Optimising reinforcements in flood defence systems with multiple lines of defence

Case study on the Eastern Scheldt

MSc thesis

Leonie Jongen

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by

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Cover: Eastern Scheldt barrier

Abstract

In 2017, new safety standards were implemented for flood defences in the Netherlands. These new safety standards require reinforcements to be done in the Netherlands. The safety standards according to these standards are determined per dike ring and do not take into account changed interactions between various flood defence system elements. Additionally, they are not based on a cost-benefit ratio but rather on required levels of flood safety, prescribed by the Dutch government. Consequently, the reinforcements required by the new safety standards are not calculated with total costs in mind and therefore deviate. The resulting investments are higher than optimal when weighing investment costs against flood risk. Additionally, sea level rise is an uncertain factor when determining the loads in the future. The investments done should be optimal for future conditions as well. Due to uncertainty in the predictions of sea level rise, the optimal solution cannot be calculated with certainty.

These two elements give rise to the question on how to invest in cost-effective reinforcements in a flood defence system with multiple lines of flood defences when sea level rise occurs. Sea level rise results in changed boundary conditions for all flood defences in the system. The optimal division of investments between flood defences must be calculated. However, the most cost-effective reinforcements might change when sea level rise occurs, this should be taken into account when implementing reinforcements. Additionally, the balance between accepting or investing to reduce flood risk should be sought after. There is currently no approach to calculate the optimal reinforcements in a flood defence system with multiple lines of defence, while taking into account sea level rise.

The aim of this thesis is to propose a method for optimising reinforcements in a flood defence system with multiple lines of defence while also taking into account the uncertainty of sea level rise. The method is illustrated with a case study on the Eastern Scheldt.

The optimisation method calculates the investment costs of reinforcements on the dikes along the Eastern Scheldt and compares these to the resulting flood risk. The lines of defence are connected by calculating the hydraulic boundary conditions at the second barrier as a result of sea level rise and adaptations at the front barrier. A maximum of 1 meter sea level rise is taken into account. The options for the storm surge barrier that are included in this thesis are improving the closure reliability and heightening the closure water level. Heightening the closure water level decreases the closure frequency of the barrier in case of sea level rise.

The results show that for the Eastern Scheldt, the reinforcements at the dikes do not depend on sea level rise, because they are built strong enough not to require any further reinforcements. Sea level rise in this thesis is limited to 1 meter for the time period considered. The optimum for overtopping and revetment failure is to leave the dikes unchanged and not invest further. Macro-stability is assumed to be independent from the water level within the Eastern Scheldt up to 1 meter sea level rise, therefore the optimal reinforcements for macro-stability do not depend on the water level as well. The total flood risk does increase when more sea level rise occurs due to overtopping and revetment failure. Improving the closure reliability of the barrier decreases the total flood risk with 19% for 1 m sea level rise. Heightening the closure water level of the barrier increases the total flood risk with 17% for 1 m sea level rise. For this case study a calculation for overtopping was executed, calculating the optimal dike heightening increment for the situation with 1.5 meter lower dikes. The results show that the optimal reinforcements differ minimally for all sea level rise scenarios, either due to low flood damages number, high construction costs or the dampening effect of the barrier on the increase in hydraulic loads.

The reinforcements of the dikes in the case study do not depend on sea level rise or adaptations to the barrier. There are three reasons for this. Firstly, the dikes were built prior to building the barrier, therefore the dikes were built much higher than currently required and reinforcing the dikes further is economically not efficient. Secondly, the revetment has been reinforced between 1997 and 2015. Lastly, the barrier decreases the loads on the dikes. One meter of sea level rise does not result in one meter of water level rise within the Eastern Scheldt. In this case study the various scenarios calculate with a maximum sea level rise of 1 meter. Higher sea levels could result in different reinforcements that do depend on sea level rise.

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Nomenclature

Below the frequently used abbreviations and symbols can be found

Abbreviations

Abbreviation	Definition
OS	Eastern Scheldt
OSK	Eastern Scheldt barrier
SLR	Sea level rise
BC	Boundary conditions
KNMI	Koninklijk Nederlands Meteorologisch Instituut (Royal Dutch Meteorological Institute)
LCC	Life-cycle cost
HBN	Hydraulic load level
NAP	Normaal Amsterdams Peil (reference level used in the Netherlands)
WBI	Wettelijk beoordelings instrumentarium (Dutch statutory safety assessment)

Symbols

Symbol	Definition	Unit
h	Water level	[m + NAP]
H	Wave height	[m]
T	Wave period	[s]
Δ	Relative density	[-]
ρ	Density	[kg/m^3]
β	Reliability index	[-]

1 Introduction

1.1. Safety standards in the Netherlands

In 2017, a new set of safety standards regarding flood safety for the Netherlands was introduced. All flood defences in the Netherlands had to be assessed according to these standards. The outcome of this assessment was that large parts of the Dutch defence system are not up to the standards that will be required in 2050. The new standards are based on maximum allowable flood probability per area as opposed to exceedance probability per dike segment, which was used in the past. Three risk indicators were considered when determining the new standards[2]:

1. Individual flood risk
2. Societal flood risk
3. Societal cost-benefit optimum

The individual risk criteria prescribe a basic level of safety for every citizen, taking all possible flood events combined into consideration. The societal flood risk takes densely populated areas into account, which need to be better protected because the risk of losing lives is greater than in case of a flood in sparsely populated areas. The third indicator requires the investment in flood safety to be a step towards the cost-benefit optimum of flood defences. The cost of a flood defence reinforcement should be lower than the amount of reduction of flood risk in order for such reinforcement to be an economically sound investment. When multiple reinforcements are possible, the combination of reinforcements with the lowest cost-benefit ratio should be chosen; this is the economic optimum. The three indicators translate to new maximum failure probabilities per flood defence segment as opposed to the safety standard which was applied before 2017. The consequence of the new safety standards is that all flood defences in the Netherlands had to be checked against these new safety standards. Many flood defences need to be improved in the coming years to comply with the new standards.

1.2. Uncertainty in future boundary conditions

Besides the need to improve the flood defence system due to higher safety standards, the loads on the flood defence system increase as a result of sea level rise caused by climate change. Taking the best decision for long-term investments is becoming increasingly more difficult due to the growing uncertainties of boundary conditions in the future. The effectiveness of a decision that requires small funding and lasts 50 years is easier to assess than one that requires larger funding and spans across 100 years. However, if an investment lasts longer, the total effectiveness is likely to be larger. The time scale of these decisions therefore matters. KNMI [28] shows clearly that the uncertainty of sea level rise predictions is very large for long timescales. Due to uncertainties associated with the conditions in the future, other ways of looking at decision making processes have been proposed. According to Lempert et al. [33], relying on predictions in a fast changing world can prove counter-productive and sometimes even dangerous. Therefore it is proposed to make use of 'robust decision making' processes where the decision maker looks for the solution that suffices in most scenario's. In order to do this, the net present value of all different strategies needs to be calculated for all possible scenarios. In this way, the quality of the decision making process improves and the chance that a flood defence is able to fulfill its function in the future increases. Another way to evaluate the different strategies is to use the real options analysis and value the flexibility a strategy offers, making it more suitable in different conditions (van der Boomen [9]). This thought process is used by Haasnoot [20] where multiple paths are distinguished that can be used to reach the end goal (the long term solution). An initial solution path for the short term leads to other options when a new decision needs to be made. Making the first decision may exclude other paths. However, some decisions can be postponed until more knowledge is available and a better, more informed decision can be made. The adaptation pathways approach couples the short-term decisions to the long term. Coupling robust decision making and adaptation pathways to decisions on reinforcement strategies of the flood defence system is a promising approach to deal with large uncertainties surrounding boundary conditions in the far future. Adopting a strategy that accounts for uncertainties is important in order to make correct decisions.

1.3. Previous research and knowledge gap

A flood defence system can consist of a primary and a secondary flood defence. In situations with such multiple lines of defence, the current strategy is to define safety targets for both the front and the rear flood defence separately (Jongejan, Kramer, and Delhez [24]). However, the two lines of defence influence each other's effectivity. When multiple lines of flood defences exist, how does system thinking influence the decisions made for reinforcing one or the other flood defence compared to the current strategy? An improved way to optimise flood defences is to look at the entire system, not at a single flood defence, and to consider the influence the elements have on each other. Klerk et al. [27] provides an economical optimal solution for the reinforcements in a dike segment while also producing a priority list. It consists of a list of measures ranking from best cost-benefit ratio to least. The priority list is important in cases where resources are not immediately available and a choice must be made as to which measure to take first. However, the limitation of his method is that it is only applicable to a single line defence system. For situations with multiple lines of defence this method is inadequate. The connection between the front and the rear flood defence is explained by Dupuits, Schweckendiek, and Kok [17], looking at the influence of the front flood defence on the rear flood defence for optimising the safety targets. The approach is aimed at optimising the safety targets, not at reinforcing the system. This has only been done on project level for a specific location such as the Maeslantkering as described by L.F.Moyaart et al. [32]. Existing literature does not describe a study that aims at improving this kind of system interventions.

1.4. Problem analysis and problem statement

The problem that can be deducted from the first paragraphs, is that there is currently a method missing to optimise reinforcements in a flood defence system with multiple lines of defence. This type of systems is now reinforced by looking at the flood defences as an individual flood defence with an individual safety target. Optimisation of the system as a whole is not performed. Optimisation over a long period of time(i.e. 100 years) is becoming increasingly more difficult due to fast-changing boundary conditions and therefore requires more attention when optimising a flood defence system.
The research question of this thesis is therefore:

How can reinforcements in a flood defence system with multiple line of defence be optimised under changing boundary conditions due to climate change?

The objective for this thesis is therefore to optimise reinforcements in a system with multiple lines of defence under the pressures of a changing environment. The optimisation therefore does not only concern the long-term or short-term solution space, but aims to connect the two timescales to arrive at the most efficient solution for a system as a whole with multiple lines of defence. For the short-term solution space, in this thesis a timescale of about 50 years is taken. A long-term timescale considers 100 years or more. This thesis will investigate a general approach to this problem, taking the Eastern Scheldt as a case study. The limitations of the current Eastern Scheldt barrier will be sought after while attempting to optimise the flood defence system with climate change in mind.

In order to help clarify and subdivide the research, sub-questions have been formulated. The sub-questions are:

1. How to model the interaction between multiple lines of defence in a flood defence system?

2. How to take uncertainty in future boundary conditions into account in the optimisation?
3. What are the separate reinforcement options for both flood defences (primary and secondary) of the Eastern Scheldt system?
4. How to select the optimal strategy for the Eastern Scheldt for the long-term?

1.5. Approach and limitations

In this thesis a method for an economic optimisation of reinforcements in a flood defence system with multiple lines of defence is investigated. The Eastern Scheldt will be taken as a case study to illustrate the method. The focus will be on flood risk estimates and evaluating the cost and benefits of different reinforcement alternatives. Different strategies at the Eastern Scheldt barrier will be discussed in this thesis, along with estimates of their costs and their effect on the dikes along the Eastern Scheldt. When in this thesis a barrier strategy is discussed, it is meant to mean a plan on what to do with the barrier, i.e. any reinforcements or changes made to the barrier. The optimal strategy for the dikes when a barrier strategy is chosen is calculated. A calculation is made of various costs between the different strategies at the barrier with the goal to decide on an optimal strategy. The flood risk will be weighed against the cost of construction and improvement of the flood safety measures. The dike sections will be considered as separate structures from the storm surge barrier. The assumption is made that the area in between the structures is stationary, not dynamic in terms of cross-section and morphology. The hydraulic boundary conditions in this area may change, but the morphology is assumed to remain constant. In reality the morphology does play a role and might change over time. The force of the waves on the defences is influenced by the morphology.

Van den Boomen [9] states that the classical net present value and life cycle cost approach does not suffice for most infrastructure projects, stating it leads to sub-optimal timing and costs. The reason for this statement is that the classical net present value approach does not take into account the flexibility a certain decision offers. Flexibility can prove to be valuable if circumstances change (i.e. sea level rise) and allow for changes to be made to better suit the new situation. In this thesis this reasoning will be disregarded. Instead, LCC and NPV is used in this thesis. This is not a problem because firstly, the choice for one reinforcement does not exclude another. Secondly, the optimisation will be done conditionally, therefore without any uncertainty, after which the uncertainty is left in the choice of how much sea level rise is assumed. The flexibility of the infrastructure will be included in the decision making process that arises after the optimisation. The changing economic climate and the availability of resources is not considered in this thesis. Making an accurate estimation of these fluctuations is not within the expertise of the author of this thesis and cannot be done accurately within the time frame. The discount rate as stated in 2020 [1] is used for simplification purposes.

Furthermore, a simplification is used of the dike sections that have been researched in this thesis. The possible heterogeneity of the soil will be included in the macro-stability calculations as simplified to dike sections. Taking soil heterogeneity further into account would result in more data, while barely contributing to the goal of this thesis, as the dike section is already at small scale compared to the scale of the flood defence system.

1.6. Reader's guide

This thesis will start by introducing the subject with literature to highlight the knowledge gap and provide some background on the subject. Next, the methodology of this thesis will be discussed in chapter 3: Methodology. Chapter 4 discusses the change in hydraulic boundary conditions within the Eastern Scheldt due to sea level rise and the program that is used to obtain these statistics. Afterwards, the Eastern Scheldt as a case study will be discussed in chapter 5. The methodology for this specific case will be considered as well as the failure mechanisms that are included. Chapter 6 discusses the results from the case study as well as results from a secondary study that has been done. The results are discussed in chapter 7, including the limitations that this thesis has. Finally, chapter 8 includes the conclusions and recommendations stemming from the results and discussion.

2 Literature review

2.1. Flood risk in the Netherlands

What is flood risk and how can flood risk be calculated? Flood risk is defined in the field of hydraulic engineering as the multiplication of the impact of a flood and the probability that the flood will occur [30].

$$Flood\,Risk = P_f * Damage \tag{2.1}$$

In which P_f is the flood probability. Damage in case of a flood includes economic damage as well as loss of life. However, as mentioned in section 1.1, the impact can also be looked at in different dimensions: e.g. individual risk or societal risk. Dividing these maximum acceptable flood risks by the expected damage will lead to a maximum acceptable flood probability per area. According to the safety standards, the failure probability of a flood defence structure is equal to the probability that a load exceeds the ability of the structure to withstand the load [30]. Since one single structure does not usually protect an entire area, the flood probability of that area is determined by the combination of all flood defences protecting the same area [27].

As mentioned in section 1.1, the safety targets in the Netherlands have been renewed in 2017. These new safety targets are stricter than the old ones. Based on the new targets, flood defence systems have to comply with three risk indicators: individual flood risk, societal flood risk and societal cost-benefit optimum. Firstly, the individual flood risk of every citizen in the Netherlands needs to be lower than 10^{-5} (*Nieuwe normering van waterveiligheid* [34]). This individual flood risk is the probability that someone dies due to a flood. This probability can be calculated by the following formula:

$$IR = P_f * m * (1 - E) \tag{2.2}$$

In which:

- IR: individual risk
- P_f: probability of flooding
- m: mortality rate
- E: evacuation rate

The mortality rate is the fraction of people that dies of all the people that stay behind after an evacuation. The individual risk can be influenced by a multi-level protection approach; improving either the evacuation fraction or the mortality rate. Secondly, the societal flood risk leads to higher safety standards in densely populated areas. The third indicator, the societal cost-benefit are costs due to the damage as a result of a flood; economic damage as well as loss of life. The costs of a reinforcement to the flood defence are included as well. The benefit of such a reinforcement is the reduction in flood risk, i.e. the expected damage due to floods. The flood defences in the Netherlands had to be checked against these new safety targets to see whether they are still strong enough. For a system with multiple lines of defence, the safety targets are determined per separate line of defence [24]. A majority of the flood defences in the Netherlands does not comply with these new safety targets and therefore needs to be reinforced [23]. The new safety targets need to be implemented before 2050. The relevance of this thesis comes into play when deciding which reinforcements to choose for a system with multiple lines of defence as the most cost efficient solution. The separate safety targets may need to be set aside in order to obtain the optimal solution for such defence system as a whole.

2.2. Economic optimisation

In this section the current literature on optimisation of flood defences is considered. Optimisation of flood defences is defined here as the minimisation of expected costs in line with previous research (e.g. Dupuits, Schweckendiek, and Kok [17]). The total cost of an element or system during its life time is called the Life Cycle Cost (LCC). All costs are translated to present value to be able to compare different options. The optimum is calculated as follows:

$$min(\,Total\;costs) = NPV \sum Flood\,risk + NPV \sum Investment\;costs \tag{2.3}$$

Costs that are considered are the costs due to investments made in flood defences or the costs due to damage when an area is flooded. Since there is no certainty on whether a flood will occur, the expected cost over the considered life time will be taken into account. Using LCC to calculate the best investment in flood defence systems in this thesis is limited to aspects that can be quantified. Other factors, such as sentiment or ecology need to be quantified to take into account, which is not always straightforward and thus not easy to calculate. The costs of damage multiplied by the probability of occurrence are weighed against the cost of a reinforcement combined with the reduced flood probability. However, the current optimisation methods assume stationarity in boundary condition statistics which, due to climate change, are likely not to be true for the lifetime of these investments. The flood risk will be underestimated if stationarity is assumed, therefore the calculated LCC will be lower than the actual LCC. Including non-stationarity is therefore important to arrive at the optimal strategy
According to Biondini and Frangopol [8] not only changing external conditions need to be taken into account when considering the life cycle (and thus its costs), but also the deterioration of the strength of the structure. Quantifying the deterioration of a structure and the development over time is essential to determine the state of the structure. Therefore, this needs to be taken into account when optimising a flood defence system. The deterioration of strength for a dike will not be included in this thesis since the order of deterioration for a dike is much smaller than the increase in loads due to sea level rise. The assumption is that the deterioration of a dike will not influence the optimisation results.

2.2.1. Optimisation of reinforcements of a flood defence series system

For a single line of defence, the optimisation is more straightforward than for a system with multiple lines of defence. Klerk et al. [27] developed an algorithm to find the optimal solution for a dike segment considering the reinforcement options. The costs of the options are weighed against the reduced flood risk until an optimum is found. The optimisation in Klerk et al. [27] is applied to a dike segment and not a single section. The strength of one section could compensate for a weaker neighbouring section and as a result the entire dike segment could comply with the safety standards. It was demonstrated that for riverine flood defences this method resulted in a decrease in investment costs. The algorithm is then used to efficiently calculate which reinforcements are most effective and therefore have priority over other reinforcements. The prioritising of the reinforcements provides guidance in the decision making process in case funding is not available immediately. However, the method does have some limitations. Firstly, a singular cost estimation is taken per reinforcement over a dike section, while in practice there are overhead costs for the reinforcement, resulting in lower costs per kilometer if the reinforcement is executed at the same time over a longer stretch of dike compared to a shorter stretch. Sobhaniyeh et al. [46] has researched different scenarios of a river system and has designed strategies for each of the scenarios described. The optimisation is reached in the context of an individual scenario. The flood risk in each scenario and strategy combination is evaluated using physical modelling to be able to calculate the failure probability. The results comprise of strategies that are optimal for each possible future system state. The optimal solution can be reached by using the robust decision making principle, which will be explained in subsection 2.3.1.

2.2.2. System modelling and optimisation of multiple lines of defence

Research has been done on the interaction between multiple flood defences within the same flood defence system. This research, which will be touched upon in this subsection. What are flood defence systems with multiple lines of defence? When talking about a flood defence system with multiple lines of defence, the primary defence is the flood defence that is located closest to the land. The secondary flood defence reduces the water levels and wave heights a body of water, which is located next to the primary flood defence, see Figure 2.1.

