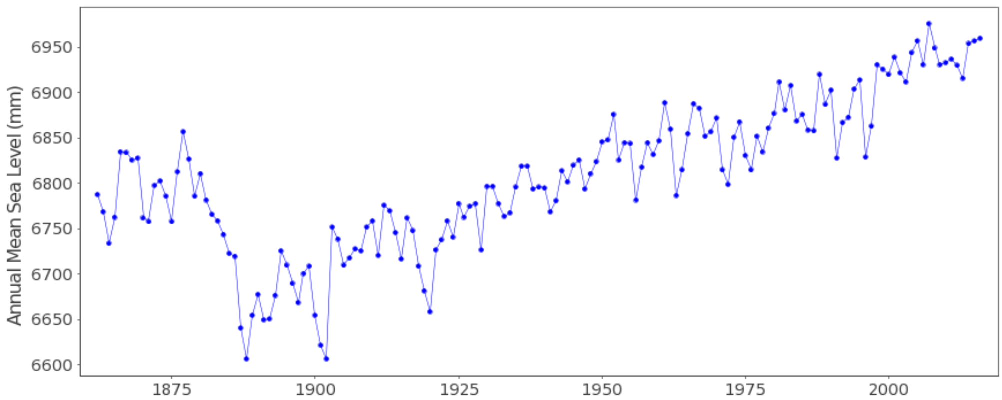


Figure B 5: Annual Mean Sea Level at Vlissingen, (PSMSL, 2018)

With the values from the graphs and by adjusting them for differences due to reference levels, we get for Roompot Buiten mean sea level at +6mm above NAP and at Vlissingen +35mm above NAP. As

Westkapelle is closer to Vlissingen than Roompot Buiten we roughly estimate the mean sea level at Westkapelle as:

$$\mathbf{MSL_{Westkapelle} = 0.7{\times}MSL_{Vlissingen} + 0.3{\times}MSL_{RoompotBuiten} = 26.3mm = +2.63cm\ above\ NAP}$$

Astronomical Tide

Although as mentioned previously, tidal movements are entirely predictable and tidal levels can be accurately predicted, sources provided quite various tidal ranges and amplitudes for the region of Westkapelle. According to Tide-Forecast, the largest known tidal range at Westkapelle is 4.67m (TIde-Forecast, 2018), which equals to a tidal amplitude of around 2.36m. In contrast, Van Santen and Steetze in (Van Santen et al., 2012) give a tidal wave amplitude of 1.7m. From the data webpage of Rijkswaterstaat an amplitude of around 2.35m is found which is more in line with the one for Tide-Forecast. On the other hand, we can find a tidal amplitude of around 2.05m in (Bosboom & Stive, 2015), which we consider more credible as more details on the data is available in comparison with the previous sources, thus this is the one chosen for our analysis. In addition, Giardino et al. (2014) found the tidal range decreasing from South to North from about 5 m to about 3 m (referring to the whole South-Westerly Delta). This range includes pretty much all values given by the previous sources.

Storm Surge

The maximum storm surge amplitude was given 1.25m by (Huisman & Luijendijk, 2009), and 2m (at Vlisingen which we assume to be the same in Westkapelle) by (Klein, 2015). The latter is chosen on the side of safety.

Wave Height

As discussed, our approach is to make our own calculations to determine the wave height at the toe of the structure. The first step was to obtain a timeseries of observations offshore of the structure. That was done with the help of the BMT Argoss wave climate tool. The dataset contains 25 years of data which is less than the selected return period. This means that the corresponding wave conditions cannot be read from the data directly and that statistical extrapolation of the data is needed. “An extrapolation is only statistically meaningful if the data is *homogeneous* and *independent*.” (Van den Bos & Verhagen, 2018). Independence can be safely assumed for a dataset consisting of storms. Homogeneity on the other hand requires that that the data all originate from similar meteorological events with varying magnitude. So, a dataset that consists of waves from different directions or both swell waves and locally generated wind waves cannot be used for extrapolation. Thus, the dataset must be turned into one or more homogeneous datasets before carrying out any further analysis.

Filtering of data

From Figure B 6 one can observe the main direction of the wave propagation from the dataset. The two main directions are 200-300 ºN (will be referred as 250 ºN) and 300 - 375 ºN (will be referred as 338 ºN). So, to analyze the data, the data is split into two datasets each one containing the waves propagating in these directions.

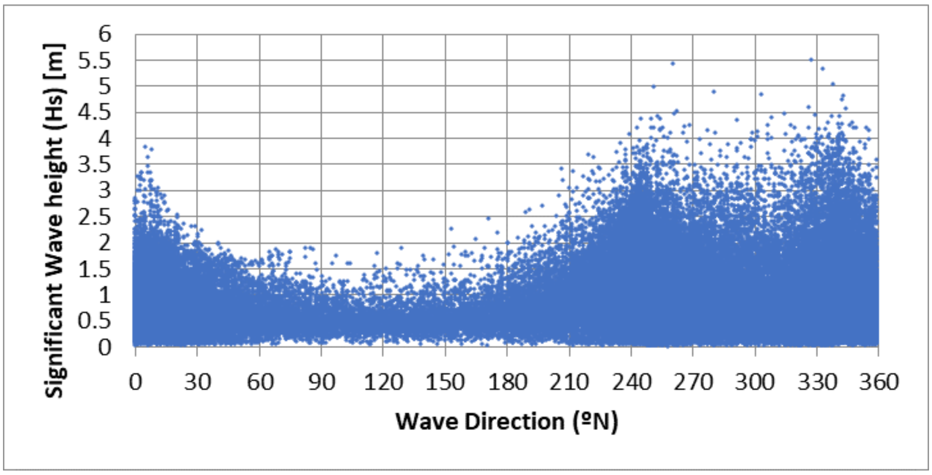


Figure B 6: Significant wave height to wave direction

To continue with the analysis, we need to have a dataset of storms, not just observations of the wave height at random moments in time. That is because several consecutive observations could be belong to the same storm. A storm should be counted only once in our statistical analysis as we are concerned of counting only one wave height during the peak of the storm, not the wave heights occurring before or after the peak. So, we must modify the dataset to transform the individual observations to storms and meet the criteria of statistical independence. This is done by means of a peak-over-threshold (PoT) analysis. We count as a storm the period in which the observed wave height is higher than a certain threshold. The time series is scanned time step by time step by a Matlab script, looking for periods in which the threshold is exceeded. Only the highest value (peaks) of the wave height during these periods of exceedance is recorded and all observations below the threshold are discarded. The number of storms per year, Ns follows directly from the remaining dataset by dividing the total number of storms N by the total number of years in the dataset (25). The result of the PoT analysis for the 250 ºN and 338 ºN datasets are shown in Table B 1, and now the criteria of statistical independency is fulfilled.

Table B 1: PoT analysis results

Dataset	Chosen threshold [m]	Number of storms per year [Ns]
250 ºN	2.7	10.56
338 ºN	2.4	10.36

According to J.W. Van der Meer et al. (2016), "Wave steepness is defined as the ratio of wave height to wavelength (e.g. $s_0 = H_{m0}/L_0$). Generally, a steepness of $s_0 = 0.01$ indicates a typical swell sea and a steepness of $s_0 = 0.04$ to 0.06 a typical wind sea." In Figure B 7 the significant wave heights are plotted to the mean period (starting dataset), as well as some steepness curves in order to distinguish between swell waves and wind waves. We can observe that some swell waves that do exist have low wave heights and are under the chosen thresholds, so they have been already discarded in the PoT analysis. Thus, also the criterion of homogeneity is fulfilled.