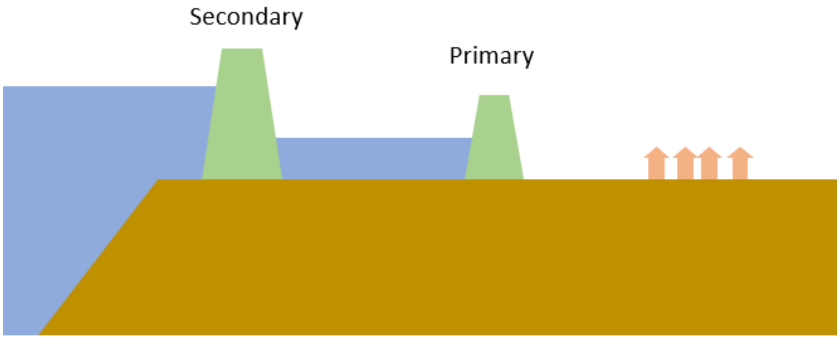

Figure 2.1: Primary and secondary dikes

How do these multiple lines of defence interact with each other? According to Ciullo et al. [13] often the hydraulic interactions between different flood defences or flood defence sections are either not taken into account or reduced to a single parameter. Risk reduction measures are commonly investigated per area without looking at the impact on other areas of the flood defence system. Ciullo argues that policies should also investigate the transfer of risk across areas when implementing measures. The investigation by Ciullo concerns the risk transfer in fluvial systems, which work differently as opposed to coastal flood protection systems due to the continuous flow of water. The underlying idea however, can be used for coastal flood protection systems. When adapting a flood defence in a system, the impact of that adaptation on the hydraulic loads on the other parts in the same system must be taken into account. The reduction of flood risk in the system as a whole is thus important, not only in the singular area that the relevant flood defence protects. Dupuits, Schweckendiek, and Kok [17] aim to optimise a flood defence system with multiple lines of defence. However, their research does not encompass the full interaction between the front and the rear defense system. Their research is modeled with and without a front flood defence, to see whether it is useful to include the interaction when deciding upon the economically optimal safety targets. The model is thus not being used to look at reinforcement options, but at safety standards. Berchum et al. [6] calculate for each combination of flood reduction measures the remaining flood risk. Multiple options are compared and analysed to decide on the optimum solution. This method works well when options are limited and the flood risk can be calculated easily. For a larger, more complex system, such as a flood defence system with multiple lines of defence, this approach is not easy to apply and will be very time-consuming. Berchum et al. [5] also created a model for multiple lines of defence in a flood defence system. This model is meant to assist in making decisions in the conceptual design phase. This model simplifies the optimisation by using basic formulas to reduce the computation time. The damage number used in the model includes the damage due to the flood, but also the damage to the flood defence structure. The model optimises the flood defence strategy, whereby the 'reinforcements' are both structural and non-structural solutions. The model does not include the possibility of not improving existing ones. What is relevant to this thesis, is the hydraulic model used by Berchum which calculates the failure probabilities of the flood protection structures and the discharge flow into the next basin (see Figure 2.1). Failure of the flood protection of the first basin thus influences the flood protection in the next basin.

Another approach to a flood defence system that could be taken is to enlarge the area of interest. In Vousdoukas et al. [51] the area of interest is Europe, simplifying the input such that a cost-benefit analysis can be made, balancing the flood risk and investments. The flood risk in this model is calculated as a result of extreme water levels, storms and sea level rise and depends fully on overtopping failure. The investments are thus also simplified as costs to heighten dikes. This approach works when deciding

on a certain general strategy, but the simplifications used in the model make the model unable to use when deciding to heighten specific dikes. Additionally, other failure mechanisms should be taken into account for a proper analysis, instead of relying solely on overtopping.
L.F.Moyaart et al. [32] optimised the flood defence system of the Maeslant barrier while using the closure reliability of the storm surge barrier to calculate the optimal height of the dikes behind the barrier. The model calculates the height with which the dikes need to be heightened, therefore only taking overtopping into account as failure mechanism. The optimisation was used to substantiate the decision to construct the Maeslant barrier. In that situation, the Maeslant barrier is assumed not to have been constructed yet to be able to calculate the best decision. In this thesis, the barrier of the Eastern Scheldt is already built and the options possible for the barrier are mere adjustments, not the decision whether or not to build the structure in the first place.

In conclusion, literature includes flood defence system optimisations, but these do not optimise the reinforcements possible for the situation with an existing barrier. The approach of Berchum et al. [6] is relevant due to the approach on reduction of flood risk by the effectiveness of the barrier. The infleucen of the multiple liens of defence is included in the flood risk results.

2.3. Managing uncertainty about the future

2.3.1. Decision making for uncertain future boundary conditions

In this section, literature is discussed that offers a view on various methods of decision making processes for uncertain future boundary conditions. It is important to reach decisions that are future-proof, taking into account the uncertainties of climate change and the possible impact of decisions that still need to be made. Lempert et al. [33] introduces the Robust Decision Making approach. This approach does not aim at making predictions, but at enabling the decision maker to reach better decisions when the conditions are uncertain. A wide range of possible scenarios are considered, after which possible decisions are valued based on how well these decisions perform across all the different scenarios. Robustness of a chosen solution means how well a solution performs under a wide range of conditions. According to Postek et al. [37] considering 40 % uncertainty in sea level rise leads to a maximum increase in total costs of 10 %. These increased costs will prove to be a more economic choice in the long run when extreme sea level rise scenarios materialise. However, some future conditions are more likely than others, which is not considered in the Robust Decision Making approach. All extreme scenarios are taken into account and no difference is made between the various scenarios. The method shows where the weaknesses in strategies lie when improbable scenarios occur, but does not show the best option for the most probable scenario. Another approach is the adaptation pathways approach. This approach indicates paths that can be taken to reach the end goal. Each of the paths comes with its own costs, side effects and is evaluated on how well it reaches the target. An approach taken now could reach the end of its useful life in 30 years, when a new decision needs to be made. This thought process is displayed in the adaptation pathways map by Haasnoot [20], which contains the paths that can be taken. A road ends when a so-called tipping point is reached and the current strategy does not suffice anymore to meet the needs set for the end-goal. The map can provide insight into the future options after a decision is made. For example, if one were to decide that the Eastern Scheldt needs to be closed off by a dam, a later option could be to heighten the dam, but the option of a reinforced storm surge barrier does not exist anymore. The adaptation pathways approach attempts to connect the end-goal to the decisions that need to be made now, sometimes revealing that the two do not match and a new vision for the end-goal needs to be created. The new vision will have different needs and objectives and other pathways will prove to be more effective to satisfy the needs in the new situation. A disadvantage of this way of thinking is that it is not possible to design a definite strategy for the long-term. Only the first step can be identified; the decision for the longer-term needs to be made when in time more information is available. At what point in time that decision moment arises, is not always clear since the tipping points need to be defined accurately, which is not always possible.

2.3.2. Scenario thinking

Complementing the Robust Decision Making approach and the adaptation pathways, Kwadijk et al. [31] uses an approach to define the tipping points within an existing system. Stress testing the existing system under different climate scenario's, results in the timing and kind of action that is required. This ' bottom-up' approach is focused on vulnerability and risk management to improve resilience and robustness under the pressure of climate change. The main advantage is that the approach does not depend on climate projections and can therefore be applied without choosing one projection as the 'truth'. The decision which climate scenario is correct therefore does not need to be made.

2.3.3. Replacement optimisation

In Van der Boomen [9] the standard application of the classical net present value and life cycle cost (LCC) is disputed. Price deviations are generally neglected, but they influence optimal replacement times. The moment at which an intervention can best be done differs, is influenced by price variations and market conditions. Price uncertainty increases exponentially in time, therefore it is difficult to include this uncertainty for long-term projects. It is recommended to consider the influence of climate change, energy transition and circular construction on the optimal replacement time. For infrastructure projects, the life cycle cost method as used commonly does not work. The method assumes a residual value at the end of the design life time, in the form of a cash flow that can be obtained if the asset is sold. However, infrastructure such as flood defence systems cannot be sold. The aspects mentioned in the article are all influencing the outcome on the best investment and best investment time. In order to optimise reinforcements, the cost of these reinforcements must be accurately calculated. In the article, six models are proposed to better calculate the optimal replacement time and net present value. When the infrastructure is to be replaced with a similar structure, meaning the replacement is repetitive, the classical replacement optimisation methods suffice. A repetitive cycle concerning uncertainty in future boundary conditions is present. Additionally, the infrastructure in question is not replaced but improved. Therefore Van der Boomen's statement that the classical LCC does not suffice does not apply here.

2.4. Impact of climate change

In this thesis climate change will be taken into account by looking at different climate scenarios. The climate scenarios that will be used originate from the most recent data produced by the KNMI in 2021 [28]. The scenarios from the KNMI are based on the sixth assessment report of the IPCC [36]. Climate change has many effects, but only a couple will be affecting flood safety and future boundary conditions. Just at the completion of this thesis, on October 9 2023 KNMI published a new report [29], with new climate scenario's. The conclusion of KNMI is that a further acceleration of melting of the Antarctic ice will have a major influence on sea level rise in the Netherlands. A sea level rise of several meters might become reality in the future. However, no probability of occurrence is awarded to the calculated scenario's. Therefore sea level rise is limited to 1 meter in this thesis, without awarding a probability to the amount of sea level rise.

Global warming

The sixth assessment report of the IPCC states with certainty that the warming of the Earth is a consequence of the emission of greenhouse gasses. The temperature between 2011-2020 is 1.1 degrees Celsius higher than the temperature between 1850-1900 [36]. While global warming has consequences, the higher temperature itself will not have a direct influence on flood safety and will therefore not be considered further in this thesis.

Extreme events

Another trend caused by climate change is the increase in the number of extreme weather events [36]. Extreme weather events could consist of long periods of drought, extreme rainfall or heatwaves. Storm events will become more frequent and more extreme, increasing loads and frequencies on flood defences. Predicting the increase of the frequency and intensity of storm events is difficult. The increased frequency of extreme storm events can be taken into account by performing a sensitivity analysis, since quantifying the increase beforehand is difficult.

Wind speed According to literature, there is no clear indication of the changing wind speed due to climate change. The natural variability of wind thus far exceeds any signal of changing wind speeds due to climate change (Pryor et al. [40]). Wind speed is thus assumed to remain unchanged.

Sea level rise

Climate change causes the Earth to warm, melting the ice caps and consequently raising the sea level.

Global warming results in thermal expansion of oceans, contributing to sea level rise. This process influences the boundary conditions at the Eastern Scheldt barrier. The expected lifetime of the barrier will be reduced if sea levels exceed the limits of the barrier. Multiple case studies research the impact of climate change on flood risk, stating that flood risk increases ([11], [12], [41], [42]) due to climate change. These results indicate that it is very important to include climate change into long-term scenario's used for decision making. Figure 2.2 shows data on the sea level at the Dutch coast. The data reveals a sea level rise of 1.8 mm/year until 1990. Between 1990 and 2020 an increase of 2.9 mm/year was measured. The sea level rise thus accelerated slightly since 1990 [47].

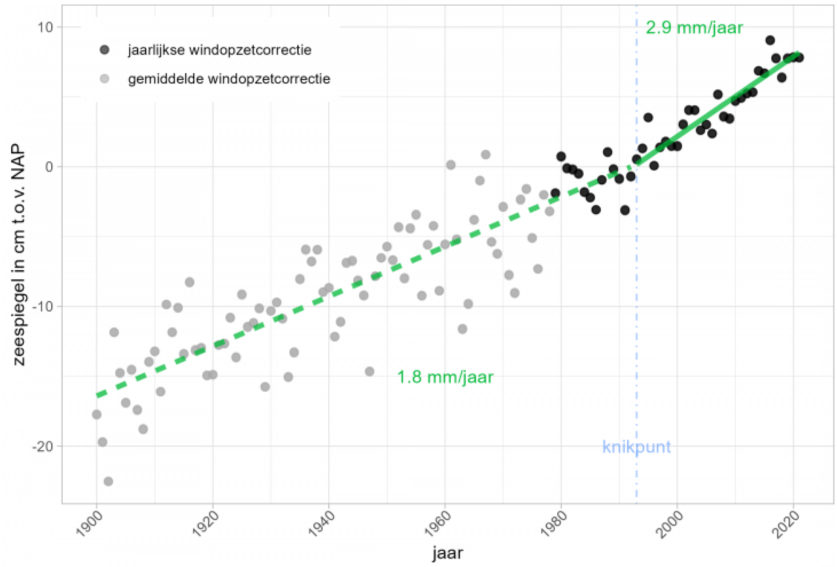

Figure 2.2: Data sea level monitor [47] on sea levels in the last century

KNMI [28] distinguishes three possible scenario's for sea level rise at the Dutch coast in 2100:

- SSP1-2.6: 30-81 cm
- SSP2-4.5: 39-94 cm
- SSP5-8.5: 54- 121 cm

The sea level rise is taken with respect to a reference level, set in 2005. This rising trend can be seen in Figure 2.3.

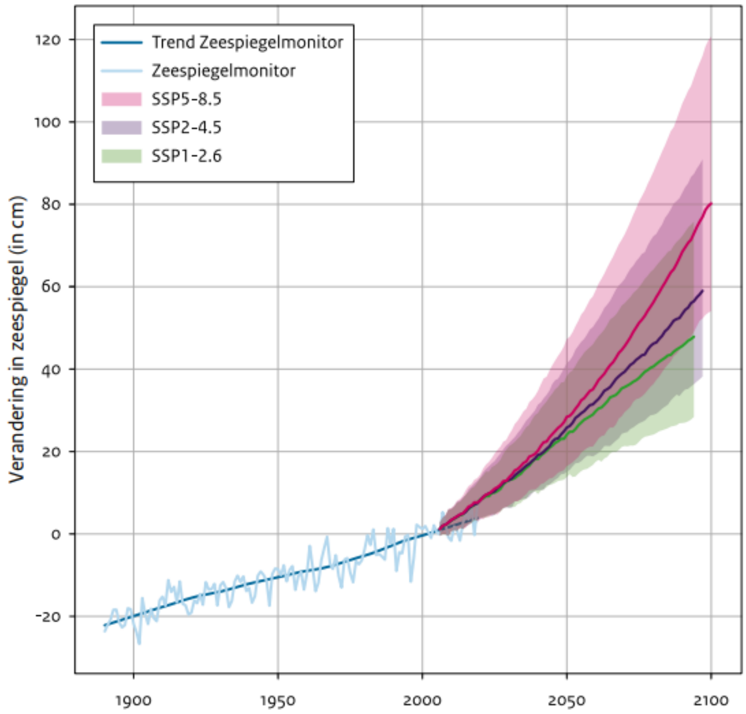

Figure 2.3: Sea level rise Netherlands according to KNMI [28]

The uncertainties in predictions of sea level rise are large, because it depends on how effective measures against for instance greenhouse gasses will be. Additionally, the system is very complex, making the uncertainty surrounding the scenarios large; the uncertainty is larger than the differences between the scenarios. Making a decision on how much sea level rise to account for in 2100 or even further in time is therefore difficult. It is useful to keep thinking in climate scenarios and adapting these scenarios each time more information becomes available.

After the publication of the sixth IPCC report [36], the new estimates for sea level rise along the Dutch coast will be closer to 1 meter sea level rise in 2100. After 2100, sea level rise is expected to increase further. Sea level rise increases the load on the front flood defence and depending on the function of that defence, it could increase the load on the rear defence as well.

The above-mentioned effects of climate change are the only effects that influence flood safety and thus the optimisation as done in this project.The sea level rise will be taken into account when calculating the water levels in the Eastern Scheldt. For the increase in extreme events, it will be explored in this thesis whether a sensitivity analysis can be performed.

2.5. Case study Eastern Scheldt

The optimisation approach as proposed in this thesis will be illustrated with a case study on the Eastern Scheldt. The literature on the Eastern Scheldt is extensive due to the history of flooding and the subsequently constructed Delta works. In this section only relevant literature for this thesis is discussed. The study area is a part of the South-West Delta of the Netherlands. The focus will be on the Eastern Scheldt; which contains the Eastern Scheldt storm surge barrier and dikes behind the barrier.

2.5.1. Eastern Scheldt lay-out

The Eastern Scheldt has a front flood defence; the Eastern Scheldt barrier. The land in the area is protected by dikes. The Eastern Scheldt is cut off from other water bodies by compartment dams; the Grevelingendam, the Philipsdam, the Zandkreekdam and the Oesterdam. The dams have multiple purposes, ranging from dividing fresh and saline water, preventing high water levels, enabling discharge from rivers into the Volkerak and facilitating navigation by removing the tide from the Scheldt-Rhine connection [3].

Figure 2.4: Eastern Scheldt barrier and dams map

2.5.2. Eastern Scheldt storm surge barrier

After the flooding in 1953 in the South-West of the Netherlands, the government decided to construct the Deltaworks, protecting the south western part of the country against flooding. The public sentiment back then was that such floods should never happen again. In making the choice for a design, the government looked at the speed of construction, the flood safety afterwards and the costs. Knowing which is the best economic decision is important for such costly projects. The Deltaworks protect the land by shortening the coastline. The Eastern Scheldt barrier is part of the Deltaworks. Multiple designs were made for the Eastern Scheldt barrier In the end 3 main alternatives were compared; one with a dam (alternative D4), one with a storm surge barrier (alternative C3) and one open option (alternative A3, which thus included heightening the dikes around the Eastern Scheldt significantly). The alternatives were compared to each other based on different aspects; costs, safety, ecology, etc. A number of these factors can be found in Table 2.1. Note that these numbers date from 1976.

Table 2.1: Comparison Easter Scheldt alternatives *Analyse Oosterschelde alternatieven* [3]

Aspect	Dam (D4)	Storm surge barrier (C3)	Open (A3)
Cost (in millions gld)	2.135	4.645	3.620
Ground confiscated (ha)	300	430	742
Yearly maintenance costs (ml gld)	10	25	15
Direct employment (men hours)	11920	26590	18100
Reinforcement opportunity	+	-	–
Length flood defence (km)	9	9	145
Pf in transition period (%)	2.5	3.5	6.5
Recreation	+	0	0
”Optimal” safety	yes	yes	no
Construction finished in year	1980	1985	1994

*costs are set in millions of guilders. One guilder equals 0.45 Euro. The guilder was replaced by Euro on 1 January 1999.

For option A3 (open estuary), the dikes along the Eastern Scheldt all had to be strengthened, therefore this alternative would take too long to construct. The people wanted the project to be finished sooner rather than later. For fishing purposes the dam was the worst alternative, as well as for most of the environmental aspects. However, the dam was the most cost-effective and most safe option. From 1972 until 1974, a closed flood defence design was explored and scale models were constructed. After protests from environmental organisations and fishermen, it was decided to preserve the tidal condition by making a partly open storm surge barrier, even though the preparations for the dam were already in progress. The research for this design alternative was performed from 1974 until 1976 [3], however the location of the storm surge barrier was not re-evaluated. The barrier was placed in exactly the same place as the dam was planned. The construction of the storm surge barrier was finished in 1986.

The Eastern Scheldt storm surge barrier is designed to close in case the predicted water level outside the Eastern Scheldt reaches +3.0 m NAP. The function of the barrier is to prevent high water levels within the Eastern Scheldt while keeping the tidal amplitude largely intact under normal conditions.The barrier is 9 km long, consists of three dams and three sections of steel gates. The dams are about 5 kilometers long. The barrier has 62 steel gates. The gates are part of the moving barrier and are lowered in case of high water levels. The elements of the barrier can be seen in Figure 2.5. Since the Eastern Scheldt barrier is partly open, the original tidal amplitude of 3.5 metres has only reduced to 2.7 metres. However, this reduction still has affected the morphology and ecology in the Eastern Scheldt, see subsection 2.5.5. The design water levels at the seaside are +5.50 m NAP for Roompot and Schaar and +5.30 m NAP for Hammen [49]. In the design for the Eastern Scheldt barrier 30 cm was accounted for; 10 cm for subsidence and 20 cm for sea level rise. Despite subsidence being measured smaller than 10 cm, the extra 30 cm for sea level rise and subsidence is expected to be exceeded around 2050 [49].

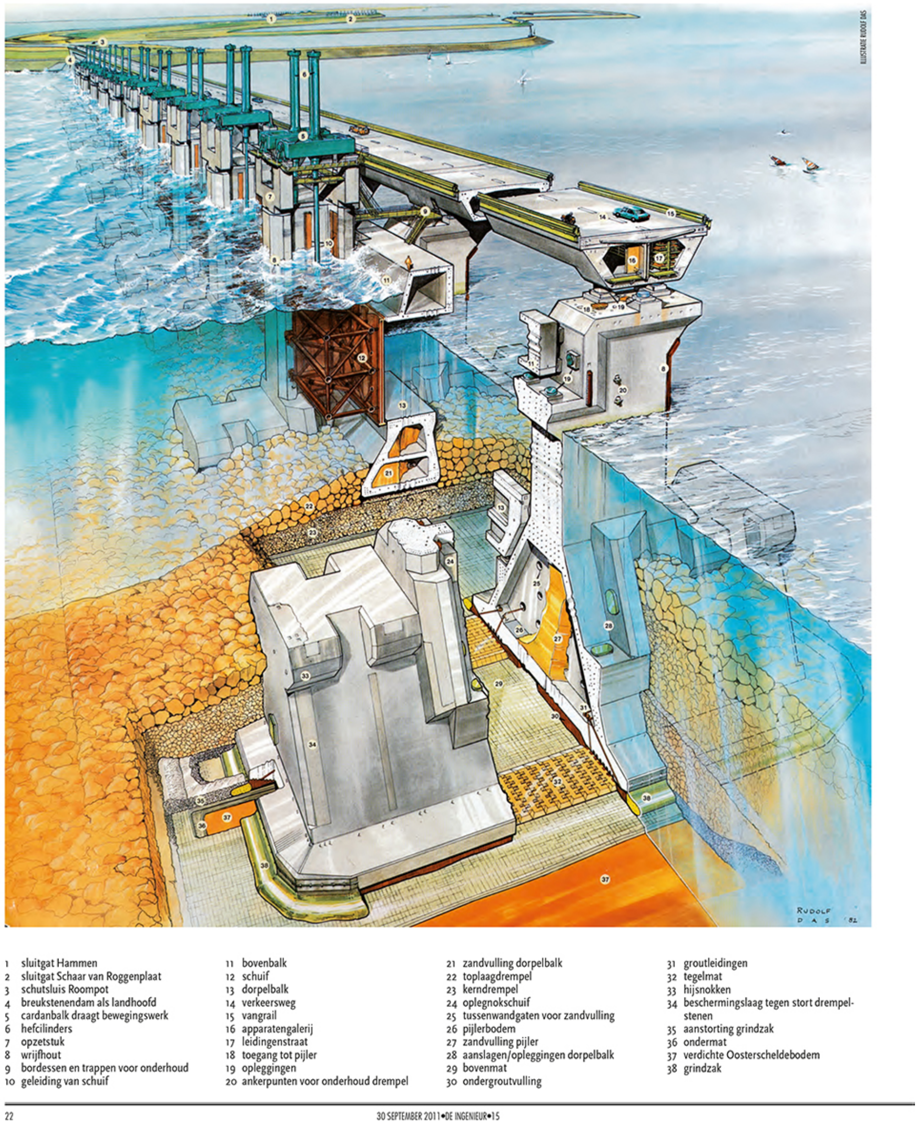

Figure 2.5: Eastern Scheldt barrier elements, [7]

2.5.3. Effect climate change on the Eastern Scheldt

As discussed in section 2.4, climate change causes sea level rise at sea. The question is how does this translate into the Eastern Scheldt through the storm surge barrier? This question has been considered in Zethof et al. [54]. The report comes with a dataset for a sea level rise of 0.5 m and 1.0 meter. The dataset considers several IMPLIC locations for which the hydraulic boundary conditions are calculated. One of the parameters that was considered in the report is an adapted closure water level; for 1 meter sea level rise, a closure water level of 3.25 m + NAP is considered and for 1.5 meter sea level rise a closure water level of 3.5 m + NAP is considered. The heightened closure water level decreases the closure frequency of the barrier.

2.5.4. Storm surge barrier reinforcement options

In this subsection, the options for the Eastern Scheldt storm surge barrier (OSK) will be discussed. A division is made into three types of options:

1. Options that retain the current barrier without any adjustments, only adjustments to the manage-

ment.
2. Options that adapt the current barrier
3. Options that replace the current barrier.

What the best course of action will be will depend on the optimisation of the flood defence system as a whole. The open ocean boundary conditions influence this optimisation by their load upon the barrier.

Management options OSK
The current storm surge barrier is part of the Deltaworks and was intended to shorten the coast line and lower the water levels in the Eastern Scheldt. Large investments have been made to realise the current barrier. Maximising the investment is a favorable path to take to avoid large investments in the near future. There are different options for changing the current barrier if one were to decide not to invest in replacements or improvements any further. The management of the barrier can be adapted to better suit future conditions. The options for changed management are:

- Changing nothing
- Heightening the closure level
- Closing the barrier permanently to avoid closure failure

The OSK has a built-in buffer of 30 centimeters for sea level rise and subsidence. When the sea level rises 30 centimeters, the barrier will not fail, but it will not be up to the standards set for it. Sea level rise will cause the current closure water level to be reached more often. Keeping the closure water level the same will result in closing the barrier more often which will increase the environmental impact. An alternative is to adjust the closing water level to rise with the sea level to prevent the barrier from closing more often. Heightening the closure level results in increased loads on the dikes in the case that the water level at the barrier exceeds +3 m NAP, but the water level has not yet reached the new closure water level. According to a report named ”The effects of sea level rise and sand hunger at the Eastern Scheldt”(*De effecten van zeespiegelstijging en zandhonger op de Oosterschelde* [14]), if the closure water level of the barrier remains the same, then the closing frequency of the barrier will increase drastically, see Table 2.2.

Table 2.2: Closure frequency

Scenario	Closure frequency [per year]
SSP1-2.6	1.7-22
SSP 2-4.5	4.5-85
SSP5- 5-8.5	4.5-228

In comparison, the current closure frequency is 0.8 times a year. According to Zethof et al. [54], in case of 1,0 m SLR, with an unchanged closure level, this will result in a closure frequency of 45 times per year. Heightening the closure level with 25 centimeters in case of 1.0 m SLR will result in a closure frequency of 11 times per year. Heightening the closure level will preserve the tide in the Eastern Scheldt. Preserving the tide in the Eastern Scheldt was originally the reason why the barrier was chosen instead of the dam. Preserving the tide would be beneficial to the ecology in the Eastern Scheldt. In case the closure water level is not heightened, the barrier will be closed large parts of the winter, since most high water levels occur in that period. The choice can be made to keep the same closure water level, but this will be at the cost of the tide and therefore the ecology and sand balance. The last option (permanent closure of the barrier) results in the same disadvantages as keeping the 3.0 m + NAP closure water level, only this option guarantees the presence of these disadvantages all year round. The failure probability regarding failure to close will become insignificantly small if the barrier is permanently closed. The function of the barrier as currently stated enables the Eastern Scheldt to remain a salt water body. The last two options, closing the barrier (semi-)permanently, will remove almost all or all of the tide in the Eastern Scheldt. The function of the barrier needs to be redefined if permanent closure of the barrier is to be considered. The value of the barrier for fishermen and environmental circumstances will decrease due to its inability to preserve the tidal amplitude.

Adaptation options OSK

Another way to optimise the investment done in the barrier is to make adaptations to the barrier to improve its suitability for future conditions. The options to adapt the current barrier are as follows:

- Heightening the sill to better close the barrier
- Heightening the gates and the upper beam
- Improving the closure reliability

The first option to adapt the current barrier is to heighten the sill. Heightening the sill will result in better closure of the barrier, decreasing the leak discharge. The barrier currently leaks, since there is a gap between the gate and the sill when the barrier is in closed position. The discharge leaking through the barrier could be decreased by closing this gap by heightening the sill. According to Zethof et al. [54], reducing the leak discharge with 50% will have a negligible effect on the hydraulic conditions in the Eastern Scheldt. Reducing the leak discharge is thus not important to include in optimisation. The second option, heightening the gates and heightening the upper beam, is meant to be able to withstand higher sea levels in case of sea level rise. However, the possibilities to increase the height of the gates and upper beam will be limited. The modelling options for such reinforcements are limited, since most models assumed a barrier that cannot be altered. The third option is to improve the closure reliability of the barrier. The goal is to decrease the failure probability in case closure of the barrier is needed. The failure probability per closure is defined as how many gates fail, there is no singular value for the failure probability per closure. The failure probability can be seen in Appendix B.

Replacement options OSK

The optimisation of Eastern Scheldt could result in the conclusion that either the storm surge barrier cannot withstand the loads as calculated for the future or that replacing the barrier by a different option is the optimal solution. Possible options for replacing the current storm surge barrier are as follows:

1. Building a dam instead of the barrier
2. Removing the barrier to restore the sediment balance

The first replacement option for the barrier is building a dam. The dam was the option that was initially selected by the government before the protest by environmental organisations and the fisheries [3]. The dam would have been a very good option economically, but the environmental impact would have been large [3]. If it were to be decided that constructing a dam is the best option, the current Eastern Scheldt barrier would have to be demolished before a new dam can be build. A dam would be able to withstand a large sea level rise because it can be heightened. The dam would shorten the coast line, making large reinforcements in the dikes along the Eastern Scheldt unnecessary. However, there are various reasons why a dam is likely not to be chosen. Firstly, the storm surge barrier keeps the ecology in the Eastern Scheldt largely intact, which a dam could not. A dam would not be able to preserve the tidal motion within the Eastern Scheldt. Secondly, the return on investment of the storm surge barrier will be lower due to the shorter life time than anticipated. The barrier has sunk costs that need to be earned back. And lastly, public opinion is likely to be against a dam. When the barrier was planned, the government first opted for a dam, which was blocked by the public. Replacing the barrier by a dam will likely be met with the same lack of enthusiasm.
Second option: Removing the barrier to restore the sediment balance. The construction of the barrier decreased the tidal prism, causing the sediment import to decrease. The decrease prevents the intertidal areas to grow with sea level rise [53]. Removing the barrier can contribute to resolving these problems. The hope is that removing the barrier will allow the intertidal areas to grow with sea level rise and increase the wave dissipation at the shore. However, there is no guarantee that the sediment balance will be restored. Even if it is restored, the intertidal areas might remain in the same state they are in currently. To build them up again, a net positive sediment transport gradient needs occur. Since the net transport volumes are much smaller than gross transport volumes, estimating the effect on the net sediment transport is difficult. Additionally, the barrier prevents high water levels in the Eastern Scheldt. The dikes surrounding the Eastern Scheldt will have to be reinforced heavily, in order to maintain the same level of protection against sea level rise.

2.5.5. Ecology
The Eastern Scheldt is a Natura 2000 area. The area is an important habitat for flora and fauna. Birds rest on the intertidal areas during their yearly migration or nest in these areas [48]. The intertidal areas are also important for the fishing industry. The ecological importance of the area have tipped the balance in the favor of the storm surge barrier instead of a dam.

Morphology
The choice for the storm surge barrier instead of the dam was to preserve the tide and to minimise the disruption of the fishing industry. However, preserving the vertical tidal motion was not enough to maintain the sediment balance; the sediment import is physically blocked by the sill of the barrier. For sediment to be imported into the Eastern Scheldt, it needs to pass a very high sill in order to be transported into the Eastern Scheldt, which proves to be nearly impossible. Since the construction of the compartment dams and the partial blockage by the storm surge barrier, the sediment import decreased dramatically. The sediment transport equilibrium is disturbed. The tidal channels still require sediment, but since the sediment import has decreased significantly, the tidal flats have eroded [10]. Reduction of the flats is undesirable from an ecological point of view (Jongeling [26]). Another consequence of the deterioration of the intertidal areas is the decrease of wave dissipation in the Eastern Scheldt. The waves dissipate energy on the intertidal areas, resulting in lower wave forcing on the dikes. Keeping the Eastern Scheldt barrier will result in losing more intertidal areas. Looking from this vantage point, it can be argued that removing the barrier will result in higher ecological value and more wave dissipation in front of the dikes [53]. However, the flood protection that the barrier offers by reducing water levels and waves in the Eastern Scheldt will be gone as well. As mentioned before, removing the barrier will not guarantee a restored sediment balance. Applying this strategy would consequentially mean the dikes around the Eastern Scheldt have to be improved and heightened significantly, which results in high costs and takes a long time.

Ebb delta
The ebb delta is the shallow sea in front of the delta. The shallowness decreases wave forces on the storm surge barrier. The existence of the ebb delta is the result of sediment transport. A transport flux discharging into the ocean from the Eastern Scheldt, causing deposition of sand due to decreasing flow velocity as the flow diverges. The water at the ebb delta is shallow, causing waves to break on the banks, transporting the sand back inland and towards the coast. If one of the two sediment transport fluxes is interrupted, the ebb delta either increases or decreases in size. When the Deltaworks were constructed, the transport flux from the Eastern Scheldt to the ebb delta was blocked, decreasing the sediment influx to the ebb delta. Since the construction of the Deltaworks, the sediment transport supplying the ebb delta has decreased, causing the ebb delta to decrease in size. The sand present at the ebb delta is still being transported away by an alongshore current flowing south to north. The ability to decrease wave forces on the storm surge barrier decreases as well due to this development [38]. These morphological processes do influence the forces acting upon the barrier and the dikes in the Eastern Scheldt, but the processes are complex. Incorporating the processes into the model would be difficult due to quantification problems. Therefore, morphology will be debated exclusively qualitatively.

2.5.6. Reliability of the dikes
In this section calculating the reliability of dikes will be discussed. Firstly, the failure mechanisms that are generally considered for dikes are discussed, after which the reinforcement options for the dikes are discussed.

Failure mechanisms dikes
In order to calculate the failure probability of a dike, it is necessary to know the various ways the dike can fail. These ways are called failure mechanisms. Multiple failure mechanisms exist for dikes. The most important failure mechanisms are listed below [30]:

- Overflow
- Overtopping
- Micro-instability

- Macro-instability in waterside slope
- Macro-instability in land side slope
- Uplift and piping
- Erosion of waterside slope

In Figure 2.6, the failure mechanisms are visualised. According to Vorogushyn, Merz, and Apel [50] piping, slope failure, overtopping and macro-instability were identified as the dominant failure mechanisms for dikes.

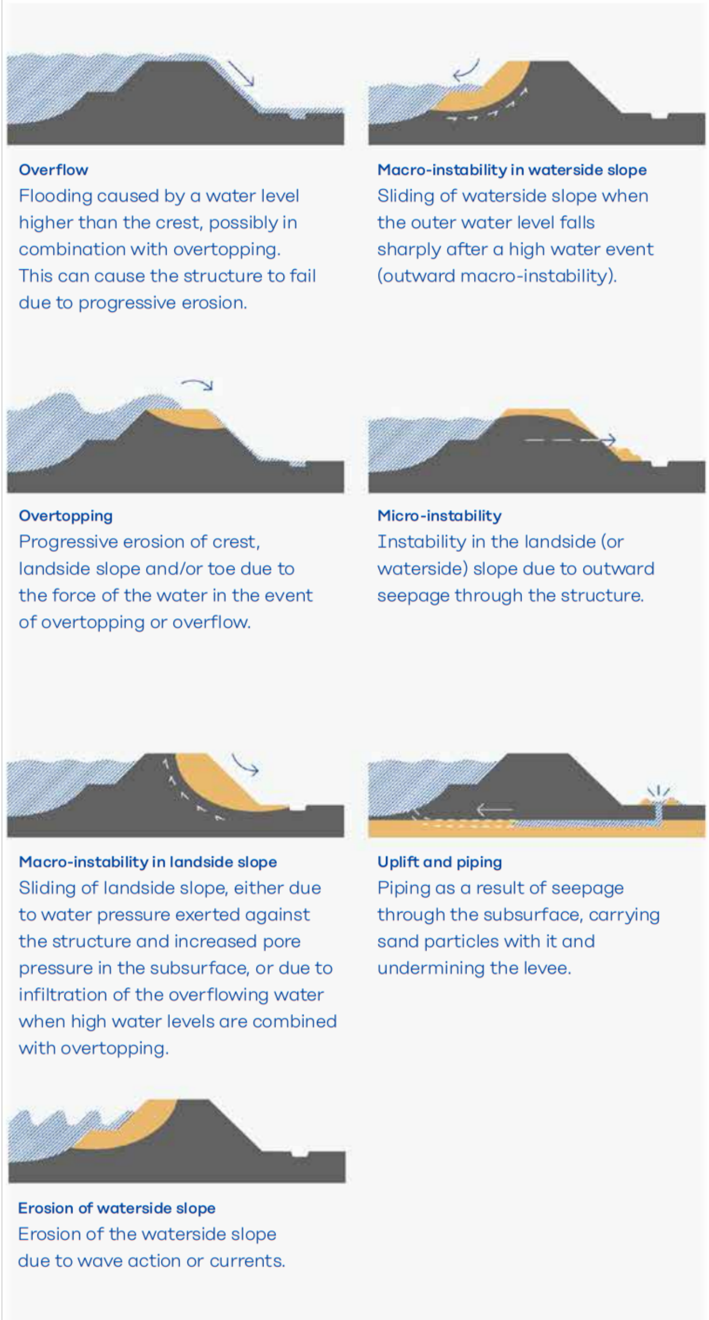

Figure 2.6: Failure mechanisms earthen dikes, [30]

A dike segment consists of multiple dike sections and other flood defence structures. A flood occurs if one of the elements in the dike segment fails [30]. The flood probability increases if a longer segment is considered. This is called the length effect. The length effect differs per failure mode, since some

failure modes are correlated. For example, overtopping depends on hydraulic boundary conditions which are very similar for dike sections situated next to each other. Soil properties however, can change over short distances and thus the risk of piping or instabilities can differ over short distances.
Once the failure probability of a dike section is determined, this probability is divided over the different failure modes. This is called the failure budget which is summarised in Table 2.3. The division of the failure budget has been determined by the Dutch Ministry of Infrastructure and Environment in 2016 [39]. The dike section can still meet the failure probability target when one of the budgets set for a failure mechanism is exceeded, provided such failure is compensated by another mechanism.

Table 2.3: Failure budget per failure mode [23]

Failure mode	Code	Failure budget
Height structure or gras cover erosion crest and inner slope	HTKW + GEKB	0.24
Piping	STPH	0.24
Macro stability inner slope	STBI	0.04
Gras erosion outer slope	GEBU	0.05
Other cover outer slope	-	0.05
Reliability closure structure	BSKW	0.04
Piping at structure	PKW	0.02
Strength and stability structure	STKWp	0.02
Dune erosion	DA	0.1
Other	-	0.2

The assessment as executed for the Eastern Scheldt is shown in Appendix A. The safety targets, the assessment per dike section and the spatial variation of weaker parts are set out here.

Failure probability correlation between dike sections
The correlation of failure probability between dike sections differs per failure mode. Soil conditions tend to be spatially heterogeneous, therefore failure due to piping erosion or macro stability shows low correlation between various sections, while overtopping has a high correlation between different dike sections due to relatively homogeneous hydrodynamic conditions. The dike sections considered are reduced to a single cross section, representative for that section. Klerk et al. [27] combines the information of different dike sections, resulting in a segment reliability. This method results in a total cost optimisation per dike segment.

Sand nourishments as a reinforcement measure
The effects of sea level rise could also be negated with sand nourishments.The sand nourishments will enable the intertidal areas to grow with sea level rise. Intertidal areas increase the wave dissipation before arriving at the dike. The sand nourishments will need to be substantial in volume if they are to grow simultaneously with sea level rise. The intertidal areas are currently decreasing in volume, so in order to allow them to grow with sea level rise, the volumes of nourished sand need to be extremely large. Sand nourishments have been applied at the Roggenplaat [52]. The research revealed that the number of benthic animals plummeted in the areas that were nourished with a thickness of more than a meter. To avoid negative effects in biodiversity, the thickness of sand nourishments has to be limited. The conclusion is that sand nourishments are not a good option to counteract sea level rise, but can assist in limiting the effect of decreasing intertidal areas.

2.6. Identified knowledge gap

Summarising this chapter, a knowledge gap can be identified. Firstly, the optimisation of a flood defence system with multiple lines of defence is currently based on two independently considered elements. The safety targets and functional requirements are defined separately for both flood defences [24], without taking changing interactions between the flood defences into account. Optimisation of the two elements separately is not wrong, but the approach is more likely to result in a conservative estimation of the strength of one barrier to ensure the safety of the other defence. Optimisation of the system as a whole can result in a better estimate of system safety as a whole against lower costs. The interaction between the two flood defences needs yet to be modelled correctly for an optimisation. Secondly, the development of system strength over time needs to be modelled in the optimisation. If one of the defences deteriorates faster than the other, this might influence the optimisation of reinforcements of the entire system. Thirdly, in order to optimise over a longer timescale than 50 years, the significant changes in boundary conditions due to climate change must be taken into account. If an optimisation aims to optimise reinforcements for a period over 100 years, the change in boundary conditions is a significant factor that cannot be ignored. Quantifying the change in boundary conditions in the future leads to large uncertainties. Therefore, a strategy needs to be chosen to encompass those uncertainties. Two strategies were proposed in the literature; robust decision making and the adaptation pathways approach. Both will be explored in order to assess which approach is the most sensible to use in a system optimisation of flood defences.