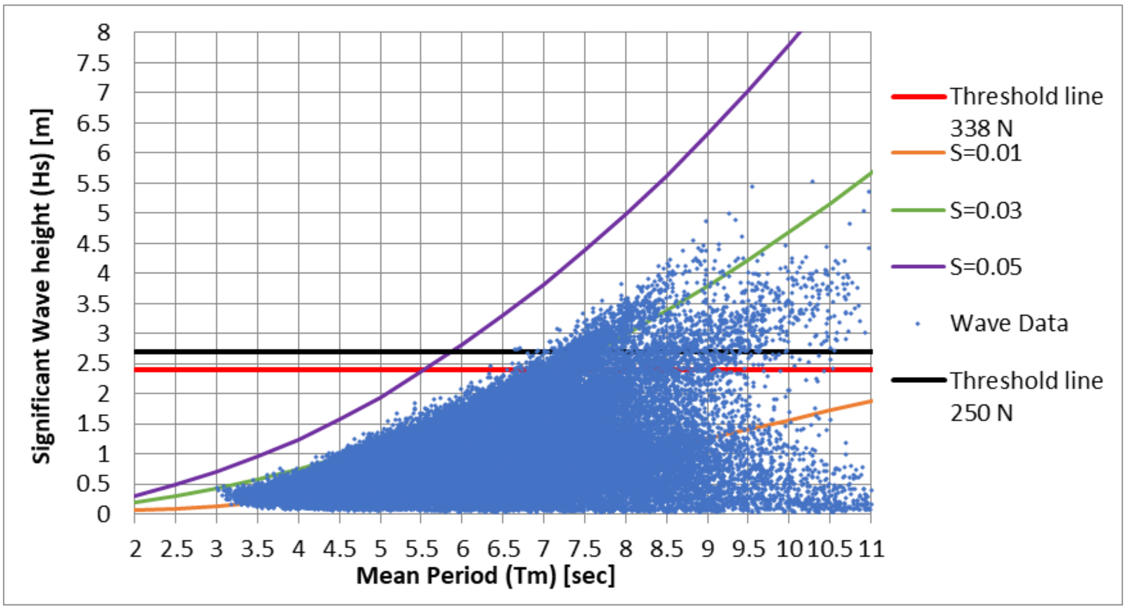


Figure B 7: Significant wave height to mean period

<u>Deep water significant wave height extrapolation</u>

The next step is the extrapolation of the significant wave height to the desired return period. We assume the data points follow a Weibull extreme value distribution which has the general form:

$$P = 1 - exp\left(-\left(\frac{H_{ss} - \gamma}{\beta}\right)^{\alpha}\right)$$

We can therefore fit the data to functions that correspond to the considered distribution. According to Van den Bos and Verhagen (2018), the non-exceedance probability P is defined as the probability that a given storm has a significant wave height that is smaller than (or equal to) a certain value. Mathematically, this corresponds to the definition of the cumulative distribution function F_H(Hs) of the extreme value distribution that we are considering.

$$\begin{aligned} P &= F_H(H_{ss}) = Pr\{H \leq H_{ss}\} \\ &= Pr\{\text{Wave height } H_{ss} \text{ is not exceeded } \textbf{per storm event}\} \end{aligned}$$

The exceedance probability Q is defined as the probability that a given storm has a significant wave height that is larger than a certain value.

$$\begin{aligned} Q &= 1 - P = Pr\{H > H_{ss}\} \\ &= Pr\{\text{Wave height } H_{ss} \text{ is exceeded } \textbf{per storm event}\} \end{aligned}$$

Or:

$$Q = \frac{1}{RN_s}$$

For a given location, all four parameters *P*, *Q*, Ns and *R* will be a function of *Hs*, in other words there will be values for these parameters for all possible wave heights. By combining the above, equations we get:

$$\frac{1}{RN_s} = exp\left(-\left(\frac{H_{ss}-\gamma}{\beta}\right)^{\alpha}\right)$$

If we plot $Hs_{,i}$ against Q_i we can fit the distribution and find the unknown parameters. This is done by means of a simple linear regression. However, the third parameter α, is selected by trying various values and selecting the one that give the smallest value of Root Mean Square Error (RMSE). Thus, the two resulting functions for the two datasets are:
For dataset 250 ºN:

$$\frac{1}{R \times Ns} = e^{-(\frac{Hs-2.6918}{0.5033})^{1.1}}$$

For dataset 338 ºN:

$$\frac{1}{R \times Ns} = e^{-(\frac{Hs-2.3359}{0.8091})^{1.36}}$$

Which for R=4000 and Ns= 10.56 and 10.36 respectively result in Hs= 7.02m and Hs= 7.23m. These two would have to be treated separately for the near- shore transformation and all the loading calculations, however as we want to be conservative with our calculations, we will use normal wave attack instead of oblique. Thus, we don't need two directions and we will assume the waves are propagating normal to the coast with deep water significant wave height **Hs= 7.23m.**

<u>Offshore-nearshore transformation</u>
This data is taken from a location in the vicinity of Walcheren coast, in deeper water further offshore from the considered dike cross-section. This means that an offshore-nearshore transformation is needed to obtain information at the toe of the structure (needed for our calculations), as to account for the wave dissipation due to the interaction between the waves and the bathymetry with processes like diffraction, refraction, shoaling, dispersion and wave breaking. This is done with the one-dimensional computation model SwanOne. In a more detailed analysis a two-dimensional computation model would be required but this is out of the scope of this project.

The model requires the following inputs:

1. *Bottom Profile*

The bathymetry greatly affects the propagation of the waves. As the depth becomes smaller the waves start to "feel" the bottom changing direction and increasing their height until, when the depth becomes too small for them to sustain their form, they eventually break greatly dissipating energy. For the model we use the bathymetry line used by Van Santen et al. (2012). In Figure B 8 the area considered by them is shown and in Figure B 9 one can see the bathymetry that they used in the one-dimensional approach of their modeling and which we adopt for our model runs.

Figure B 8: Bathymetry area considered by Van Santen et. al.

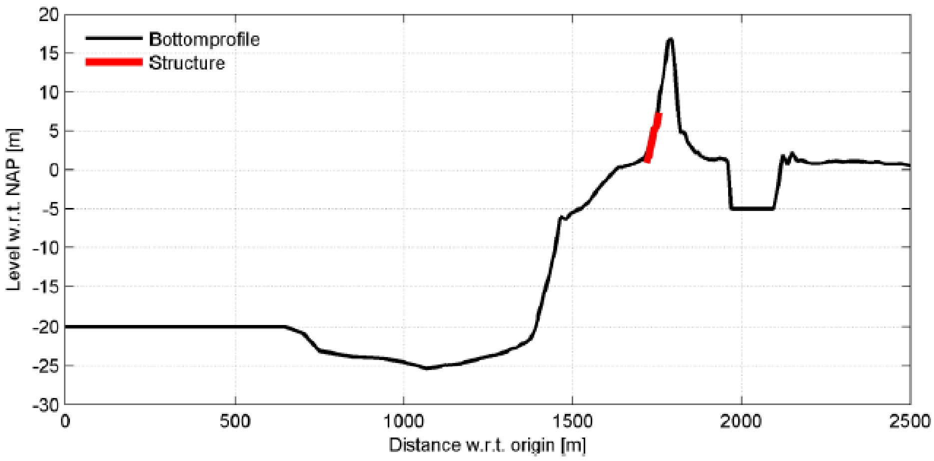


Figure B 9: 1-D Bathymetry considered for modelling

2. *Water Level*

The water level input refers to the existing water level above NAP (considered zero for the bathymetry and the model) The inputs for each case are:

- 0.0263m for the case of no extreme weather conditions (no storm surge or tide)
- 4.076m for the case of extreme weather conditions in the present situation (present water level (0.0263m) + tide (2.05m) + storm surge (2m))
- 5.291m for the case of extreme weather conditions in the future, including sea level rise due to climate change (preset extreme scenario water level (4.076m) + sea level rise (1.215m))

3. *Wave Set-up*

Wave set-up is localized near to the shoreline. It is mainly caused by energy dissipation caused by depth-induced breaking of the incoming waves (CIRIA et al., 2007). It causes an increase in water levels within the surf zone at a particular site due to waves breaking as they travel shoreward (J.W. Van der Meer et al., 2016). We chose to enable the option for the model to calculate and include wave set-up. In addition, the resulting calculated set-up is used as an added water level for the overtopping calculations.