3 Methodology

The problem of this thesis is defined and outlined in Chapter 1. In Chapter 2 , the knowledge gap in existing literature was discussed. In this Chapter 3, the methodology used to answer the research questions will be explained.

This thesis in general has two parts: the general part on the optimisation method and the case study that is done on the Eastern Scheldt. The case study is used to illustrate what steps are taken in the optimisation. This chapter will first discuss the type of system the optimisation is meant for. Next, the steps taken will be discussed.

3.1. Modelling of the system

The type of system considered in this thesis is a flood defence system with multiple lines of flood defences. The number of lines that are included in this thesis is two. The complexity of the optimisation increases if more lines of defence are included, because all flood defences influence each other boundary conditions. In order to grasp the main elements of the flood defence system, the system is simplified. The simplification is shown in Figure 3.1. The assumption is that one element influences the next, but the interaction does not work in reverse, i.e. the barrier influences the boundary conditions in the Eastern Scheldt, but the boundary conditions in the Eastern Scheldt do not influence the barrier in any way. In reality, this could be false, because if a dike breaches, the water levels could decrease behind the first defence. However, the assumption is made that the water body connecting the two lines of defence is large enough not to be influenced by this effect. The interaction is shown in Figure 3.1.

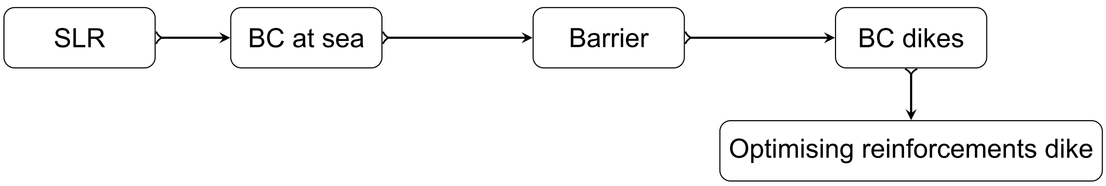

Figure 3.1: Flowchart simplification flood defence system

To be able to determine the performance of both flood defences in relation to each other, the influence of the first flood defence on the dikes needs to be captured. The interaction is included in the boundary conditions(BC) in the Eastern Scheldt. These boundary conditions are influenced by two elements:

- Boundary conditions at sea
- Performance of the barrier

The boundary conditions at sea can change due to climate change, which in turn will change the boundary conditions in the Eastern Scheldt. In what manner and to what extent the boundary conditions within the Eastern Scheldt change due to climate change depends on how well the barrier performs. If the barrier does not fail and closes well, the change in boundary conditions in the Eastern Scheldt will be limited. Due to the interconnection of the elements of the flood defence system, it is important to work from the outside to the inside of the system, starting with the boundary conditions at sea and ending with the reinforcements of the dikes.

3.2. Steps in the optimisation method

The optimisation can be described in multiple steps. These steps are as follows:

1. Determine the sea level rise
2. Determine possible options for the barrier
3. Calculate the hydraulic boundary conditions resulting from (1) and (2) at the dikes
4. Determine the relevant failure mechanisms at the dikes
5. Determine the reinforcements that reduce the failure probability for these failure mechanisms
6. Calculate the costs associated with these reinforcements
7. Calculate the resulting flood risk after the reinforcements are implemented
8. optimise investment costs and resulting flood risk to find the required reinforcements associated with the boundary conditions
9. Repeat the process for another combination of SLR and barrier option
10. Analyse the results to advise on the best option

In the following subsections, each step is considered and the choices made are discussed. The steps can be coupled to the sub-questions as posed earlier in section 1.4. The question on how to model the interaction between the two flood defences is answered in step 3. The uncertainty in future boundary conditions is captured in the first step. Next, the question on the reinforcement options for both flood defences is answered by step 2 and 5. The selection of the optimal strategy will be answered in steps 8 and 10.

3.2.1. Sea level rise and time period considered

The first step is quantifying the sea level rise. The amount of sea level rise included depends on the time period considered. In this thesis, the time period considered is 100 years. Shorter time periods would cause reinforcements carried out in the system to be less valuable. Considering larger time periods increases the uncertainty in sea level rise, increasing the uncertainty of the optimisation. Shortening the time period would therefore reduce the number of required reinforcements but lengthening the time period would introduce more uncertainty in the results. After the time period to be considered is chosen, the amount of sea level rise that will be considered should be determined. Sea level rise has been considered in section 2.4, in which scenarios were discussed that range from 30 cm - 121 cm of sea level rise. The expectation however is that these scenarios need to be changed to higher levels of sea level rise after the publication of the last IPCC report [36]. In order to not to have to rely on a single sea level rise prediction, it is important to include multiple levels of sea level rise. The optimisation will be conducted for different levels of sea level rise to be able to compare results. The sea level rise levels that are included in the calculation are 50 cm and 100 cm. For sea level rise greater than 100 centimeter, no reliable boundary conditions can be estimated with the current available software, see chapter 4.

3.2.2. Efficiency barrier and boundary conditions at the dikes

After determining the time period and the considered levels of sea level rise, the boundary conditions at sea need to be translated to boundary conditions at the dikes. The two main elements influencing the boundary conditions at the dikes are the boundary conditions at sea and the performance of the barrier. Since the optimisation aims to optimise the flood defence system, not only reinforcements at the dikes, but also the possible options for changes at the barrier need to considered. The performance of the barrier is influenced by the changed made to it. It is therefore necessary to determine which barrier options will be considered in the optimisation and to calculate how these changes influence the boundary conditions at the dikes. In this thesis the choice has been made to use a software application to calculate the boundary conditions at the dikes. Using a software application saves time and facilitates the decision making process. The software application used in this thesis will be described in chapter 4.

3.2.3. Failure mechanisms at the dikes and associated reinforcements

After calculating the boundary conditions at the dikes, the possible reinforcements to the dikes need to be determined. Firstly, the dominant failure mechanisms of the dikes need to be determined. Including all relevant failure mechanisms is important in order to make the solution space large enough to ensure an accurate solution. However it is only relevant to include a failure mechanism if the failure mechanism contributes significantly to the total failure probability. In that case a reinforcement should be considered. If a failure mechanism contributes very little to the flood probability, the investment costs to reduce that flood probability should be very low for the reinforcement to be optimal. Since investment costs are usually not low, reinforcements that contribute little to reducing the flood probability will not be considered. Such a failure mechanism is therefore not sufficiently relevant to include in the optimisation of reinforcements. Therefore, it is important to first be determine which failure mechanisms contribute significantly to the total failure probability. Once that determination has been executed, the possible reinforcements that can reduce the failure probability should be identified, along with the investment costs. For the calculation of investment costs, it is important to choose the correct number of reinforcement options. For example, looking at dike heightening, it is important to choose the intervals with which such reinforcement can be executed. An interval should be small enough to be accurate, but large enough to be able to construct. For example, applying a 10 cm interval for dike heightening will provide an accurate analysis, but will not be feasible to construct accurately. The final step is to calculate the resulting flood risk after the reinforcement has been carried out. As mentioned before, flood risk is the failure probability of the flood defence multiplied by the expected damage. The expected damage includes monetised fatalities as well as economic value.

The equation for calculating the failure probability for a segment for multiple failure mechanisms is [27]:

$$P_f(t) = 1 - \prod_{m \epsilon M}(1 - P_{f,m,segment}(t)) \tag{3.1}$$

In this thesis the failure probability of a dike segment consists of the sum of the failure probabilities of all failure mechanisms. The failure mechanisms are assumed to be independent from each other. The failure probabilities are very small, therefore this assumption does not deviate far from reality. If the dependency between the failure mechanisms would have been included, the failure probability would be even smaller than in the case the failure probabilities are summed.

3.2.4. Dependency of dike segments on each other

The vulnerability of a dike is either expressed as a failure probability, i.e. the probability that a dike fails in a year, or in a reliability index β. According to Rosowsky [43] the reliability index is usually formulated as:

$$\beta \approx \phi^{-1} * (1 - P_f) \tag{3.2}$$

Where ϕ^{-1} is the inverse standard normal CDF and P_f is the failure probability.

Every dike segment is assumed to impact a different area of land and thus have its own damage number. If the optimisation would be done over smaller dike segments, these would protect the same area of land and the failure probability would need to be combined differently.

3.2.5. Optimisation of reinforcements

The next step is optimising the possible reinforcement to the dikes. The function used to optimise is Equation 2.3: minimising the total costs. The resulting costs are conditional to the sea level rise and the barrier option chosen as input. These two factor determine the boundary conditions at the dikes, which in turn are used to weigh the investment costs of reinforcements against the resulting flood risk. The reinforcement option with the lowest total cost (= investment cost + flood risk) is then chosen as the optimal reinforcement for that failure mechanism. The optimal solution could also result in the decision not to invest in reinforcements for that failure mechanism. This could be the case if the flood risk for that failure mechanism is low or if the investment costs are too high compared to the reduction of flood risk. To be able to compare costs, all investment costs and flood risks are discounted back to the present value. The summation of these costs will reflect the cost of the boundary conditions that were created. If a different sea level rise or barrier option is chosen, both the total costs as well as the optimal reinforcements change.

As mentioned in section 2.2, the net present value(NPV) of costs can be discounted to the present using the following formula:

$$NPV = \frac{C_t}{(1+r)^t} \tag{3.3}$$

In which:

- NPV: net present value
- C_t: costs at time t
- r: discount rate
- t: time

This means that the costs need to be associated with a certain year in which the costs are incurred. For investment costs this is usually straightforward: the year used in the calculation is the year in which the investment is done. In this thesis this will be t = 0. Discounting flood risk to present value depends on the character of the flood risk. If the flood risk remains the same for all years considered, it can be discounted to the present by using the following formula [22]:

$$PV = \frac{FR}{r} * (1 - \frac{1}{(1+r)^n}) \tag{3.4}$$

In which:

- FR: Flood risk per year
- r: Discount rate
- n: Total number of years

3.3. Analysis

Finally, the results of the optimisation in the different scenarios have to be analysed. Firstly, an analysis is done to consider the effect of sea level rise without any changes to the storm surge barrier. The investment costs and flood risk of any scenario can be calculated. The investments can be allocated to the different failure mechanisms to see which one contributes the most to the investment costs or to flood risk. Secondly, the best option for reinforcement of the storm surge barrier can be calculated. Since the calculation is dependent on sea level rise, the optimal choice for the barrier has to be calculated within this context. Lastly, it could be argued which barrier option might be best for the future. This could be done by defining which reinforcement option is a sound choice in most of the scenarios. Another way of examining the results of the model is by drawing an adaptation pathways map for the dike and sketching the different options at various points in time. Adopting this approach prevents taking a path that makes the necessary reinforcements for higher sea level rise in the future impossible.

3.4. Scope

This thesis is focused on finding a method to optimise costs for reinforcements of a flood defence system with multiple lines of defences while taking future sea level rise into account. The scope of this thesis will be further highlighted in this section.

First of all, not all possible failure mechanisms are included in the conceptual model of this thesis. Only the failure mechanisms whose failure probability was deemed significant to the total failure probability are included. If more failure mechanisms were to be included, the model would become more complicated, while the benefit of adding another failure mechanism would be very small since it likely does not contribute much to the failure probability.

The connection between the barrier and the dikes along the Eastern Scheldt is captured in the response of the hydraulic boundary conditions at the dikes. There are other consequences that are excluded from this analysis when merely looking at the response of the boundary conditions. Firstly, the response of the morphology within the Eastern Scheldt and the ebb tidal delta is not taken into consideration. Secondly, the frequency with which the barrier has to close in certain scenarios can increase. If one were to choose not to increase the closure water level of the barrier in case of 1 m sea level rise, the closure frequency of the barrier will increase drastically. The inflow of salt water from sea will decrease which will have an effect on the flora and fauna within the Eastern Scheldt. Additionally, the barrier itself

may deteriorate faster than previously calculated and may need to be replaced sooner compared to a situation where the closure water level is heightened and the closure frequency is not increased much.
Thirdly, the damage in case of a flood is simplified to a single number. For the model in this thesis it does not matter which failure mechanism occurs; the damage is assumed to be the same in case of flooding. In reality it does matter what failure mechanism occurs; the water level at overtopping failure is for instance usually higher than for other failure mechanisms. For instance, revetment failure does not lead to an immediate breach in the dike. Moreover, in reality it matters where the dike failure occurs, as evacuation rates and speed of flooding are influenced by it, which will both have an immediate effect on the damages resulting from the failure.
Lastly, the model optimises costs of reinforcement measures only. Any element that needs to be inserted into the model, needs to be translated into costs. It is therefore difficult to translate elements such as ecology or public sentiment into costs to be used in the model. The model only balances costs of investments versus flood risk. The model can therefore not be used as the sole basis of an investment decision, but can be a useful tool to assist in the decision making process.

4 Hydraulic boundary conditions with sea level rise

4.1. Choice of program for boundary conditions

The optimisation of the flood defence system needs to couple the storm surge barrier to the primary flood defence. The storm surge barrier combined with the conditions at sea determine the loads upon the dikes along the Eastern Scheldt. The choice of the program to capture these boundary conditions in the Eastern Scheldt is therefore important to accurately capture the influence of the barrier up on the dikes. The first program that was considered for this thesis was Hydra-Ring. This program could capture the hydraulic boundary conditions well in the current state and with sea level rise. However the boundary conditions in the Eastern Scheldt are computed using a preprocessor for the barrier. Changing the influence of the barrier, i.e. through a heightened closure water level, would not be possible using the Hydra-Ring program. Therefore it was decided to use Hydra-NL for the hydraulic boundary conditions. The influence of the barrier can be changed slightly more in Hydra-NL and through databases obtained for the program. Version 2.8.2 of Hydra-NL is used. In the next sections, the Hydra-NL program is described as well as the output of that program. The influence that sea level rise has on the boundary conditions within the Eastern Scheldt is also discussed.

4.2. Background Hydra-NL

Hydra-NL is a probabilistic model that is based on statistics. In the case of the Eastern Scheldt, these statistics are obtained from the performance levels (*'prestatiepeilen'*) model which calculates the exceedance frequency of the local water level for a limited number of locations in the Eastern Scheldt. The calculations are performed on the data of a database for the hydraulic conditions in the Eastern Scheldt created with the program IMPLIC. Given a certain barrier configuration and outside conditions, the water level is calculated by IMPLIC for different locations in the Eastern Scheldt. Combining the data with the probability of occurrence of a certain configuration and outside conditions, the performance levels (*'prestatiepeilen'*) model can devise the exceedance frequency lines [16]. Hydra-NL was previously used for performing the dike assessments [21]. The hydraulic loads that can be obtained from the program are:

- Water level
- Wave height and peak period
- Hydraulic load level
- Overtopping discharge

Hydra-NL is adapted to different types of water bodies, with each its own statistics [15]. There are eight different types, for the Eastern Scheldt the type 'Estuary with barrier' is applicable. The water levels in the Eastern Scheldt are not only dependent on the wind and water level at sea, but also on the barrier, the tide and the tidal phase. The barrier influences the water level in the Eastern Scheldt by the way it closes (emergency closure or manual closure) and the failure probability per closure. The program can be used in three different types of modes; an assessment mode, a design mode and the test mode. In this thesis the test mode is used, because more input can be adapted in the test mode and more output options are possible. To be able to use Hydra-NL, a boundary condition database must be selected beforehand. To be able to use the program for the Eastern Scheldt, the information on closure of the barrier must be available to insert into the Hydra-NL program. A table with the probability of closure failure is included, as well as a table with the probability that closure of the barrier is attempted and what kind of closure will be done. The program can calculate accurately for return periods up to 100,000 years, for longer return periods than 100,000 years a warning is included that the output will become less accurate. In case of an assessment of the required height of a dike section, the assessment can be done by using the 2 % run up or by defining a critical overtopping discharge and checking this against a combined overtopping and overflow failure mechanism. For revetment, the significant wave height can be calculated to assess whether the revetment satisfies the requirements.

4.3. Limitations Hydra-NL

For the case study in this thesis there are three main limitations for the application of Hydra-NL. Firstly, sea level rise needs to be included in the calculations in Hydra-NL. It can be set as a parameter in the program, but this is limited to about 1 meter. The original database does not contain data for higher sea level rise than 1 meter. This means that the case study optimisation is limited to 1 meter sea level rise, causing the time scale to be limited as well. The Sea Level Rise Knowledge Program (*'Kennisprogramma Zeespiegelstijging'*) includes a new dataset with 1.5 m sea level rise and a closure water level that is 0.5 m higher than the closure water level of the original database. Calculations can be performed with this dataset, but since no dataset is available without a heightened closure water level, the effect of the heightened closure water level cannot be calculated.

The second limitation for the case study concerns the closure water level of the barrier. The closure water level of the barrier is currently 3 + NAP, but could be heightened in order to reduce the closure frequency when sea level rise occurs. However, the closure water level of the Eastern Scheldt barrier is not a parameter that can be changed as input into the Hydra-NL program. If a changed closure water level needs to be implemented into Hydra-NL, a new dataset is needed. The Sea Level Rise Knowledge Program (*'Kennisprogramma zeespiegelstijging'*)[54] also includes scenario's with a changed closure level; for 1 meter sea level rise, the closure level becomes 3.25 m + NAP and for 1.5 m sea level rise it becomes 3.5 m + NAP. The implication of this however is that only for 1 meter sea level rise the closure water level can be heightened and the new closure water level is already set in the database and cannot be changed. The closure water level can therefore not be changed for 0 or 0.5 m sea level rise.

Thirdly, structural changes to the barrier cannot be implemented in the program of Hydra-NL. The program is based on a specific storm surge barrier. If such data on other structures is desired, then the underlying IMPLIC dataset needs to be changed. Therefore not all options for reinforcement of the barrier that are imaginable (e.g. replacement by a dam) can be fed into Hydra-NL.

Lastly, the data output is highly dependent on the IMPLIC dataset. If any errors exist in the dataset, Hydra-NL will continue to build on those errors. The limitations of Hydra-NL also highly depend on the quality and limitations of the dataset used.

4.4. Effect of SLR on the hydraulic boundary conditions within the Eastern Scheldt

In this section the effect of sea level rise will be discussed. Since the optimisation is strongly linked to Hydra-NL datasets, the choice was made to use the same sea level rise increments as the Hydra-NL datasets; 0.5 meter and 1.0 meter sea level rise. The data resulting from Hydra-NL will be most accurate in this way. 0 m sea level rise will be included in the analysis to calculate the difference that sea level rise makes in total costs. Firstly, we look at the influence of sea level rise on the water level in the Eastern Scheldt. In Figure 4.1, the water level change within the Eastern Scheldt due to sea level rise at Colijnsplaat and Goese Sas is shown.

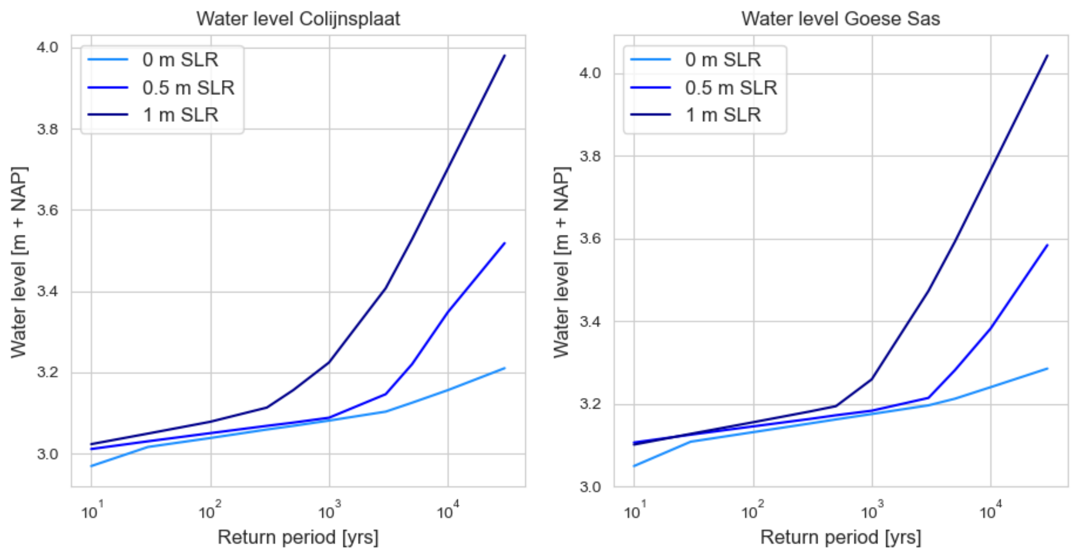

Figure 4.1: Water level at Colijnsplaat and Goese Sas

Figure 4.1 sets out the increased water levels at both Colijnsplaat and Goese Sas in case of higher sea levels. Figure 4.2 depicts the location of these IMPLIC stations. It is important to note that 0.5 m SLR at sea is not equal to 0.5 m increase in water level in the Eastern Scheldt. The barrier dampens the increase in water level at sea to the Eastern Scheldt, causing the increase in water level in the Eastern Scheldt to be smaller.

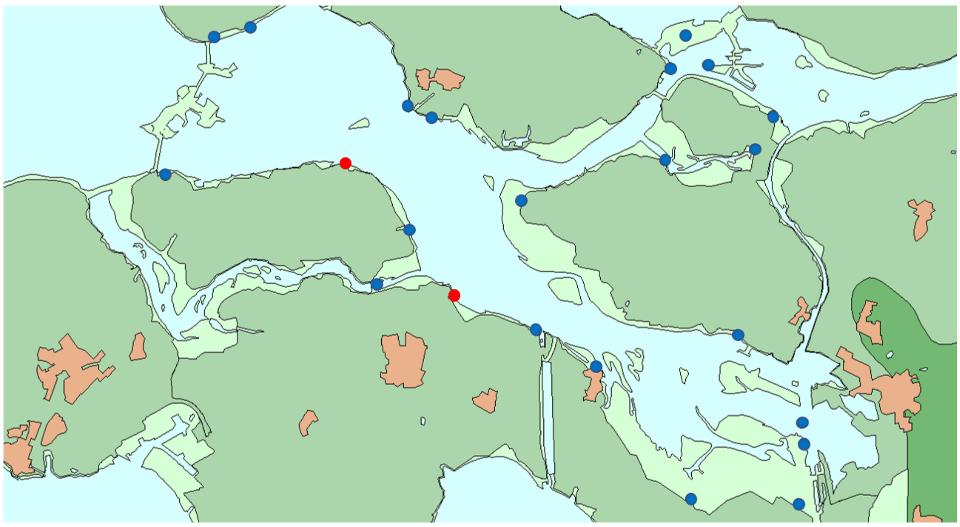

Figure 4.2: Location of Colijnsplaat and Goese Sas within the Eastern Scheldt

In Figure 4.3, the wave height versus return period for different levels of sea level rise is shown for the locations Colijnsplaat and Goese Sas.

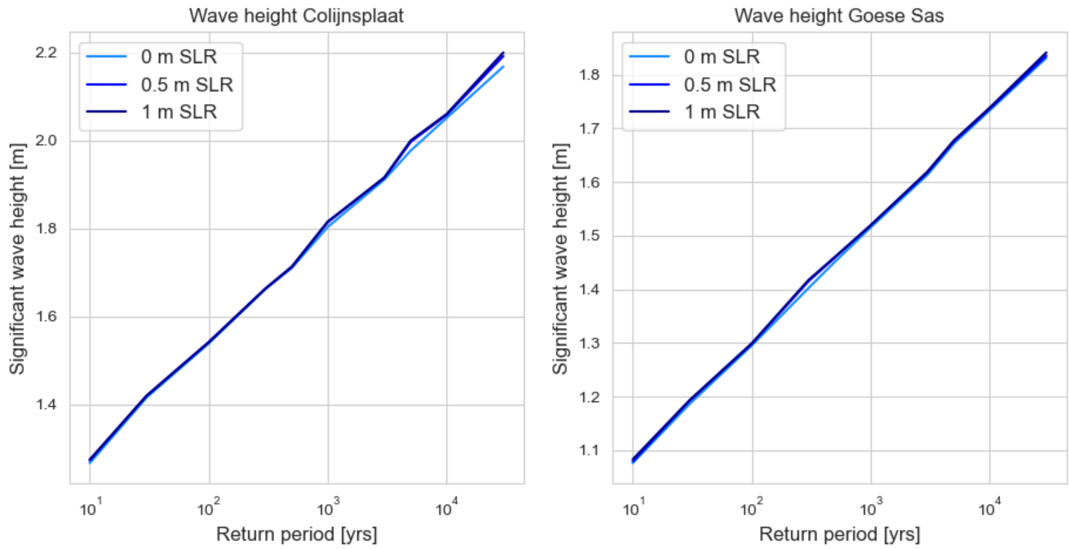

Figure 4.3: Wave height at Colijnsplaat and Goese Sas

Figure 4.3 shows that the wave height does not change when sea level rise occurs. This is an indication that the waves within the Eastern Scheldt are not depth limited. Increasing water levels result in increasing water depth. If the waves would have been depth limited, the wave height would have increased with increasing water levels. Therefore sea level rise does not have an effect on the wave height. Since the water level within the Eastern Scheldt does increase when sea level rise occurs, the point of impact of the waves will change; this point will be situated higher upon the dike. This indicates that the forces of the waves do not change, only the location. Since the waves and water levels determine the loads upon the dikes, they contribute to determining the flood probability. Knowing that the wave height does not change is important information in deciding on effective reinforcements for changing hydraulic loads due to sea level rise.

5
Case study

This chapter shows how the methodology is applied to the case study of the Eastern Scheldt in this thesis in more detail. Firstly, the barrier strategies included in the optimisation are considered. Secondly, the IMPLIC stations included in the case study are discussed, after which the calculation of flood risk is considered. Next, failure mechanisms of the dikes are considered to evaluate which contribute significantly to the failure probability. The associated reinforcements are discussed to see the effect on the failure probability and the investment costs when implemented.

5.1. Impact of a changed storm surge barrier

In this section, the change in boundary conditions due to a changed barrier is discussed. Only the dataset from the Sea Level Rise Knowledge Program (*'Kennisprogramma Zeespiegelstijging'*) is used in Hydra-NL to obtain consistent data. In section 2.5 different alternatives for the Eastern Scheldt barrier have been discussed. The strategies are:

1. Heightening of the closure water level
2. Changing nothing
3. Improving the closure reliability

Heightening of the closure water level will be included as heightening from 3 m + NAP to 3.25 m + NAP at 1 m SLR. The closure frequency of the barrier will reduce due to the heightened closure water level. Changing nothing on the barrier will result in more frequent closures due to sea level rise. The improvement of the closure reliability of the barrier is included as a factor 10 improvement. That results in a factor 10 smaller failure probability per closure.

5.1.1. Effect of a higher closure water level

The effect of a higher closure water level of the storm surge barrier at the Eastern Scheldt will be discussed here. In this thesis calculations have been made whereby the closure water level of the barrier is changed from 3 m + NAP to 3.25 m + NAP. Figure 5.1 shows the effect of heightening the closure water level of the Eastern Scheldt barrier at 1 m sea level rise. The dataset from the Sea Level Rise Knowledge Program (*'Kennisprogramma Zeespiegelstijging'* [54]) that includes a closure water level change of 25 cm is calculated specifically for 1 m sea level rise. Changing the closure water level reduces the closure frequency of the barrier, preserving the tidal amplitude. For 0 or 0.5 m SLR, the closure frequency is not high enough to require a heightened closure water level. That is why the data for 0 or 0.5 m SLR with a changed closure water level is not available.

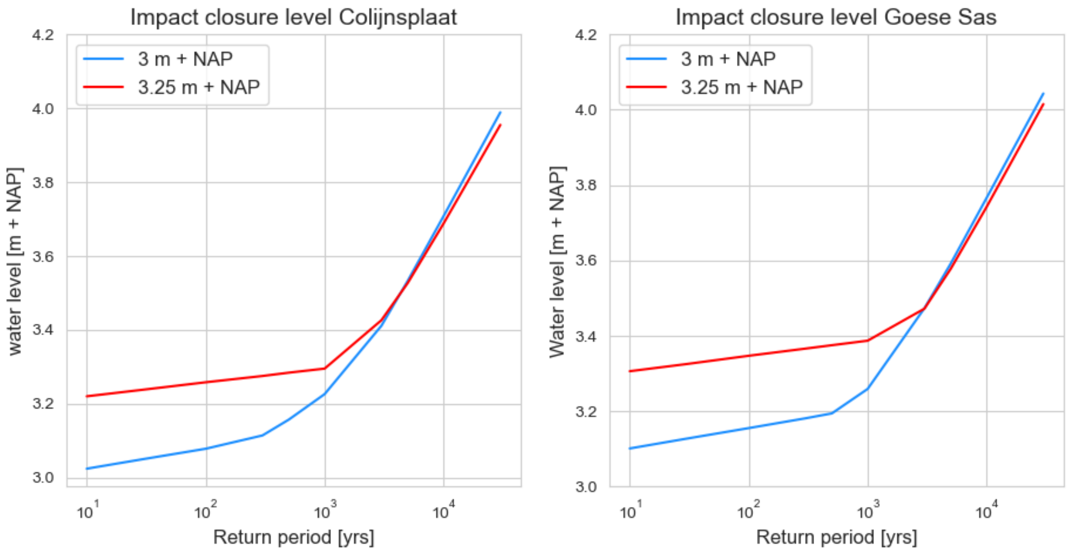

Figure 5.1: Effect of 25 cm higher closure water level at 1 m SLR

Figure 5.1 shows that for low return periods, a higher closure water level results in a higher water level in the Eastern Scheldt(Figure 4.2 shows the locations of Colijnsplaat and Goese Sas within the Eastern Scheldt). This difference is to be expected, since the barrier in the new situation does not close for water levels between 3 m + NAP and 3.25 m + NAP. Thus, the result is a higher water level within the Eastern Scheldt in such a case. The loading on the dikes under normal conditions becomes greater, since a higher closure water level at the barrier results in higher water levels for low return periods. However, damage due to this kind of repetitive loading on the dikes can be monitored more easily than damage due to a single storm. If any damage is detected due to the repetitive loading on the dikes, there is sufficient time to repair the dike before any failure occurs. Therefore, the effect of the higher closure water level on the water level in the lower return periods does not contribute significantly to the total failure probability. For water levels above the 3.25 m + NAP, the barrier closes in both situations and the difference between the two scenarios becomes small. For higher water levels other effects (e.g. failure of the barrier: non-closure, overtopping, overflowing) play a larger role and the effect of a higher closure water level is limited [54]. In Figure 5.2, the wave height is shown for a closure water level of 3 m + NAP and 3.25 m + NAP respectively.

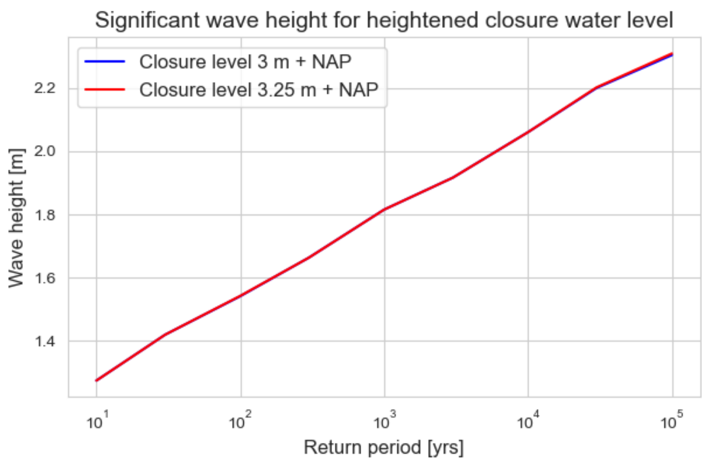

Figure 5.2: Wave height at Colijnsplaat for higher closure water level at 1 m SLR

As shown in the graph, the wave height does not change when a higher closure water level is implemented. The change in closure water level at the storm surge barrier will thus not lead to differences in wave height at the dikes, only water levels will be impacted.

5.1.2. Effect of improved closure reliability

Another option for the Eastern Scheldt storm surge barrier that is included in the case study, is the improvement of the closure reliability. The closure reliability in this context is the inverse of the probability of failure to close the barrier when it is supposed to close. The barrier consists of 62 gates, therefore a distinction is made between the failure of 0, 1, 2, 5, 10, 16, 31, 47 or 62 gates (see Appendix B). The failure probability of the entire barrier can not be simplified to a single number, because the barrier can fail partially. Reducing the closure failure probability by a factor 10 will therefore result in a reduction of all failure scenarios by a factor 10. The effect of this measure is shown in Figure 5.3.

Figure 5.3: Effect of closure reliability improvement on the water levels within the Eastern Scheldt

The effect of an improved closure failure probability is larger for 1 meter sea level rise than for 0 meter sea level rise, which makes sense, since the barrier closes more frequently for 1 meter sea level rise because the water level exceeds 3 m + NAP more frequently. It can be expected that this measure of improved closure reliability of the storm surge barrier will be more effective as sea level rise increases further.

To summarise, the water level in the Eastern Scheldt increases in case of a higher closure water level, but only in the low water level range. The wave height in the Eastern Scheldt is not impacted by the change in closure water level. Improving the closure reliability of the barrier decreases water levels in the Eastern Scheldt. The effect is greatest in case of higher sea level rise, because the barrier is required to close more frequently for such sea water levels. The lower water level does not lead to failure of the dikes at the Eastern Scheldt, since the dikes can resist these forces. The lower water levels do contribute to failure due to repetitive loading, however this can be monitored and repaired and therefore does not increase the failure probability significantly.

5.2. Elimination of IMPLIC stations

Following the decision to make use of the Hydra-NL datasets from the Sea Level Rise Knowledge Program (*'Kennisprogramma Zeespiegelstijging'*) [54], in this thesis only the data for the number of datapoints in the dataset is available. The number of datapoints is equal to the number of IMPLIC stations, for which the hydraulic conditions were calculated. These can be seen in Figure 5.4.

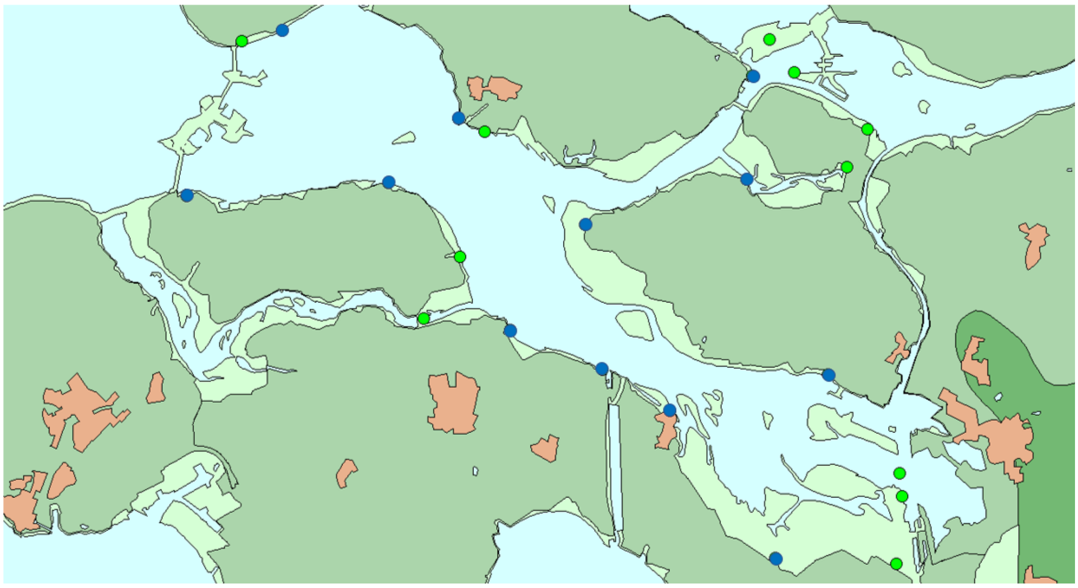

Figure 5.4: IMPLIC stations included (blue dots)

However, since the area of interest is relatively small, the hydraulic loads do not differ significantly and therefore using only the IMPLIC stations will not lead to an oversimplification. Figure 5.5 shows an example for dike segment 26-2. The water level differs minimally for the shorter return periods. For the longer return periods, the difference between data points becomes even smaller. Since the highest water levels are governing for the failure probability, this is the most important part of the spectrum. The differences in between the data points for the highest water levels are smallest. Since the differences along the dike segment are small in that part, the assumption of only using the IMPLIC stations is assumed sufficiently accurate. The results are likely not to differ if all dike sections are considered separately.

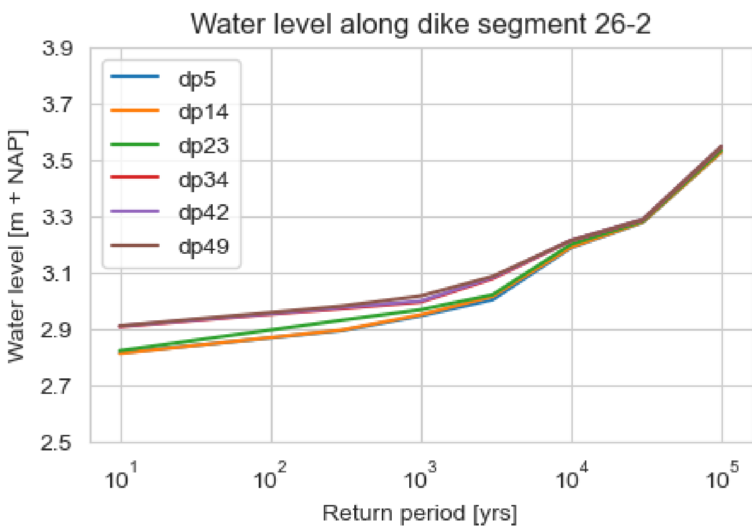

Figure 5.5: Water level along dike segment 26-2

Figure 5.4 shows the IMPLIC stations available in the dataset. The number of IMPLIC stations in the dataset is too large to optimise within the available time for this thesis. Since the optimisation is not meant to provide accurate numbers but rather a conceptual model, the number of IMPLIC stations included in the optimisation has been reduced. Specific stations can be eliminated based upon the proximity to another station. The hydraulic boundary conditions do not change significantly over short distances along the dike segments. If two stations are relatively close to each other, they will have approximately the same boundary conditions. Figure 5.6 shows three examples of this. The water level at these locations behaves very similarly and thus the number of IMPLIC stations included in the optimisation can be reduced. Additionally, the wave height at these locations is in the same order of magnitude.

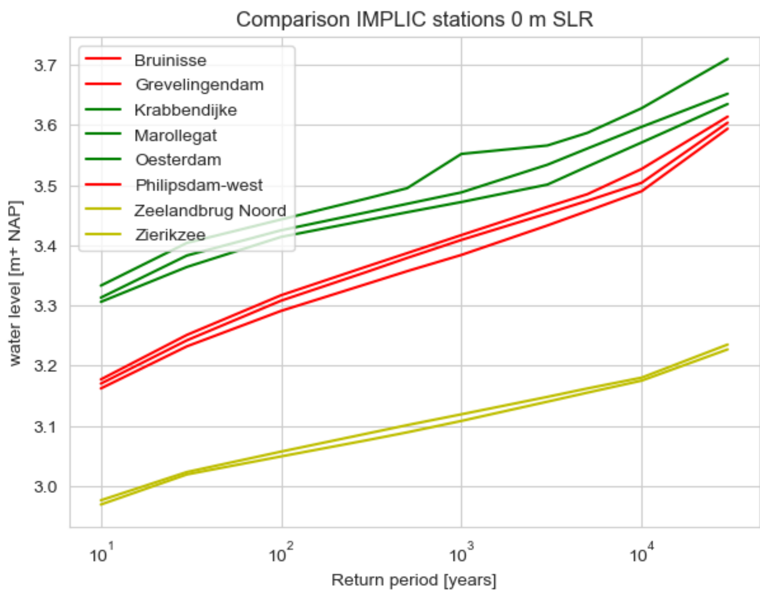

Figure 5.6: Combined IMPLIC stations

The second reason for eliminating an IMPLIC station is its location. If the station is located on an compartment dam, the area behind the station consists of more water (without damage value) behind which there are primary flood defences, but these are considered out of scope for this thesis. The optimisation is based on resulting flood risk and investments costs. If there is no value in the water behind the station, there is no immediate flood risk. The change in hydraulic conditions of the water behind the compartment dam is disregarded in this thesis. The blue dots in Figure 5.4 indicate the stations whose hydraulic boundary conditions are used in the optimisation. The green dots in Figure 5.4 indicate the stations that are excluded.