4. Wind Velocity

The wind velocity affects the propagation of the waves, as a strong wind in the direction of the propagation can add to the wave energy or one on the opposite direction can counter them dissipating some of their energy. In order to calculate the wind that corresponds to each scenario of deep water wave height we plot the significant wave height data to the wind velocities that corresponds to them, from the dataset that only had the storms (after PoT), and make a trend line as shown in Figure B 10:

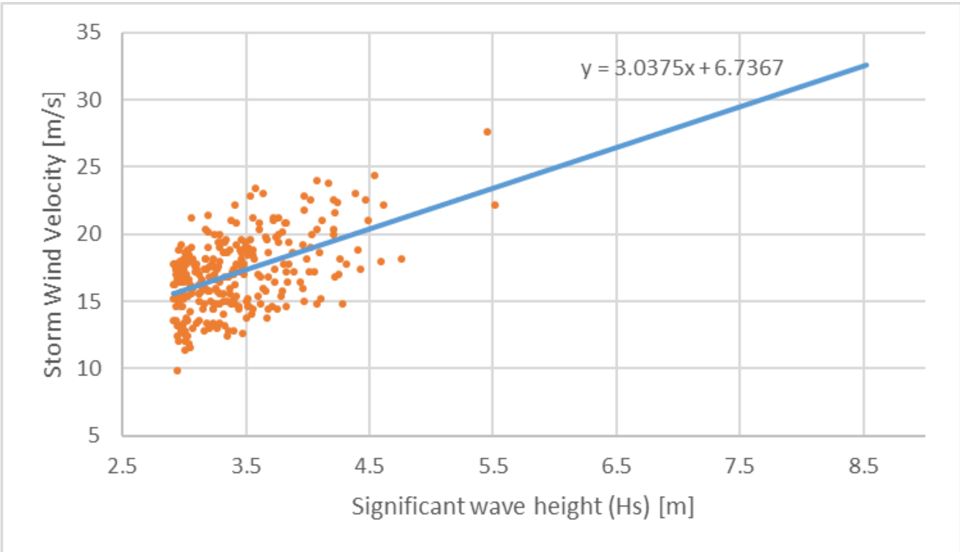


Figure B 10: Wind velocity to significant wave heights in storms

From the graph we get the equation:

$$U_{10} = 3.0375 \times Hs + 6.7367$$

We assume that this equation connects all significant wave heights with respective velocities (although that might not be always the case) and thus calculate the wind velocities for the extrapolated wave heights.

So, for Hs = 7.23m we get U_{10}= 28.7 m/s and for Hs = 8.53 we get U_{10}= 32.65 m/s

5. Wind Direction

As just mentioned the direction of the wind plays an important role for the wave height estimation. In our case we assume that the winds have the same direction with the waves as to a worst-case scenario.

6. *Currents*

As discussed previously, currents are not included in this analysis, thus considered zero. By running the computational model for the three scenarios we get the following Table B 2 with the results.

Table B 2: Results from computational model SwanOne

		Offshore			Nearshore			
Water Level		**Hs [m]**	**Tp [sec]**	**U_{10} [m/s]**	**Wave set-up [m]**	**$H_{m0design}$ [m]**	**Tp [sec]**	**$T_{m-1,0}$ [sec]**
Normal condition WL (present)	0.0263	7.23	12.2	28.7	0.784	0.68	12.2	7.75
Extreme condition WL (present)	4.076	7.23	12.2	28.7	0.392	3.16	12.2	8.51
Extreme condition WL (climate change)	5.291	8.53	12.2	32.65	0.407	3.98	12.2	8.65

<u>Run-up and Overtopping</u>

As discussed for the calculation of run up and overtopping we use the method and formulas presented by J.W. Van der Meer et al. (2016). As seen we use the formulas for run-up and overtopping used for design and assessment of relative gentle slope dikes.

The wave run up calculation is being done using the following formulas:

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \times \gamma_b \times \gamma_f \times \gamma_\beta \times \xi_{m-1,0}$$

Where:

- $R_{u2\%}$ is the run-up level exceeded by 2% of incident waves [m]
- H_{m0} is the spectral wave height at the toe of the structure [m]
- γ_b is the influence factor for a berm [-]
- γ_f is the influence factor for the permeability and roughness of the slope [-]
- γ_β is influence factor for oblique wave attack [-]
- $\xi_{m-1,0}$ is the breaker parameter based on $S_{m-1,0} = \frac{2\times\pi\times Hm0}{g\times T_{m-1,0}^2}$ [-], $\xi_{m-1,0}$ is calculated as $\frac{tan\alpha}{\sqrt{S_{m-1,0}}}$ (α is the slope of the structure)

One can see a sketch of the basic variables in Figure B 11.

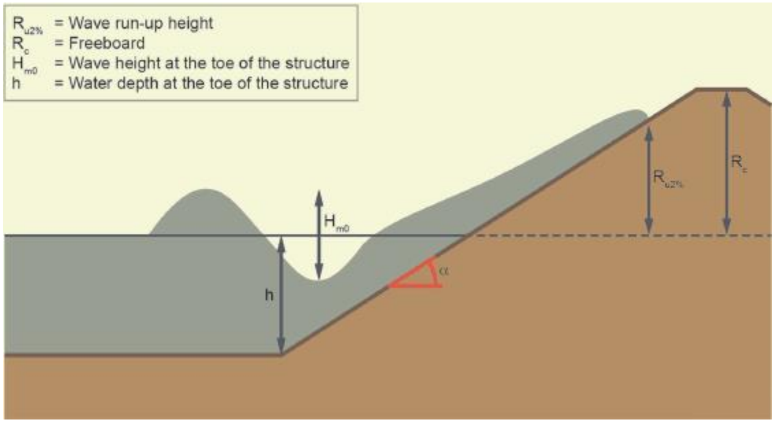


Figure B 11: Definition of the wave run-up height Ru2% on a smooth impermeable slope, (J.W. Van der Meer et al., 2016)

In the used equations, the factor γ_β which is the influence factor for oblique wave attack is taken as 1, as only normal waves to the structures are considered. The factor γ_f that is the influence factor for the permeability and roughness of the slope is roughly estimated and assumed to be 0.85 as the examined cross-section is covered in rubble rock and on the higher part of the slope with asphalt. The estimation of the third reduction factor, γ_b the influence factor for a berm, required a more thorough calculation which also provided the slope angle α to be used in the calculation.

The considered dike does not have a straight slope from the toe to the crest but consists of a composite profile with different slopes and a berm. A berm is defined by the width of the berm B, by the vertical difference dB between the middle of the berm and the still water level and by the characteristic berm length as can be seen in Figure B 12.