Figure 5.7 shows the dike segments assigned to each IMPLIC station included in the optimisation.

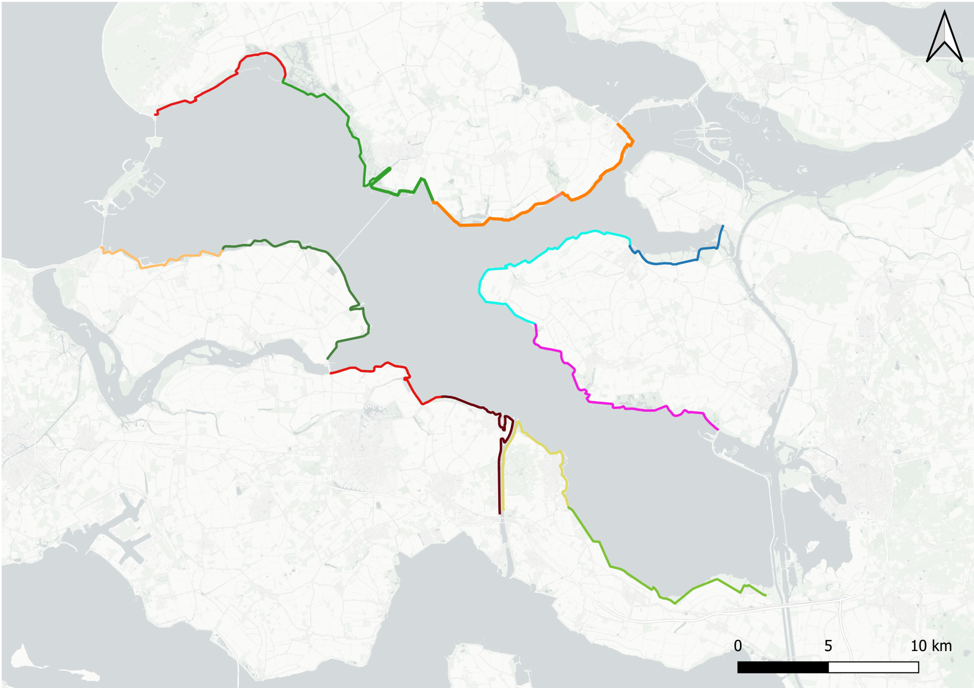

Figure 5.7: Map of dike segments assigned to the IMPLIC stations

5.3. Flood risk calculation

In order to calculate the flood risk, the expected damage in case of a flood needs to be determined. The values used for the case study have been derived from Slootjes and Wagenaar [45]. The expected damage includes fatalities as well as economic damage due to a flood. The expected damage used in the optimisation is the worst-case scenario, in which the dike breaches at the worst possible location. The IMPLIC stations that are used in the optimisation are assigned a compatible dike length and associated expected damage.

5.4. Failure mechanisms dikes

As mentioned in subsection 2.5.6, the dominant failure modes for dikes are overtopping, piping, macro-instability and slope erosion. The failure mechanisms are assumed to be independent from each other. It is probable that overtopping and revetment erosion are not completely independent, due to their dependency on the same hydraulic loads. Assuming that the three failure mechanisms are independent is a conservative assumption. However, the difference between the real value and the assumed failure probability will not be significant due to the low failure probability.

As can be seen in Appendix A, the dikes along the Eastern Scheldt are mostly vulnerable to macro stability problems. Overtopping is currently well below signal failure probability, since the dikes were built before the barrier was constructed. The failure probability due to piping does not contribute significantly to the failure probability of the flood defences. The failure mechanisms are discussed in the following sections, along with the formulas used and the possible reinforcements.

The following subsections will discuss the dominant failure mechanisms for dikes and consider whether they are relevant for the case study of the Eastern Scheldt. Additionally, the possible reinforcements for the failure mechanisms are discussed, along with the cost functions.

5.4.1. Overtopping

The first failure mechanism considered for the case study is overtopping. Hydra-NL is able to calculate the HBN (in Dutch: Hydraulisch Belasting Niveau, English: hydraulic load level) for different scenarios. HBN indicates for a certain dike height, the probability that the maximum overtopping discharge is exceeded. For example, for a dike of 6 m + NAP, the frequency that 1 l/s/m is exceeded for 1 m SLR. The HBN does not change significantly along a dike segment, since the hydraulic boundary conditions do not change significantly over a hundred meters. If the HBN does increase along a dike segment, the dike height increased as well. The HBN is therefore chosen to be coupled to a change in dike height relative to the initial dike height, not the height relative to NAP. The HBN along the dike segment does not change when this approach is taken. The assumption is made that the hydraulic boundary conditions taken at an IMPLIC station on the dike segment are representative for the entire dike segment. If the optimisation is done for 1 meter sea level rise, the HBN levels associated with that are calculated. The 1 meter sea level rise is assumed to occur at the end of the considered time period, the HBN levels in between will be interpolated between 0 m sea level rise and 1 meter sea level rise.

The reinforcement taken into consideration for overtopping is heightening the dike. The height increases are calculated with increments of 0.5 meters from 0 until 1.5 meter increase. Smaller increments are not taken into consideration due to construction limits. In Figure 5.8, a schematisation of the reinforcement can be seen. The assumption is that the slope of the dike remains the same and the dike expands inwards.

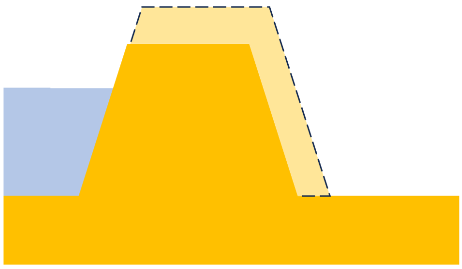

Figure 5.8: Dike heightening reinforcement schematisation

The costs of heightening a dike depend mainly on the volume of soil used. The costs are calculated using the following formula of Klerk et al. [27]:

$$C = C_{initial} + C_{soil} * V_{soil} + C_{building} * B \tag{5.1}$$

In which:

- $C_{initial}$: the starting costs of construction
- C_{soil}: unit costs of soil per m^3
- V_{soil}: volume of soil that needs to be added
- $C_{building}$: the cost of removing a building that has to be removed
- B: number of building that have to be removed

The unit costs originate from KOSWAT [19]. The costs of soil-based reinforcements on the dike come down to €58.76 per m^3 and the initial costs are €252.44 per m.

5.4.2. Piping

The second failure mechanism considered in the case study is piping. In Appendix A, it can be seen that piping does not play a role in most of the assessments for the dikes, only stretches of 31-2 were valued as being III_t, which means that only those stretches of dike where assessed to be at risk for piping failure. According to Joost Pol [35], piping is a highly time-dependent process. The loading time along the Eastern Scheldt is expected to be short (in the order of 4 hours). Including the time dimension in the calculations decreases the risk of piping compared to the instantaneous scenario. Based on the fact that for the instantaneous calculation piping is indicated as unproblematic, the contribution of piping to the total failure probability is negligible. Therefore piping will not be taken into account as a failure mechanism for the Eastern Scheldt case study in this thesis.

5.4.3. Macro-stability

The third failure mechanism considered is macro-stability. Macro-stability is considered one of the dominating failure mechanisms of the dikes at the Eastern Scheldt. A dependency of the failure probability due to macro-stability on the water level was researched, see an example for 6 locations in Figure 5.9.

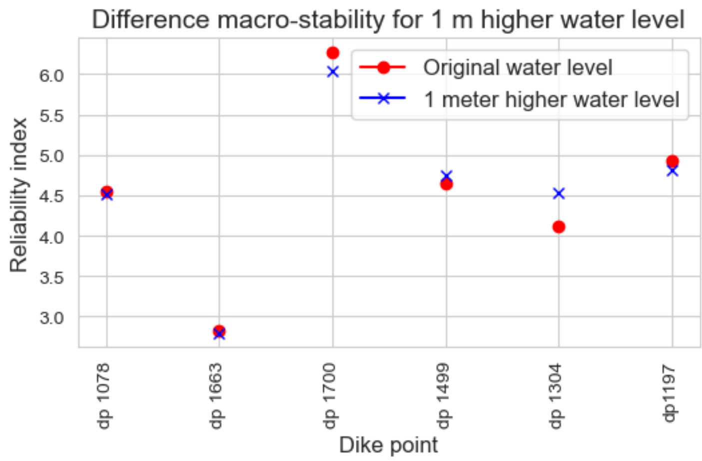

Figure 5.9: Change in reliability index macro-stability for 1 meter higher water level

The reliability index is not influenced significantly when calculated for 1 meter of water level rise. Due to the dampening effect of the storm surge barrier, 1 meter of sea level rise will lead to less than 1 meter of water level rise in the Eastern Scheldt. The results from this analysis are therefore even more extreme than if 1 meter of sea level rise were to occur. This analysis indicates that the soil properties are the dominating factor when determining the safety factor. When calculating the safety factor, the influence of uncertainties is larger than the influence of 1 meter water level rise. Therefore it is decided not to include a changing safety factor for different sea level rises, but to assume the safety factor to remain constant.

To be able to calculate the failure probability for macro-instability, a calibrated formula was used, see Equation 5.2.

$$SF = 0.15 * \beta + 0.41 \tag{5.2}$$

In which:

- SF: safety factor
- β: reliability index

The formula provides a relation between the safety factor for macro-stability and the reliability index. The calibrated formula is calculated specifically for the semi-probabilistic safety assessment for macro-stability as was performed along the Eastern Scheldt [25].

The initial situation for macro-stability has been assessed in the WBI assessment [23] and the safety factor for each dike segment can be found in the appendices of this thesis. The WBI is the source used for this project. The safety factor of macro-stability is not constant along a dike segment. fig:MacrostabSF shows the change of the safety factor over dike segment 28-1. The safety factor is not constant and the variation is deemed to be too large to neglect.

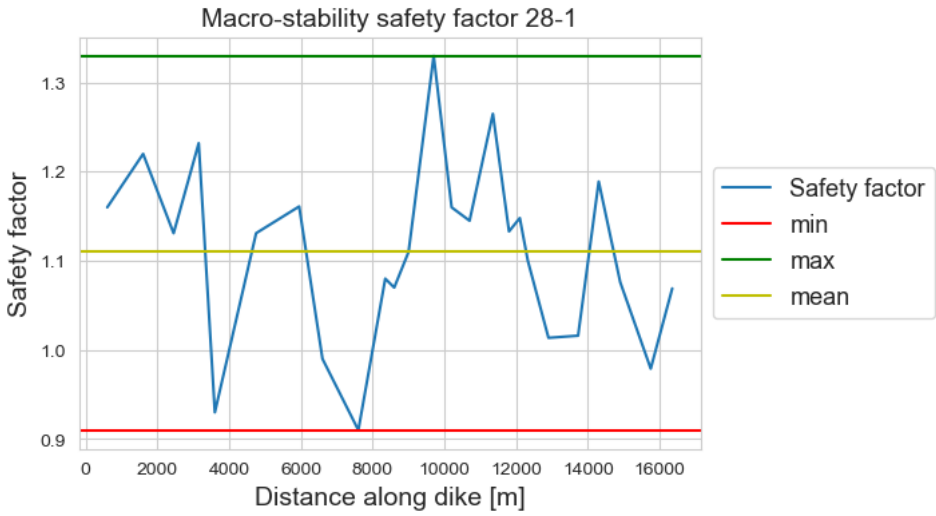

Figure 5.10: Macro-stability safety factor dike segment 28-1

The variation along a dike segment is taken into account in the optimisation by including the safety factor of every single dike section. The costs of reinforcing that dike section are set of against the resulting flood risk to be able to calculate the best option. This cost-benefit ratio is the parameter that should be lowest for the optimal reinforcement. The assumption here is that if a dike section fails for macro-stability, the area behind it floods. The calibrated formula is used to convert the safety factor to a reliability index, which in turn can be translated into a failure probability for macro-instability. Considering every dike section individually instead of simplifying by taking an entire dike segment as the basis, creates the possibility to reinforce the weak spots in a dike segment while leaving the stronger parts untouched. The investment costs should therefore be lower using this method than when the safety factor is simplified into a single digit for the entire dike segment. The disadvantage of this method is that costs associated with starting a reinforcement project could deviate from the costs calculated. These costs are included in the unit costs, but are underestimated if the resulting reinforcement is very small and the overhead costs are thus larger than included in the unit costs. Including different safety factors enables making a distinction between areas without any buildings close to the dike and areas with buildings. The costs associated with resolving the buildings conflict are one of the deciding factors when choosing a macro-stability reinforcement.

The reinforcement options associated with macro-stability are:

- Berm
- Diaphragm wall
- Stability screen

Diaphragm wall

A measure that can be taken to reduce the flood probability due to macro-instability is constructing a diaphragm wall. The design of a diaphragm wall considered in this thesis reduces the failure probability due to macro-stability to 10^{-8}. However, a diaphragm wall is very costly to construct. A schematisation of a diaphragm wall can be seen in Figure 5.11.

Figure 5.11: Schematisation diaphragm wall

The diaphragm wall is assumed to be 15 meters deep and 0.8 meter thick. The costs associated with these measurements are €24,170 per meter of diaphragm wall. The choice could be made to make the diaphragm wall even thicker, but the construction costs will only increase, while in most the cases the failure probability of 10^{-8} will be sufficient, which is obtained with a diaphragm wall of 15 meters deep and 0.8 meter thick.

Stability screen

Another option to reinforce the dike for macro-stability is constructing a stability screen. A schematisation of this can be seen in Figure 5.12.

Figure 5.12: Schematisation stability screen

The stability screen design considered in this thesis increases the safety factor with 0.2. To calculate the costs of a stability screen, the thickness of the cover layer must be known. The cost can then be calculated as follows:

$$C = C_{ss} * (d_{cover} + 2) * L \quad (5.3)$$

In which:

- C_{ss}: the unit cost per square meter of sheet pile
- L: the length of the dike segment
- d_{cover}: the thickness of the cover layer

The costs of a stability screen are €580 per m^2 according to KOSWAT[19].

Berm

The last reinforcement option considered for macro-stability is constructing a berm. In situations where a berm is already present, the berm can be extended. In Figure 5.13, a schematisation of a berm can be seen.

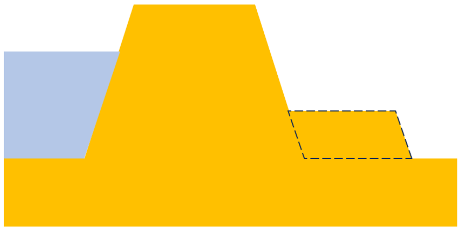

Figure 5.13: Schematisation berm

When constructing a berm, buildings need to be demolished, while that is not necessary when diaphragm walls or stability screens are constructed. In case there are many buildings present near the dike, the best option could be to implement a diaphragm wall or a stability screen to avoid having to resolve the building conflict. Keep in mind that resolving a building conflict is a sensitive subject and is in reality not as straightforward as used here in the optimisation. In case a berm is applied, the improvement of the safety factor depends on the cover layer and length of the berm:

$$SF = SF_0 + f * L_{berm} \tag{5.4}$$

In which:

- L_{berm}: berm length
- SF_0: Initial safety factor
- f: factor indicating rate of change (if the cover layer > 7 m: f = 0.008, otherwise f = 0.02)

This equation stems from Klerk et al. [27]. The construction of a berm is associated with initial construction costs, costs associated with the soil needed for the berm and costs for the demolition of buildings. These costs are calculated as follows:

$$C = C_{initial} + C_{soil} * V_{soil} + C_{building} * B \tag{5.5}$$

In which:

- $C_{initial}$: costs associated with starting construction
- C_{soil}: unit costs of soil per cubic meter
- V_{soil}: volume of soil to be added for the berm
- $C_{building}$: demolition costs for a building
- B: number of buildings to be demolished

To calculate the optimal berm length, the berm length is calculated with increments of 5 meters from 5 meters berm length until 30 meters. The berm height in this thesis is taken as 2 meters.

In conclusion, there are three possible reinforcements for macro-stability: a berm, a stability screen or a diaphragm wall. The berm is the cheapest option and can easily be extended to increase its effect. However, the costs for the berm increase when it interferes with buildings. Both the diaphragm wall and the stability screen do not interfere with buildings, but these options are more expensive to construct. The diaphragm wall is the strongest reinforcement, but also the most expensive. Depending on the presence of buildings and the initial safety factor, the optimal reinforcement for macro-stability can be determined.

5.4.4. Revetment erosion

The fourth failure mechanism that is considered in this thesis is revetment erosion. The failure probability due to revetment erosion depends on water level and wave height. Revetment can fail in two ways:

- The revetment is not strong enough
- The revetment is not extended far enough up the dike

The first mechanism of failure, not having strong enough revetment, indicates that there is revetment present at the point of the dominant load. It means that the revetment is not thick or strong enough to withstand the wave force. This can happen if the blocks used to make the revetment are not large enough. The second way of failure is the revetment not extending far enough up the dike slope. Revetment is always constructed on the part of the dike slope that is under loads larger than what grass could resist. In case the revetment is not extended far enough, the loads upon the grass exceed its strength and the grass fails. Since at the Eastern Scheldt the wave height is not expected to increase due to sea level rise (see Figure 4.3), it is expected that only the second mechanism will be relevant for the Eastern Scheldt case. The wave load on the revetment will not increase, only the location upon the dike where this load acts will be higher. The assumption made here is that the current revetment thickness at the dikes at the Eastern Scheldt is large enough to withstand the wave force. Reinforcing the dikes for revetment erosion will therefore consist of extending the existing revetment further up the dike.

Calculation method revetment

In order to optimise the revetment, the failure probability must be calculated. As discussed above, the revetment failure probability is only taken as the probability that the transition between revetment and the grass is not high enough. To calculate the failure probability of the revetment for a certain transition height, first the loads upon the revetment have to be calculated. These loads are obtained using Hydra-NL as well, making use of the revetment calculation mode. The loads are obtained for wave run-up and wave impact. Since revetment does not fail instantaneously, but gradually fails over time, the load duration matters. The hydraulic loads on the dike revetment during a storm have to be reconstructed to accurately calculate the failure probability. The duration of certain waves and water levels depends on the tide, the storm surge barrier and the storm surge levels. The WBI assessment has indicated a generalised water level time line during a storm in case of closure of the Eastern Scheldt barrier, see Schematisation Manual Grass Revetment (*Schematiseringshandleiding Grasbekleding* [44]). According to the Base Report of WBI 2017 (*Basisrapport WBI 2017* [4]), the following water level timeline was used for the assessment:

Table 5.1: Water level timeline in case of closure [4]

Time [hours]	Water Level [m]
0-10	NAP + 1 m
10-30	MHW - 2 m
30-55	MHW - 1 m
55-60	MHW + 0.2 m

This approach combines the water level occurring at a manual closure and an emergency closure of the storm surge barrier at the Eastern Scheldt, always choosing the highest load.
Using this water level timeline, the Q-variant data could be constructed. The Q-variant data is constructed for wave run-up as well as wave impact. The Q-variant data consists of the water level timeline as in Table 5.1 and the associated wave height and wave period for that water level and return period. Subsequently, the program DiKernel was used, which is able to calculate a damage number at a certain location, for a certain location (the transition level) and a certain return period. The damage number is defined as the number of displaced units within a strip on the dike with a width of D_{n50}. across the slope DiKernel takes into account the cumulative loading on the dike in the given storm. The damage number relates to either wave impact or wave run-up, so both of these failure mechanisms need to be calculated separately. The failure mechanism with the highest failure probability is ultimately taken as the dominant failure mechanism. The safety factor can be calculated from the damage number using the following formula:

$$SF = \frac{1}{Damage\ number} \tag{5.6}$$

The revetment is assumed to fail at the return period with a safety factor equal to one. The return period associated with this safety factor can be found through interpolating between the return periods for which the safety factor was calculated.

5.5. Effect of reinforcements

This section will discuss the impact of possible reinforcements at the second line of defence on the failure probability. The efficiency of a reinforcement to reduce the flood risk determines at what cost a reinforcement is cost-effective. If a reinforcement reduces the flood risk enormously, the investment costs can be higher also and the reinforcement can still be cost-effective.

5.5.1. Reinforcements Macro-stability

In subsection 5.4.3, the options for reinforcements regarding macro-stability were discussed; constructing a berm, a diaphragm wall or a stability screen. There are multiple dimensions possible for the berm: the thickness and the berm length can be altered. In the case study the berm length is calculated from 0 to 30 meters with intervals of 5 meters. The increase of the safety factor is calculated according to Equation 5.4. The effect of different berm lengths is shown in Figure 5.14.

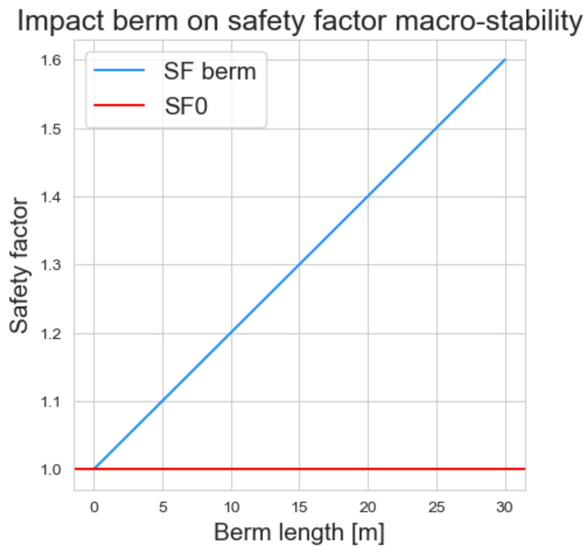

Figure 5.14: Safety factor due to construction berm

As mentioned in subsection 5.4.3, the failure probability when a diaphragm wall is constructed reduces to 10^{-8}. A stability screen increases the initial safety factor with 0.2. If the initial safety factor is very low, it is better to construct a diaphragm wall instead of a stability, despite its high costs, due to its independence from the initial safety factor. However, if there is no conflict with buildings, a berm could still be a cheaper option.

5.5.2. Reinforcements overtopping

The reinforcement option taken into account for overtopping is heightening the dike. The effect of heightening the dike is shown in Figure 5.15. The effectiveness of the reinforcement differs per location, due to the difference in hydraulic conditions and the initial dike height.

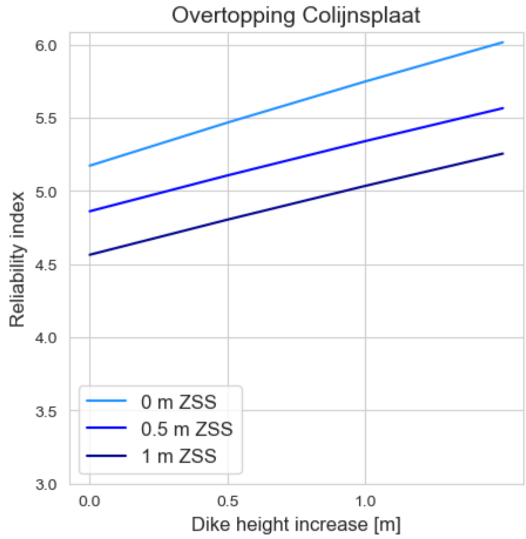

Figure 5.15: Heightening the dike against overtopping failure

5.5.3. Reinforcements revetment failure

The reinforcement that can be constructed to reduce the failure probability due to revetment failure is to heighten the revetment on the dike. Figure 5.16 shows an example of the difference in failure probability when the revetment is heightened with 0.5 meter on the dike.

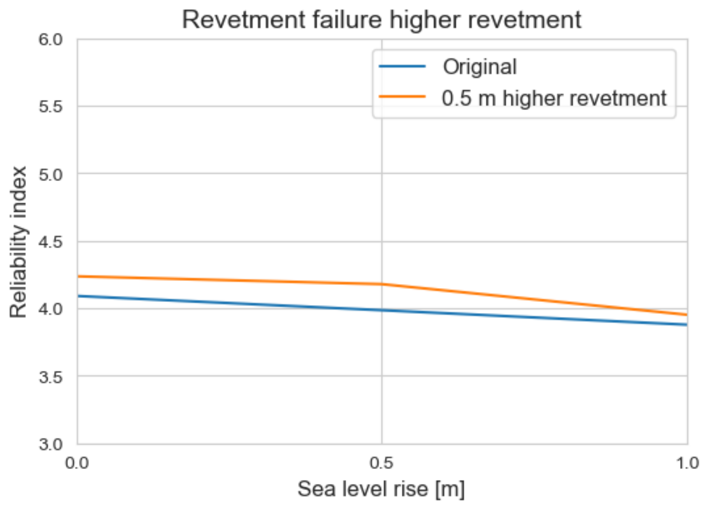

Figure 5.16: Revetment reinforcement impact at Bergse Diepsluis

Extending the revetment does have impact on the failure probability, however this effect is limited. The costs will determine whether the reinforcement will be a sound decision.

5.6. Computed failure probabilities for different barrier options

In this subsection the effect of the options for the storm surge barrier on the failure probabilities is discussed. The options included in this thesis are:

- Improving the closure reliability by a factor 10
- Heightening the closure water level

The effect of these options will be discussed per failure mechanism. The failure probability due to macro-stability is not included as it is assumed to remain constant.

5.6.1. Overtopping

This subsection discusses the impact of different barrier options on overtopping. Figure 5.17 shows the difference in reliability index between the barrier options for overtopping for 0 meter sea level rise.

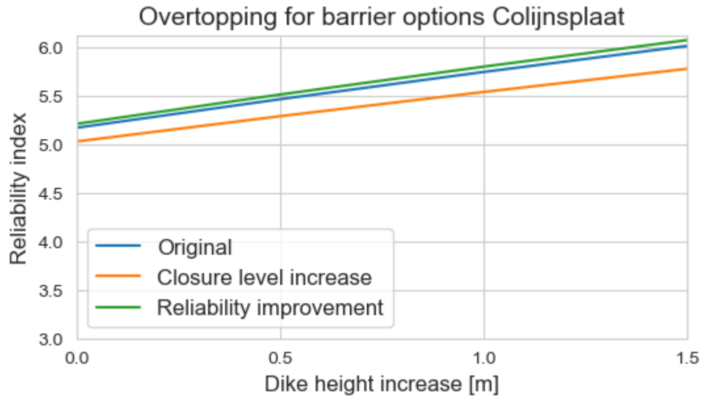

Figure 5.17: Overtopping reliability for different barrier options at Colijnsplaat for 0 m SLR

The figures shows that the reliability index decreases for overtopping if the closure water level is heightened with 25 centimeters. The reliability index increases if the closure failure reliability is improved by a factor 10. The difference is not large compared to the original situation. Additionally, the reliability index for overtopping in the Eastern Scheldt is already high in the current situation.

5.6.2. Revetment failure

Figure 5.18 shows the development of the failure probability due to revetment failure.

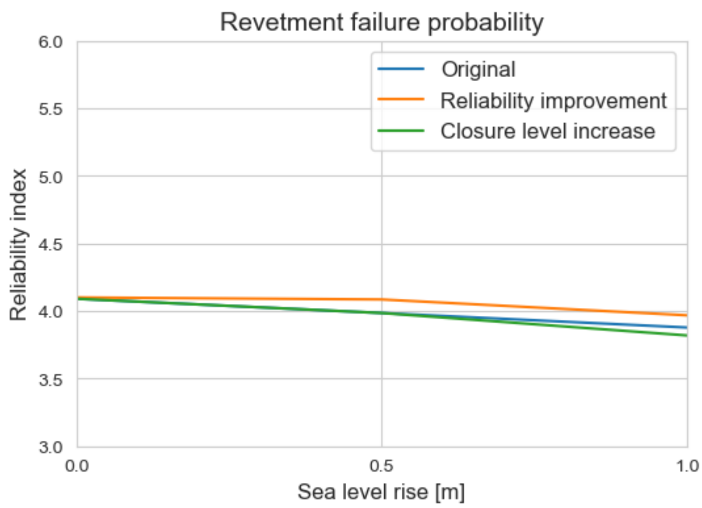

Figure 5.18: Revetment failure for different barrier options at Bergse Diepsluis

Improving the closure reliability reduces the failure probability, however the effect is limited. Heightening the closure water level of the storm surge barrier increases the failure probability compared to the current situation. This effect is to be expected because the water levels increase when a higher closure water level is implemented (see Figure 5.1). However, the effect is also limited in this case.

6 Results

The results of the optimisation will be discussed in this chapter. Firstly, the results from the optimisation in the case study will be discussed in section 6.1. Next, the taken approach to optimising any system with multiple lines of defence will be illustrated with a second case study in section 6.2.

6.1. Results from the optimisation of the system

In chapter 5 the impact on the hydraulic boundary conditions due to sea level rise or different barrier options is discussed. The optimisation performed over the flood defence system balances the flood risk against the costs of reinforcements. The results for the case study in the Eastern Scheldt are included in this section. Firstly, the impact of sea level rise is considered, after which the impact of changes to the storm surge barrier is considered.

6.1.1. Resulting flood risk from sea level rise

In Table 6.1, the flood risk resulting from the optimisation is shown. In this analysis, the Eastern Scheldt barrier is assumed to remain unchanged and the reinforcements at the dikes along the Eastern Scheldt are optimised.

Table 6.1: Flood risk resulting from optimisation

SLR at t = 100 [m]	Flood risk incl. reinforcements [million €]	Investment costs [million €]
0	64.90	108.76
0.5	80.47	108.76
1.0	105.78	108.76

Since macro-stability was assumed constant for changing water levels, the difference in flood risk when sea level rise occurs, originates from revetment and overtopping flood risk. The results are in line with the expectations, since higher sea level rise causes higher water levels in the Eastern Scheldt and therefore higher flood risk. The investment costs remain constant for sea level rise, which indicates that there are no further investments needed when sea level rise occurs. This will be adressed further in this chapter.

6.1.2. Macro-instability reinforcements

Large parts of the Eastern Scheldt need to be reinforced for macro-instability according to the WBI assessment (see Appendix A). Macro-instability depends on water levels, but the dependency is not as strong as for overtopping and revetment erosion. The macro-instability safety factors as they were calculated in the WBI assessment are therefore taken as a starting point to calculate the optimal reinforcements. The macro-stability reinforcements are determined per length of a dike for which the data is available for. An example of results is shown in Table 6.2.

Table 6.2: Reinforcements at Stavenisse for macro-stability

SF	Length[m]	Reinforcement	Berm length [m]
0.86	1100	Stability screen	-
0.85	500	Berm	15
1.13	1500	None	-
1.14	400	None	-
1.05	1850	None	-
1.03	650	Stability screen	-
1.09	650	None	-
1.07	1750	None	-
1.11	800	None	-
1.13	900	None	-
1.14	700	None	-
1.19	700	None	-
0.98	1500	Berm	5
1.05	1700	None	-

These reinforcements are determined for smaller dike lengths than the other failure mechanisms. The reinforcement projects that are marked to be optimal are therefore smaller in costs and length. An example is shown in Figure 6.1.

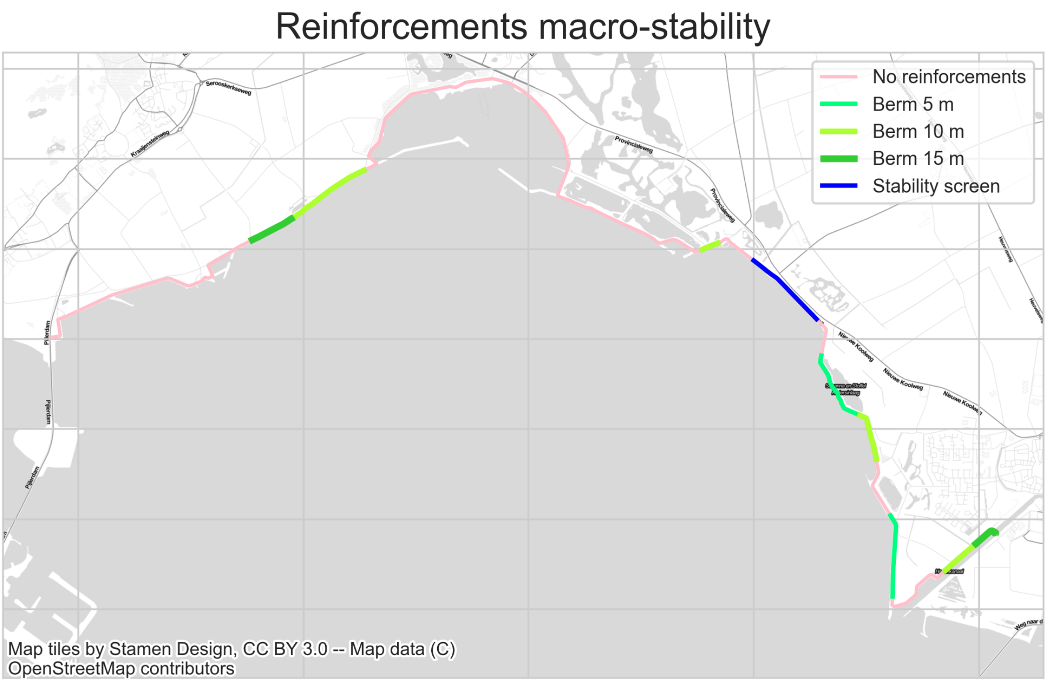

Figure 6.1: Macro-stability reinforcements for dike segment 26-2

6.1.3. Reinforcements for overtopping and revetment

Flood risk is known to increase when sea level rise occurs. In this case study however, the increased flood risk is not great enough to require reinforcements for either overtopping or revetment failure.

Table 6.3: Reinforcements dikes for overtopping and revetment erosion

	Height increase	Revetment increase
Bergse Diepsluis	0	0
Bruinisse	0	0
Burghsluis	0	0
Colijnsplaat	0	0
Goese Sas	0	0
Krabbendijke	0	0
Roompot Binnen	0	0
Sint Annaland	0	0
Stavenisse	0	0
Wemeldinge	0	0
Yerseke	0	0
Zierikzee	0	0

The conclusion of these numbers is that no investment costs for overtopping or revetment are needed. Since it would have made sense to heighten the dikes for sea level rise, the following table shows the total costs of a dike reinforcement against overtopping:

Table 6.4: Costs for dike heightening at Colijnsplaat for 1 m SLR

Dike height	Flood risk [€]	Investment cost [million €]	Total costs [million €]
Initial	29,576	0	0.029
+0.5	7,850	19.52	19.53
+1.0	2,103	36.49	36.50
+1.5 m	570	54.97	54.97

An example of costs for the location Colijnsplaat is shown in Figure 6.2. The total costs consist of flood risk and investment costs. The flood risk is significantly lower than the investment costs when the dike is significantly heightened.

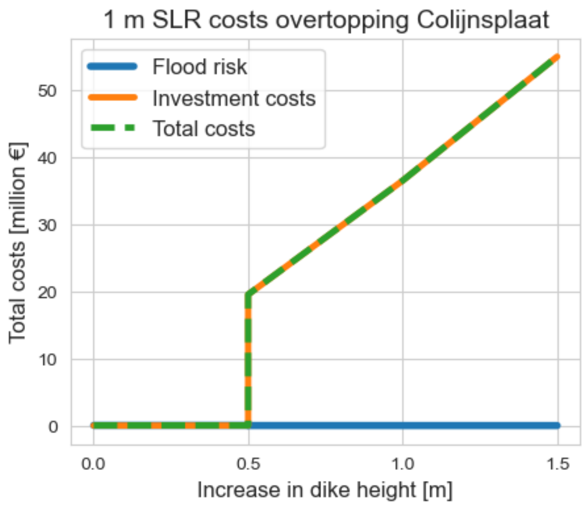

Figure 6.2: Costs overtopping at Colijnsplaat

Table 6.4 and Figure 6.2 clearly show that the total costs (flood risk and investment costs) are lowest when no reinforcements are carried out. Additionally, the flood risk is only a fraction of the total costs, investments only increase the total costs and are therefore not cost effective.

Figure 6.3 shows the total costs for revetment failure at 1 m sea level rise and an unchanged storm surge barrier. The costs consist of flood risk and investment costs when the revetment is extended.

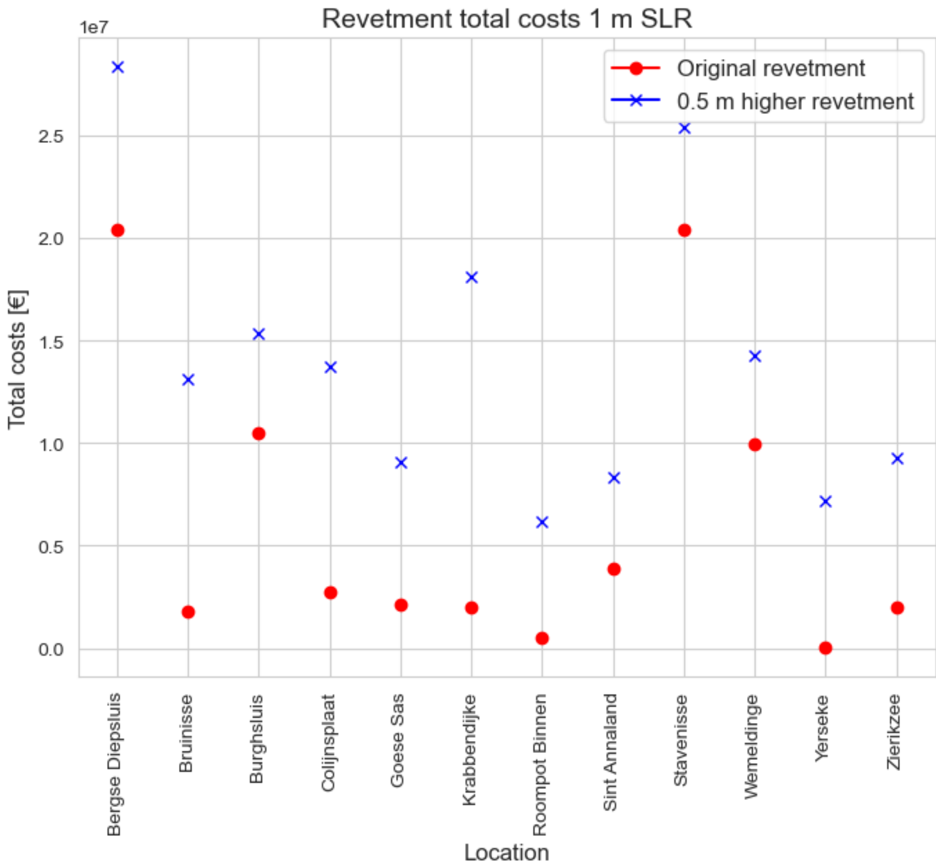

Figure 6.3: Difference costs revetment reinforcement at 1 m SLR

The total costs are lower for all locations when no reinforcements are implemented. This conclusion supports the results from the optimisation that no reinforcements are required for revetment.

The need for reinforcements does not change if sea level rise occurs. This can be attributed to three things:

- The dikes were built at 'Delta hoogte'
- Revetment has been reinforced
- The barrier reduces the increase in loads

Firstly, the dikes were built at 'Delta hoogte', meaning that they were built to withstand the hydraulic loads that occurred before the storm surge barrier was built. The dikes along the Eastern Scheldt are therefore built for a situation without the storm surge barrier. Secondly, Projectbureau Zeeweringen has improved the revetment along the dikes of the Eastern Scheldt between 1997 and 2015 expertise [18]. The revetment is designed to withstand a 'superstorm' (storm that occurs once every 40,000 years) and therefore it makes sense that the revetment does not need to be extended further. Thirdly, the storm surge barrier reduces the increase in loads on the dikes by sea level rise. One meter of sea level rise does not translate to one meter of water level rise within the Eastern Scheldt. The overall result is that no investments in height or revetment at the dikes of the Eastern Scheldt are required.