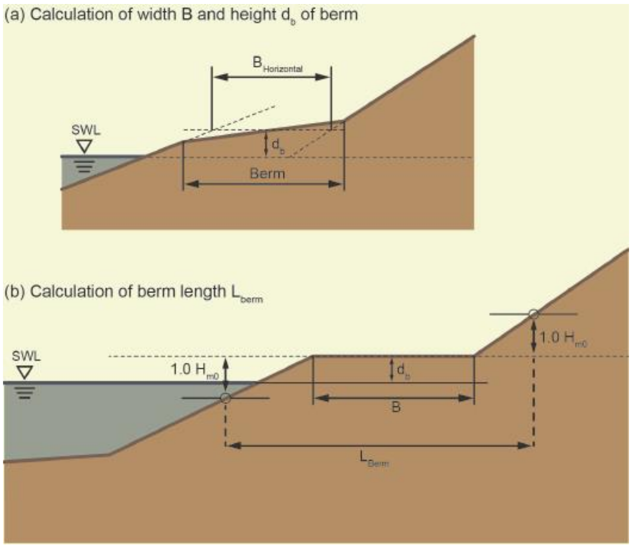


Figure B 12: Definition of horizontal berm width B, vertical difference dB and characteristic berm length LBerm, (J.W. Van der Meer et al., 2016)

The factor is given by: $\gamma_b = 1 - r_B(1 - r_{db})$ for $0.6 \leq \gamma_b \leq 1.0$

Where:

- $r_B = \frac{B}{L_{Berm}}$
- $r_{db} = 0.5 - 0.5cos\left(\pi \frac{d_b}{2 \times H_{m0}}\right)$ for a berm below still water line such in our case

"A characteristic slope is required to be used in the breaker parameter $\xi_{m-1,0}$ for composite profiles or bermed profiles to calculate wave run-up or wave overtopping." (J.W. Van der Meer et al., 2016)
It is recommended to calculate the characteristic slope from the point of wave breaking to the maximum wave run-up height. This approach needs some an iterative solution since the wave run-up height $R_{u2\%}$ is still unknown. For the breaking limit a point on the slope which is $1.5\times H_{m0}$ below the still water line is chosen. Also, a point on the slope $1.5\times H_{m0}$ above water is used as a first estimate to calculate the characteristic slope. As a second estimate, the wave run-up height from the first estimate is used to calculate the average slope and so on.

So, for the 1st estimate: $tana = \frac{3 \times H_{m0}}{L_{Slope} - B}$ as shown in Figure B 13:

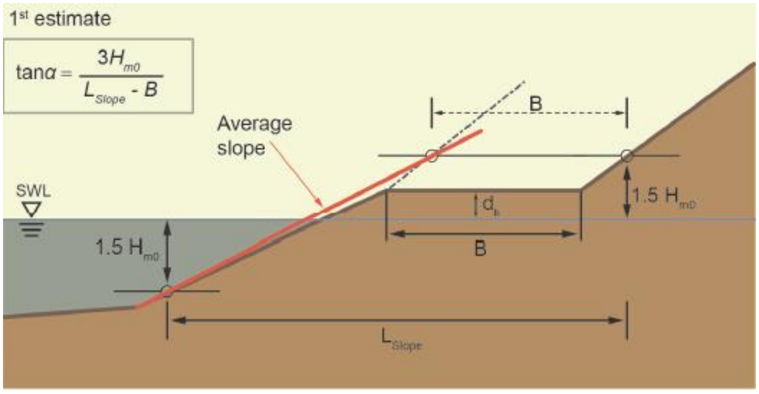


Figure B 13: Determination of the average slope (1st estimate)

For the 2nd estimate: $tana = \frac{\left(1.5 \times H_{m0} + R_{u2\%\ (1st\ estimate)}\right)}{L_{Slope} - B}$ as shown in Figure B 14:

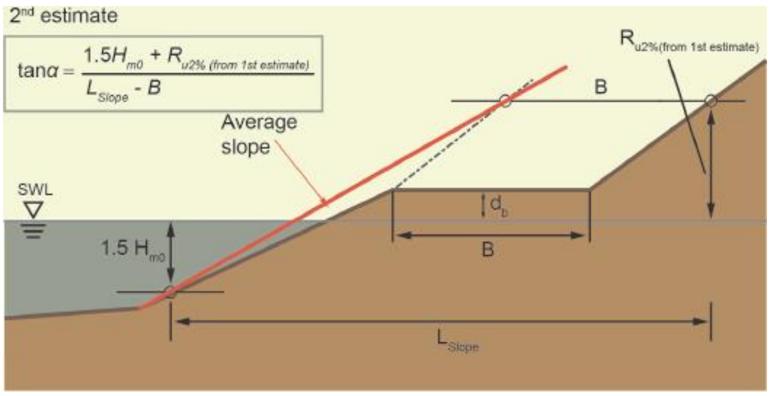


Figure B 14: Determination of the average slope (2nd estimate)

With iterations done in a Matlab script we calculate the $R_{u2\%}$,the average slope angle and the influence factor for a berm.

In a similar way the overtopping calculations are carried out with the use of the following equations:

$$\frac{q}{\sqrt{g \times H_{m0}^{3}}} = \frac{0.026}{\sqrt{tan\alpha}} \times \gamma_{b} \times \xi_{m-1,0} \times \exp\left[-\left(2.5 \times \frac{R_c}{\xi_{m-1,0} \times H_{m0} \times \gamma_{b} \times \gamma_{f} \times \gamma_{\beta} \times \gamma_{v}}\right)^{1.3}\right]$$

Where γ_v is the overall influence factor for a storm wall on slope or promenade

For the overtopping calculations the slope angle that was used was the calculated average slope and the overall influence factor for a storm wall on slope or promenade γ_v is taken as 1 due to the absence of such a structure.

17.3 Appendix C - Costing

Table C 1: CPI data

Year	Avg CPI
2018	1.35
2017	1.39
2016	0.32
2015	0.64
2014	0.98
2013	2.53
2012	2.47
2011	2.34
2010	1.28
2009	1.19
2008	2.49
2007	1.61
2006	1.17
2005	1.67
2004	1.24
2003	2.11
2002	3.29
2001	4.16
2000	2.31
1999	2.19

17.4 Appendix D - Wave Height Uncertainties

The reasons for the uncertainty in wave heights for the region of interest are listed below.

a) Data

The data for the analysis was taken from the model of BMT Argos webpage and refers to a 50-square kilometer area, which is larger than the location of interest. Satellite data, calibrated against buoys, were used to calibrate model wave and wind parameters. The final bias in model wave height relative to buoys is at most 5cm while wind speed is off by less than 20cm/s (BMT, 2018). With reference to buoys, the quality of satellite data introduces a relative error in wave height of 11%. Scatterometer data is calibrated with relative errors of 15% for wind speed after calibration with buoys (BMT, 2018). Overall error in 'best' model wind speed is 17-18% and the error in 'best' wave height is 15-16%. Correlation between buoy and model samples is high, i.e. linear correlation coefficients are 0.92 for wind and 0.96 for waves. The overall error statistics of the model can be observed in Table D 1. The quality of the model varies over the years and different regions.

Table D 1: Overall error statistics of model version v361 rel. to buoys per grid. Source: (BMT, 2018)

Model	Hourly Mean Wind speed u10					Significant Wave height Hs				
	Mean (m/s)	Bias (m/s)	Std (m/s)	Rrmse (%)	Corr (-)	Mean (m)	Bias (m)	Std (m)	Rrmse (%)	Corr (-)
Global	6.78	-0.08	1.29	17.0	0.92	1.99	-0.01	0.35	15.2	0.96
EU-shelf	7.89	0.06	1.54	17.7	0.92	2.01	0.05	0.39	16.4	0.96

Furthermore, additional error is to be expected during extreme conditions (which are the data used in the analysis) as according to the site: "You cannot assume that the sensors will produce reliable data up to the listed maxima under all conditions because the instruments are calibrated to optimize overall performance, not performance during extreme conditions (BMT, 2018). Moreover, very few satellite-buoy locations with high waves and windspeeds are available for calibration. Finally, the highest winds occur in a small region so the spatial resolution of a scatterometer will limit its ability to measure these high winds.

Another error is introduced as the calibration was done in an automated way. "Extreme conditions and in the vicinity of land or shallow water need special attention and cannot be dealt with by means of automated calibration. For these cases, manual calibration by an expert is required" (BMT, 2018).

b) Peak Over Threshold Analysis (PoT)

During the PoT analysis a threshold was chosen resulting in number of storms per year (Ns). There is no specific manual or recommendation for setting this threshold and it is usually selected with the experience of the user for each specific case. In our analysis the recommendation of (Van den Bos & Verhagen, 2018) was used of choosing a threshold that will result in a Ns value close to 10. Having another threshold would result in a change in the offshore significant wave height value.

c) Extrapolation-Weibull distribution

During the extrapolation of the significant wave height to the 1/4000-year return period, it was assumed that the data follows a Weibull distribution. However, the data does not perfectly match this distribution, especially when referring to the future extrapolated values, resulting in an uncertainty. Intuitively it becomes clear that this margin of error is larger the further the extrapolation to the future. A method to estimate this uncertainty is based on empirical research by (Goda, 2000) **.** As the deep water wave height for the desired return period has been found, Goda suggests that the uncertainty in the prediction can be modeled as a normal distribution with mean μ= *Hs (in this project case Hs=7.23 as shown in chapter 4)* and standard deviation σ_H. He presents an empirical formula to calculate σ_H as follows:

$$\sigma_H = \sigma_x \times \sigma_z$$

$$\sigma_z = \sqrt{\frac{1+\alpha(y_R - c)^2}{N}}$$

$$\alpha = a_1 \times \exp(a_2 \times N^{-1.3})$$

where σ_x is the standard deviation of the original *Hs* data values, *N* is the number of storms in the dataset, *α* is the shape factor of the Weibull distribution and the coefficients *a*1, *a*2 and *c* can be read from Table D 2. The parameter y_R for Weibull distributions is given by:

$$y_R = \ln((N_s \times R)^{1/a})$$

Table D 2: Coefficients for GODA (2000) method of uncertainty estimate.

distribution	a_1	a_2	c
Gumbel	0.64	9.0	0
Weibull, $\alpha = 0.75$	1.65	11.4	0
Weibull, $\alpha = 1.0$	1.92	11.4	0.3
Weibull, $\alpha = 1.4$	2.05	11.4	0.4
Weibull, $\alpha = 2.0$	2.24	11.4	0.5

If the standard deviation is known, the confidence intervals around the mean prediction can be drawn by assuming that the uncertainty can be modelled as a normal distribution around the mean, see Table D 3.

Table D 3: Confidence intervals around mean prediction for normally distributed uncertainty

Confidence level	Lower bound	Upper bound
68.2%	$\mu - 1.0\sigma$	$\mu + 1.0\sigma$
90%	$\mu - 1.64\sigma$	$\mu + 1.64\sigma$
95%	$\mu - 1.96\sigma$	$\mu + 1.96\sigma$
99%	$\mu - 2.58\sigma$	$\mu + 2.58\sigma$

By use of these formulas, the data and the previously calculated values result in the following Table D 4 and Table D 5.

Table D 4: Deep water wave heights uncertainty estimation calculation.

Data		Calculations	
N_s	10.36	**α**	2.05
N	259.00	**y_R**	5.18
R	4000	**σ_z**	0.43
c	0.39	**σ_H**	0.19
a_1	2.04		
a_2	11.40		
σ_x	0.45		

Table D 5: Confidence intervals around the mean of 7.23 for uncertainty of extrapolated deep water wave heights

Confidence level	Lower bound	Upper bound
68.20%	7.04	7.42
90%	6.92	7.54
95%	6.85	7.61
99%	6.73	7.73

These values will introduce a difference in wave heights at the toe of the dike and thus on overtopping values.

d) Directions of wave propagation and obliqueness
As previously discussed in chapter 4, there were two main directions of wave propagation, 250 ºN and 338 ºN. The waves propagate obliquely in reference to the bed contours and the structure. Therefore, refraction, on the one hand, and wave attack on the structure at an angle, on the other hand, can be expected. This would result in reduced wave heights and also a reduction factor due to obliqueness in the overtopping calculations.
Nevertheless, the conservative option of considering a single normal direction of wave attack was chosen.

e) Wind Velocity
In order to calculate the wind velocity that corresponds to each scenario of deep water wave height, a trend line was derived. To obtain the line, the significant wave height data for each storm were plotted against the corresponding wind velocities. Curve fitting to the graphs was then employed to obtain the equation of the line. However, one can observe from the graph in Figure B 10 that the data are quite spread around this trend line, thus its use adds an error. Furthermore, it was assumed that this equation connects all significant wave heights with respective wind velocities. It was then used to calculate the wind velocities for the extrapolated wave heights. This assumption will not hold in many cases. Anyhow, the change in wind velocities will not introduce as large an error as that of the wind direction. The sensitivity of the calculated parameters to wind velocity variations can be observed if the wind velocity is increased and reduced by 50%. These results can be seen in Table 10.

f) Wind direction

As mentioned above, the direction of the wind plays an important role for the wave height estimation. If wind direction and the wave propagation direction are the same, increase in wave heights will occur. The opposite is true for opposing directions. If wind direction and wave propagation direction are at an angle, the angle of wave attack might be alienated as the waves propagate.

The team assumed that the winds had the same direction as the waves, a conservative option. Consequently, it is expected for the wave heights to be lower than what was calculated. The sensitivity can be better understood by calculating the parameters for wind perpendicular and opposite to the wave propagation direction as shown in Table 10 .

g) Bathymetry

The bathymetry greatly affects the propagation of waves. Using a 1-D bathymetry instead of a 2-D bathymetry for the whole area is expected to greatly affect the wave height values.

In real life situations, bathymetry does not only change along the cross-shore direction but also along the along shore direction. Additionally, interactions with bottom formations (shoals, rocks, pits etc.) will affect the wave height values and the direction of wave attack. Furthermore, the coastal morphology was assumed steady. However, the seabed profile is continuously changing while affected by the waves and currents and especially in the long term (2100) one can expect significant changes in the considered bathymetry. To better grasp the sensitivity of the wave height and overtopping to bathymetry changes, the team calculated the parameters for the cases of bathymetry reduced by 1m, bathymetry increased by 1 m and for the case of a 2m bar at the toe of the structure. These results can be seen in Table D 6.

h) SwanOne

The SWAN model represents the wave field in terms of the 2D-frequency-direction wave spectrum which then evolves towards the coast including effects of wind, current, water level, depth, shoaling and refraction effects. It uses the Jonswap wave spectrum. SwanOne uses one-dimensional variance density spectra. SwanOne does not model diffraction (SWAN). While modelling with SwanOne, the wave energy is spread due to directional spreading, so that less energy is expected at the toe of the structure. This reflects the real physical process, however the level of spreading that the model uses might not match the one met on site. The uncertainties introduced by the model can be observed in Table D 6.

Table D 6: SWAN uncertainty about HS and TZ simulation results from different wave models considering all available data. shows mean bias value; shows mean RMSE; is equal to the mean scatter index values of HS. Source: (Ambühl, Kofoed, & Sørensen, 2015)

$\overline{B_{H_S}}$ [m]	$\overline{RMSE_{H_S}}$ [m]	$\overline{SI_{H_S}}$ [-]	$\overline{B_{T_Z}}$ [s]	$\overline{RMSE_{T_Z}}$ [s]	$\overline{SI_{T_Z}}$ [-]
0.0713	0.4436	0.3316	0.2034	2.5673	0.3640

In a more detailed analysis a two-dimensional computation model would be required.

17.5 Appendix E - Multi-Criteria-Analysis

Table E 2: Scores in Multi-Criteria-Analysis

Scores		Resistance		Resilience			Aesthetics/Environment			Economy						
	Options\Criteria	Prone to breach/catastrophic failure	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to future	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
Resistance	Dike Heightening	8	5	0	0	7	0	-3	0	-6	0	-3	7	-5	0	0
	Dike Widening	10	2	0	0	2	0	-4	0	-6	0	-3	2	-10	3	0
	Flood Wall	6	10	0	0	10	-5	-2	-8	-7	-2	-5	7	6	0	0
	Surface Protection	4	6	5	0	0	0	0	0	7	3	5	0	10	2	0
	Outer Berm	3	0	3	0	0	0	-1	0	-8	0	0	0	-4	0	0
Buildings	Raising structures	0	0	5	7	0	-3	0	-7	-4	0	0	-2	0	4	0
	Flood Proofing	0	0	3	4	0	-1	1	-1	4	-1	1	0	0	0	0
Urban	critical infrastructure	0	0	3	8	0	-3	-1	-5	2	-3	8	0	-3	0	0
	multifunctionality of buildings	0	0	1	5	0	0	0	0	1	0	0	0	5	10	0
	Resilient Transport	0	0	3	5	0	-1	0	-1	-8	-1	0	0	-2	3	0
Natural	salt marshes	2	4	0	6	0	-5	-6	-3	7	0	0	0	-8	3	0
	sand nourishments	3	5	0	7	0	7	-5	-2	-7	-1	6	0	5	6	0
	living breakwaters (reefs, oyster/mussel ree	3	5	0	5	0	0	4	0	-3	0	5	0	0	3	0
Policies	Risk Zoning and societal planning	0	0	0	9	0	0	0	-8	5	0	8	-2	-5	0	0
	Awareness and preparedness	0	0	0	8	0	0	0	3	7	0	8	0	0	0	-8
	Financial management	0	0	0	7	0	0	0	1	7	0	8	0	0	0	-5
	Shared initiatives	0	0	0	7	0	0	0	0	8	0	8	0	0	0	-5
Room for Water	Diversion canals	0	0	-1	8	0	-2	-3	-2	-3	0	0	0	-1	0	-2
	Wetlands/groundwater replenishment	0	0	0	8	0	-1	-4	0	8	0	6	0	-2	2	8

Table E 1: Ranks in Multi-Criteria-Analysis

Ranks		Resistance		Resilience			Aesthetics/Environment			Economy						
	Options\Criteria	Prone to breach/catastrophic failure	Resistance to overtopping	Prone to damages	Resilience gain	Expected amount of overtopping	Landscape, visual amenity and recreation	Environmental Impact	Cultural Modification	Cost	Maintenance	Flexibility/ Adaptability to future	Impacts on local businesses	Land use	Multi-Functionality	Ease of Implementation
Resistance	Dike Heightening	2	3	8	15	2	2	14	3	14	2	17	1	16	10	2
	Dike Widening	1	7	8	15	3	2	16	3	14	2	17	3	19	4	2
	Flood Wall	3	1	8	15	1	18	13	18	16	18	19	1	2	10	2
	Surface Protection	4	2	1	15	4	2	3	3	3	1	8	4	1	8	2
	Outer Berm	5	8	3	15	4	2	11	3	18	2	11	4	15	10	2
Buildings	Raising structures	9	8	1	6	4	16	3	17	13	2	11	18	5	3	2
	Flood Proofing	9	8	3	14	4	12	2	11	8	15	10	4	5	10	2
Urban	critical infrastructure	9	8	3	2	4	16	11	16	9	19	1	4	14	10	2
	multifunctionality of buildings	9	8	7	11	4	2	3	3	10	2	11	4	3	1	2
	Resilient Transport	9	8	3	11	4	12	3	11	18	15	11	4	12	4	2
Natural	salt marshes	8	6	8	10	4	18	19	15	3	2	11	4	18	4	2
	sand nourishments	5	3	8	6	4	1	18	13	16	15	6	4	3	2	2
	living breakwaters (reefs, oyster/mussel ree	5	3	8	11	4	2	1	3	11	2	8	4	5	4	2
Policies	Risk Zoning and societal planning	9	8	8	1	4	2	3	18	7	2	1	18	16	10	2
	Awareness and preparedness	9	8	8	2	4	2	3	1	3	2	1	4	5	10	19
	Financial management	9	8	8	6	4	2	3	2	3	2	1	4	5	10	17
	Shared initiatives	9	8	8	6	4	2	3	3	1	2	1	4	5	10	17
Room for Water	Diversion canals	9	8	19	2	4	15	14	13	11	2	11	4	11	10	16
	Wetlands/groundwater replenishment	9	8	8	2	4	12	16	3	1	2	6	4	12	8	1

17.6 Appendix F – Sensitivity to Failure Probability

Table F 1: Dataset to determine water level functions - do nothing scenario

OFFSHORE		TOE OF THE DIKE							
Water level [m]	**Hs [m]**	**Tm01 [sec]**	**Hm0 [m]**	**Set-up**	**NWL**	**Ru2% [m]**	**q [l/s/m]**	**alpha**	**Rc**
4.076	7.23	8.51	3.16	0.392	4.468	4.89	2.28	0.191	4.532
5.291	8.53	8.65	3.98	0.407	5.698	6.04	64.52	0.2063	3.302
4.966	8.53	8.59	3.78	0.369	5.335	5.76	30.75	0.2034	3.665
4.891	8.53	8.56	3.73	0.375	5.266	5.69	25.82	0.2029	3.734
4.676	8.53	8.54	3.59	0.391	5.067	5.49	15.37	0.2003	3.933
4.641	8.53	8.54	3.57	0.453	5.094	5.47	15.42	0.1998	3.906
4.591	8.53	8.53	3.53	0.398	4.989	5.41	12.25	0.1991	4.011
4.476	8.53	8.5	3.46	0.407	4.883	5.31	9.01	0.1985	4.117
4.426	8.53	8.48	3.43	0.411	4.837	5.27	7.88	0.1978	4.163
4.276	8.53	8.44	3.34	0.369	4.645	5.13	4.7	0.1964	4.355

Table F 2: Dataset to determine water level functions - robust design scenario

OFFSHORE		TOE OF THE DIKE							
Water level [m]	**Hs [m]**	**Tm01 [sec]**	**Hm0 [m]**	**Set-up**	**NWL**	**Ru2% [m]**	**q [l/s/m]**	**alpha**	**Rc**
4.076	7.23	8.33	2.85	0.398	4.474	4.43	0.42	0.1862	4.876
5.291	8.53	8.61	3.67	0.436	5.727	5.58	26.77	0.1996	3.623
5.226		8.59	3.63	0.401	5.627	5.53	21.82	0.1992	3.723
5.161		8.59	3.59	0.406	5.567	5.47	18.73	0.1984	3.783
5.096		8.57	3.54	0.411	5.507	5.4	15.68	0.1977	3.843
4.891	8.53	8.51	3.42	0.427	5.318	5.24	9.31	0.1963	4.032
4.641	8.53	8.45	3.26	0.483	5.124	5.01	4.77	0.1939	4.226
4.476	8.53	8.4	3.15	0.462	4.938	4.86	2.61	0.1923	4.41
4.276	8.53	8.33	3.01	0.479	4.755	4.65	1.24	0.1902	4.6

Table F 3: α- value results - stochastic approach

	Do nothing	Breakwater	Dike Heightening	Robust design
Alpha	0.5801	0.563	0.5809	0.5608
C1	0.086	0.08643	0.08704	0.0875
C2	0.3148	0.3157	0.3259	0.325
Hm0	0.3038	0.3305	0.3075	0.3344
Rc	0.4215	0.4269	0.4061	0.4139
Tm01	0.5357	0.5333	0.5379	0.5376

Table F 4: α- value results – water level functions approach

	Do nothing (qt=5)	Do nothing (qt=30)	Robust design (qt=5)	Robust design (qt=30)
C1	0.06369	0.1217	0.08036	0.08303
C2	0.2293	0.3239	0.3004	0.2278
WL	0.9713	0.9382	0.9504	0.9702

17.7 Appendix G – Countering Salinity

As briefly discussed in Chapter 9, the impact of SLR on the coastal zones' agriculture is due to saline seepage. An increase in salt water intrusion will have a large impact not only on Westkapelle region crops but also on all coastal zones affected by SLR. Hence, agricultural adaptation strategies are recommended to avoid damages to the salt intolerant crops. The following options are available (Stoorvogel, 2009):

a) Crop choice

The introduction of new crops, that are better suited to new abiotic conditions. A good example is the development of salty agriculture in areas affected by the salt intrusion can be replaced by more salt tolerant crops.

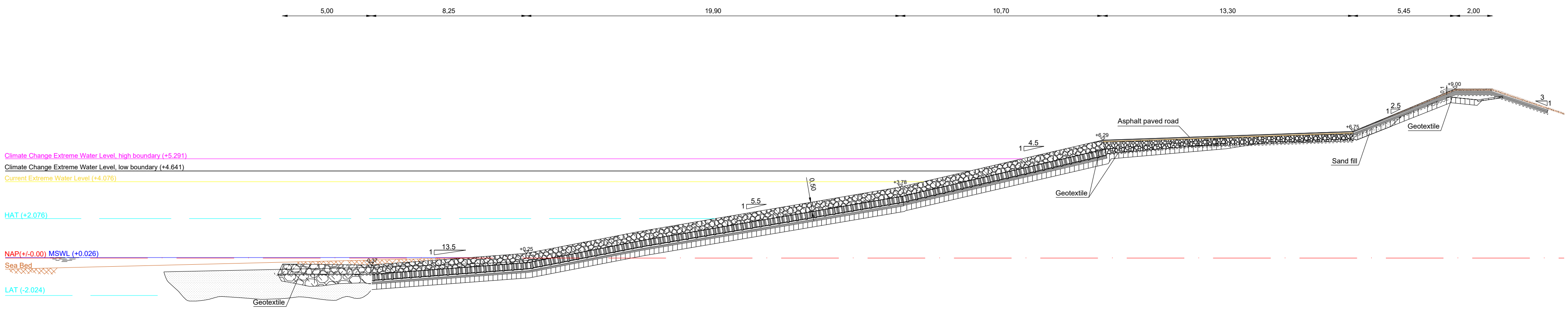
Genetic development of new crop varieties that are modified to be more salt tolerant and thus capable to deal with indirect effects like increased pest and disease pressure.

b) Crop management

This can be done by changing in cropping calendar (e.g., changes in planting dates), changing in input use which include changes in fertilizer use and/or pesticide use and finally changing in irrigation and drainage in order to deal with more extreme droughts as well as with wetter conditions.

c) Agricultural policy

This could be achieved by promoting policy changes that are more flexible in introducing new crops and cropping patterns, that are better adapted to climate change, and to create awareness that gets the sector well prepared for changes.



Profile 7 (Revised)

Legend

Measurements in meters relative to NAP

Scale 1:100

- Existing profile
- New profile
- Rock Armour
- Rock Underlayer
- Core (gravel)
- Bed protection (gravel)
- Sand
- Quarry rubble stone berm, asphalt penetrated (10-60kg)
- Asphalt penetrated crushed stone (10-60kg)
- Basalt column blocks
- Clay
- Old concrete blocks
- Phosphorus rubble
- Capped with Soil
- Soil Heightened Dike Section
- Grass Layer

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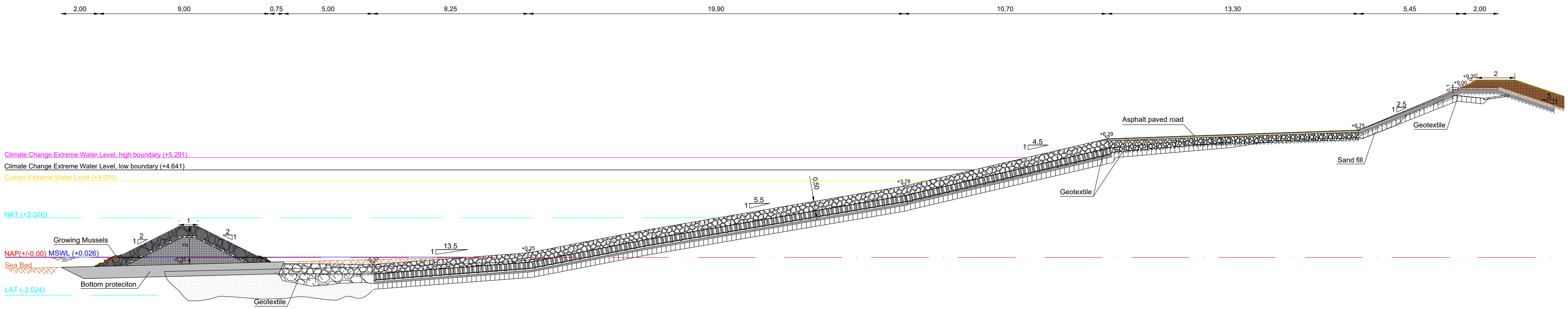

Profile 7 (Revised)

Legend

Measurements in meters relative to NAP

Scale 1:100

Existing profile	Sand	Old concrete blocks
New profile	Quarry rubble stone berm, asphalt penetrated (10-60kg)	Phosphorus rubble
Rock Armour	Asphalt penetrated crushed stone (10-60kg)	Capped with Soil
Rock Underlayer	Basalt column blocks	Soil Heightened Dike Section
Core (gravel)	Clay	Grass Layer
Bed protection (gravel)		

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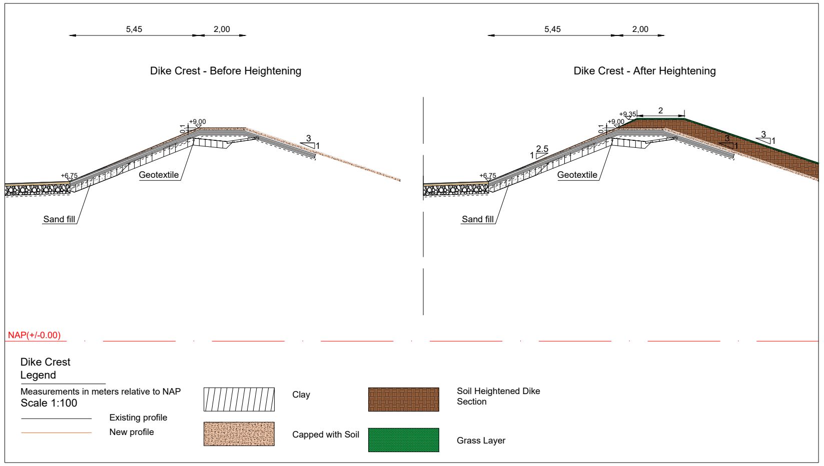

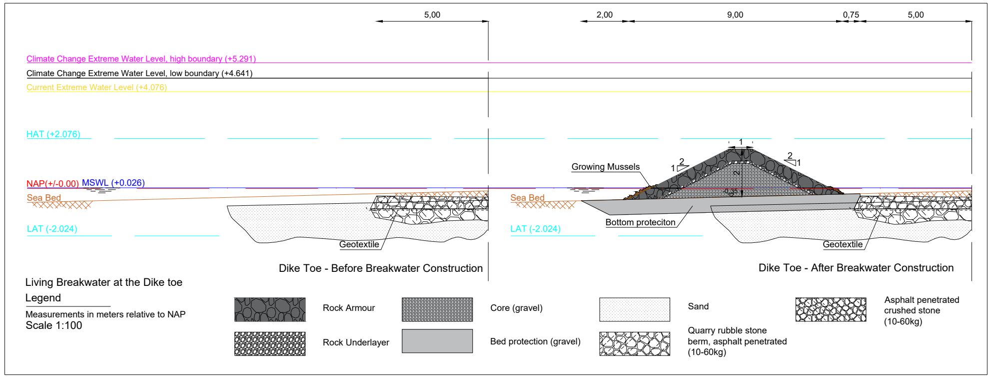

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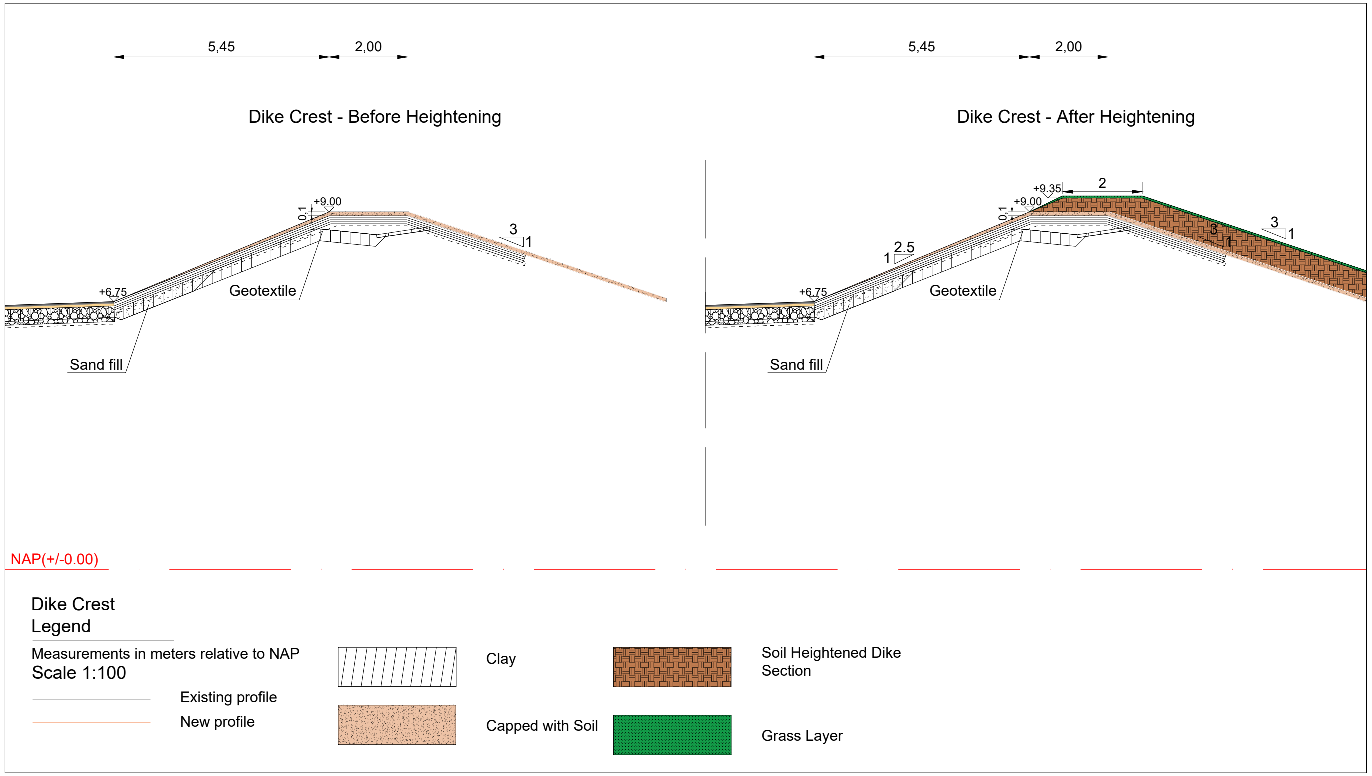

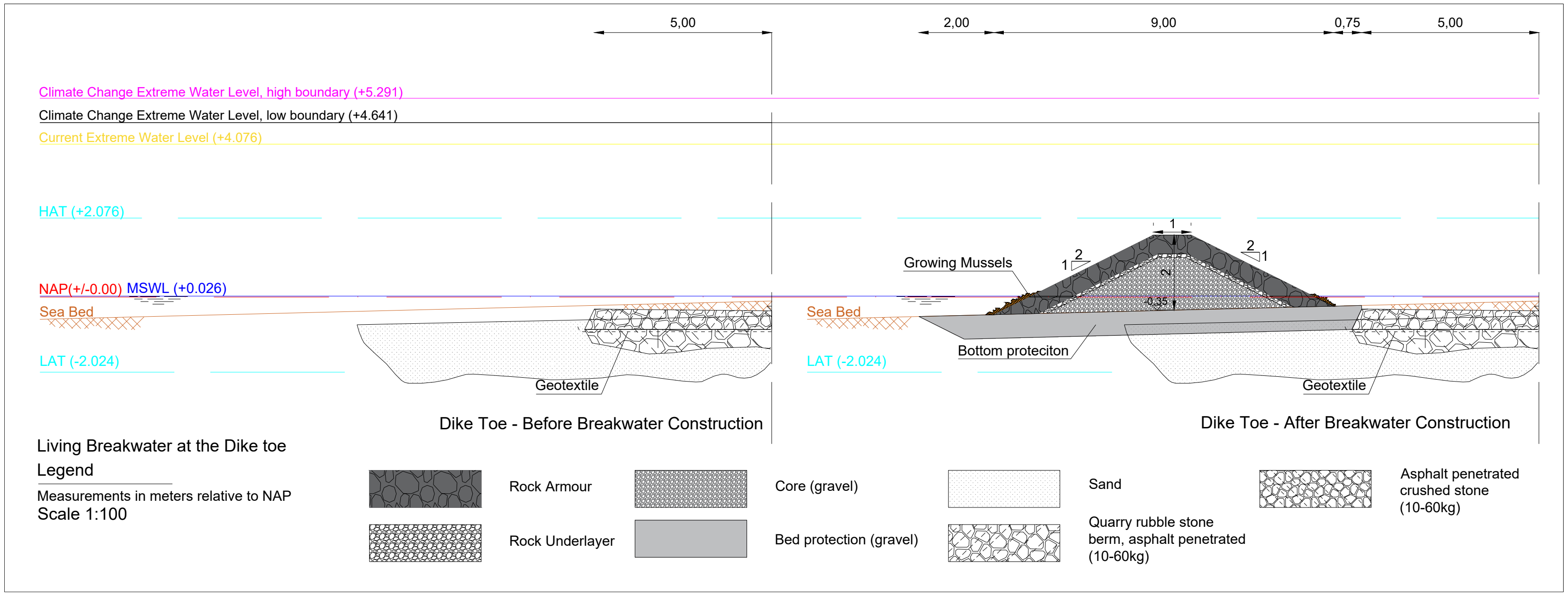

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