6.1.4. Resulting costs for storm surge barrier options

The resulting costs for the storm surge barrier options are visualised in Figure 6.4. The investment costs do not change if a different storm surge barrier option is chosen, as the reinforcements do not change either. However, the investment costs do not include the costs of changing the storm surge barrier itself. These costs cannot be estimated accurately within the time frame of this project. The flood risk increases if the closure water level of the storm surge barrier is heightened with 25 cm, due to the increased loads at the dikes. If the closure reliability of the barrier is improved by a factor 10, the flood risk reduces with 16.54 million euros. Improving the closure reliability of the barrier should therefore cost less than 20.82 million euros will this option be cost effective. In case the improvement costs are higher than 20.82 million euros, it is not optimal to choose in this strategy.

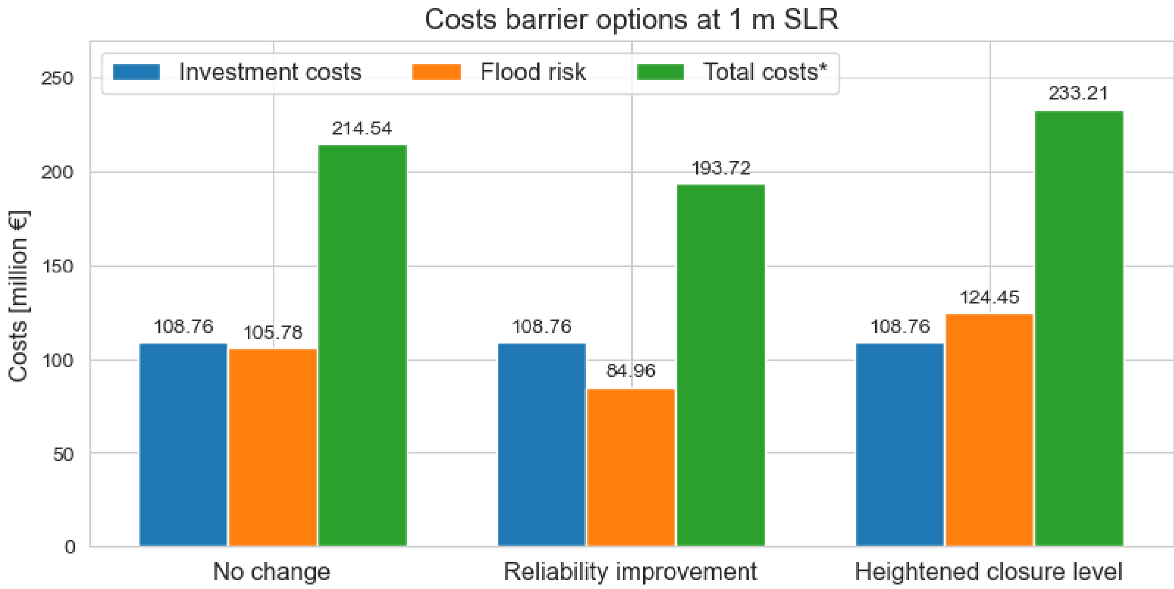

Figure 6.4: Costs barrier options at 1 m SLR

*The costs of changing the barrier are not included

Heightening the closure water level results in an increase of flood risk of 18.67 million euros. The advantage of this is the decrease in closure frequency. Decreasing the closure frequency preserves the tidal motion within the Eastern Scheldt, while it would decrease in case the same closure water level would be maintained. The increase in flood risk is a risk that would have to be accepted in order to preserve the tidal motion.

6.1.5. Summary results case study

The results from the case study at the Eastern Scheldt show that macro-stability reinforcements are needed in each scenario, since the reinforcements are demonstrated to be independent from water level changes (up to 1 meter). Additionally, the results show that no reinforcements are required for overtopping or revetment, even when sea level rise occurs. Changes to the barrier do increase or decrease the flood risk, but the investment costs remain the same.

6.2. Results from height dependent system

The optimal reinforcements in the Eastern Scheldt case study do not depend on sea level rise. Therefore a secondary exploration was performed to investigate whether the optimisation method is sensitive to changes in hydraulic loads. The case study area was used as a template, but all dikes were assumed to have 1.5 meter lower crests to ensure more reinforcements to be optimal instead of applying none. Additionally, the dike heightening increments were reduced to 0.1 m, ranging from 0 to 1.5 m higher dikes. As a result, the change in loads can be smaller to see changes in optimal dike height.

In Table 6.5 the optimal solution for overtopping is depicted. The optimal solution differs in some locations when sea level rise occurs. Only for three locations at the dikes at the Eastern Scheldt there are changes in the optimal reinforcement: Sint Annaland, Stavenisse and Wemeldinge. However, the changes prove that the method is sensitive to load changes and does respond to higher loads with more reinforcements.

Table 6.5: Dike height increase for 1.5 m lower dikes for SLR

Location	Height increase [m]		
	0 m SLR	0.5 m SLR	1.0 m SLR
Bergse Diepsluis	0.5	0.5	0.5
Bruinisse	0	0	0
Burghsluis	0	0	0
Colijnsplaat	0	0	0
Goese Sas	1.0	1.0	1.0
Krabbendijke	0.3	0.3	0.3
Roompot Binnen	0	0	0
Sint Annaland	0.8	0.8	0.9
Stavenisse	0.6	0.6	0.7
Wemeldinge	0.5	0.6	0.7
Yerseke	0	0	0
Zierikzee	0	0	0

Figure 6.5 shows the costs for dike heightening at Wemeldinge. The increase in optimal dike height is shown, which is the result of the moving of the minimum in total costs. However the optimum is quite flat, indicating that the difference in total costs is not large between 0.6 m and 0.9 m dike heightening.

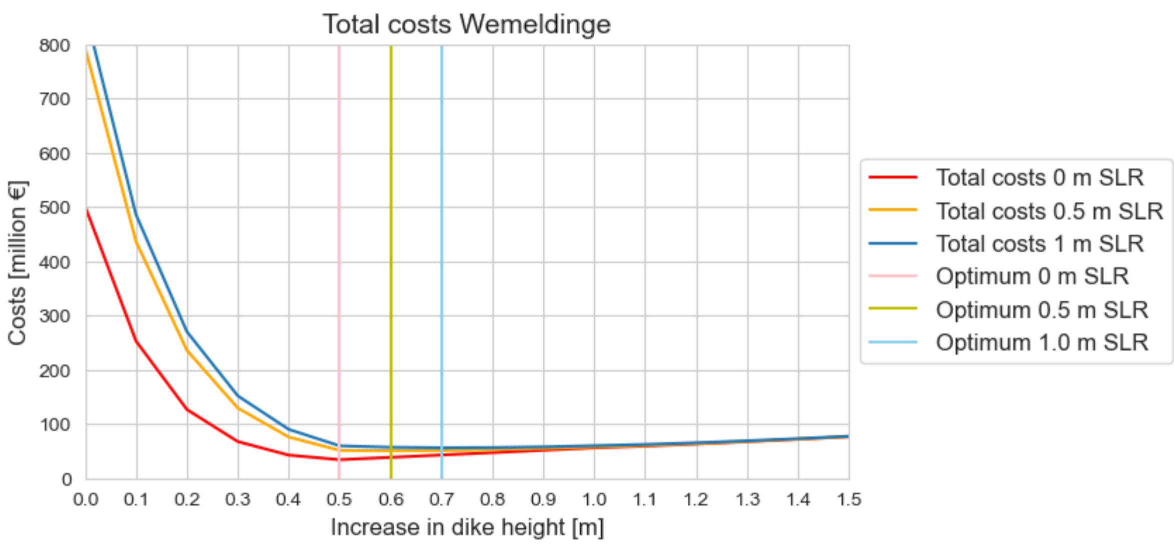

Figure 6.5: Costs Wemeldinge for sea level rise

Furthermore, the influence of changes to the storm surge barrier is examined. Table 6.6 shows optimal reinforcements given different barrier strategies. The sea level rise scenario displayed here is 1.0 m SLR, due to the larger differences in loading.

Table 6.6: Dike height increase for 1.5 m lower dikes for OSK options with 1 m SLR

Location	Height increase [m]		
	Original	Improved closure reliability	Heightened closure water level
Bergse Diepsluis	0.5	0.5	0.6
Bruinisse	0	0	0
Burghsluis	0	0	0
Colijnsplaat	0	0	0
Goese Sas	1.0	1.0	1.0
Krabbendijke	0.3	0.3	0.4
Roompot Binnen	0	0	0
Sint Annaland	0.9	0.8	1.0
Stavenisse	0.7	0.6	0.7
Wemeldinge	0.7	0.7	0.8
Yerseke	0	0	0
Zierikzee	0	0	0

If the closure reliability of the storm surge barrier is improved, only Sint Annaland and Stavenisse show different required reinforcements than in the original situation. If the closure water level of the barrier is heightened, there are four locations for which the optimal reinforcements change compared to the original situation: Bergse Diepsluis, Krabbendijke, Sint Annaland and Wemeldinge. The changes in dike heightening are minimal, being a mere 10 centimeters compared to the original situation.

Figure 6.6 shows the costs as function of dike heightening at Stavenisse for 1 m SLR and heightened closure water level at the storm surge barrier. The optimal reinforcement, heightening the dike with 0.7 meter, is situated at the minimum of the total costs (= flood risk + investment costs). Again the optimum is very flat, indicating that the difference in total costs between 0.5 and 0.9 m dike heightening is not large.

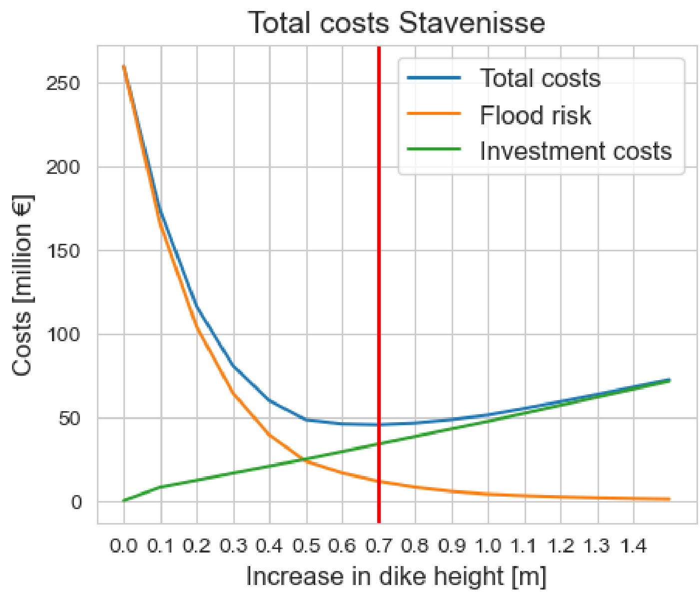

Figure 6.6: Costs for Stavenisse for 1 m SLR and heightened closure water level

6.2.1. Explanations results

The expectation was that the flood defence system would be more dependent on sea level rise than the results from the Eastern Scheldt case study indicate. The flat minimum of total costs results in minimal differences in optimal dike height. There are several explanations for this. These are:

- High construction costs of reinforcements

- Expected damage in case of flooding
- The dampening of water levels in the Eastern Scheldt by the storm surge barrier

Firstly, the minimal changes in optimal reinforcements can be explained by looking at the costs of construction. The costs of heightening a dike are be relatively high compared to the flood risk reduction. Related to that is reason number two: the value of the land. The area surrounding the Eastern Scheldt is not densely populated, therefore the expected damage in case of flooding is low. Due to the low expected damage, the impact of dike reinforcements is minimal. After all, the optimal reinforcement is found using the resulting flood risk, which is a multiplication of flood probability and expected damage in case of a flood. Thirdly, the dampening effect that the storm surge barrier has on the water level increase within the Eastern Scheldt due to sea level rise. If the water level does within the Eastern Scheldt not increase significantly, the hydraulic loads on the dikes of the Eastern Scheldt do not increase much either, resulting in a minor increase in flood probability.

7 Adaptation strategies for the height dependent system

In chapter 6, the results of the two case studies were discussed. These results however, are based on conditional calculations; conditional on sea level rise and barrier option. In reality the level of sea level rise is unknown. Therefore, it is not possible to make a decision based on the conditional results alone. The decision making needs to be as dynamic as the conditions determining it. An approach on how to deal with the unknown sea level rise is needed to define a strategy on how to use the optimisation results.

This chapter will give insight into an approach for the decision making process. The approach will be illustrated by making use of the results from the second case study on the 1.5 meter lower dikes at the Eastern Scheldt. The adaptation pathways approach, which is explained in subsection 2.3.2, will be used in this example. The pathways approach identifies possible actions that can be taken in case the current strategy does not suffice anymore.

A constraint is implemented in this example; at 1 meter SLR the closure water level needs to be heightened with 25 centimeters. Heightening the closure level with 25 centimeters will reduce the closure frequency and preserve the tidal motion. In the example, the dikes will be heightened at t = 0 as calculated to be optimal in case of 1 meter SLR at t = 100. The differences in optimal dike height are small for 1 meter SLR compared to 0.5 m or 0 m SLR at t = 100. Figure 7.1 illustrates the identified pathways. In this example, decisions can be made at t = 0 and at the the time that sea level rise equals 1 meter.

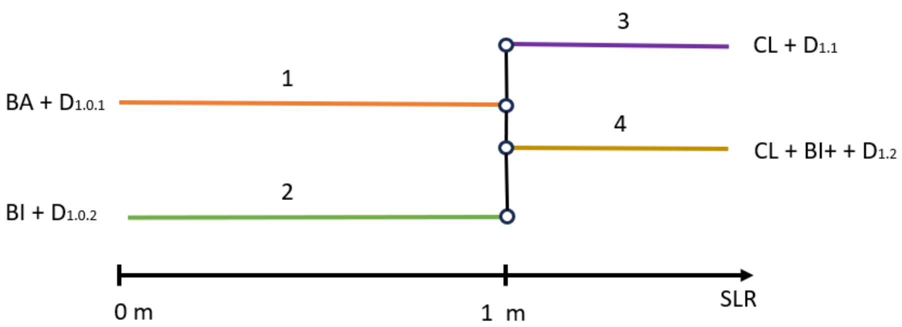

Figure 7.1: Pathways for second case study

In which:

- BA: unchanged storm surge barrier
- BI: storm surge barrier with improved closure reliability
- CL: heightened closure water level (3.25 m + NAP)

- $D_{1.0.1}$: Dikes heightened according to Table 6.6 and original barrier
- $D_{1.0.2}$: Dikes heightened according to Table 6.6 and improved closure reliability barrier
- $D_{1.1}$: Dikes heightened for 1 m SLR, infinite horizon and heightened closure water level storm surge barrier
- $D_{1.2}$: Dikes heightened for 1 m SLR, infinite horizon, heightened closure water level and improved closure reliability storm surge barrier

At t = 0, the decision maker has two options:

1. Make no changes to the storm surge barrier + heighten the dikes accordingly
2. Improve the closure reliability of the storm surge barrier + heighten the dikes accordingly

If the decision maker chooses to improve the closure reliability of the storm surge barrier at t = 0, he needs to take action when 1 meter sea level rise occurs. This is due to the constraint that is implemented; the closure water level needs to be heightened at 1 meter sea level rise to preserve the local ecology. The decision maker has two choices:

- Heighten the closure level and heighten the dikes accordingly (path 1-3 in Figure 7.1)
- Heighten the closure level, improve the closure reliability and heighten the dikes accordingly (path 1-4 in Figure 7.1)

If the decision maker has improved the closure reliability at t = 0, only path 4 can be taken at 1 meter sea level rise, since the closure reliability is already improved.

The total costs are influenced heavily by the moment at which 1 meter SLR is reached. To be able to compare the different paths in terms of total costs (flood risk + investment costs), the costs need to be converted to their Net Present Value (NPV) at t = 0. This enables the decision maker to compare the different alternatives. Figure 7.2 shows how this works for the example.

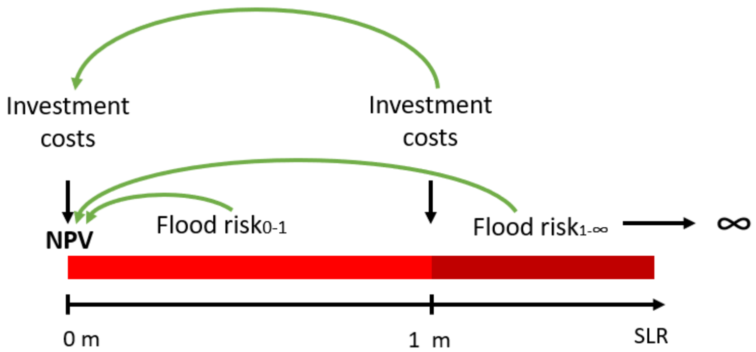

Figure 7.2: Converting costs back to NPV at t = 0

From t = 0 until 1 meter sea level rise is reached, the flood risk per year increases linearly as sea level rise increases as well. After 1 meter sea level rise, it is assumed that sea level rise will not increase any further and therefore flood risk is calculated with an infinite horizon. At the moment that 1 meter sea level rise is reached, the decision maker is required to make further investments into the system; this includes changes to the storm surge barrier as well as additional heightening of the dikes. In case the dikes are heightened more at t = 0, the investment at 1 meter sea level rise into heightening the dikes further will be smaller. The investment costs at 1 meter sea level rise as well as the flood risk are discounted back to t = 0 to be able to compared the total costs of the paths.

In Figure 7.3, the total costs (flood risk and investment costs) are shown. The left figure shows the costs for the situation in which 1 meter sea level rise is reached at t = 10 years and the right figure shows the costs for the situation in which 1 meter sea level rise is reached at t = 90 years.

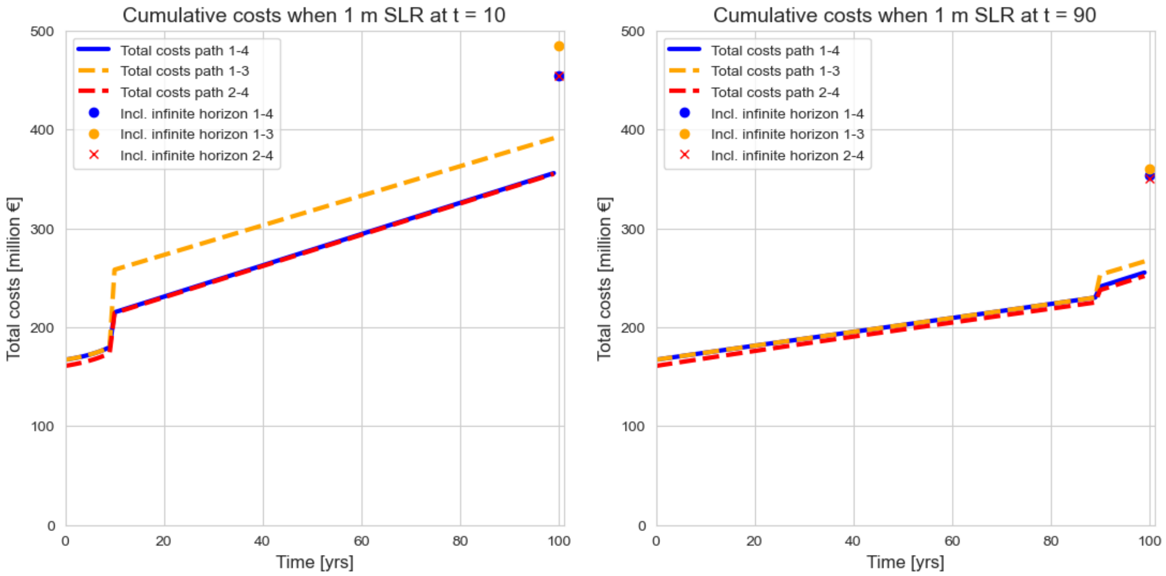

Figure 7.3: Cumulative costs for 1 m SLR at t = 10 and t = 90

A couple things can be noticed from these two figures. Firstly, the total costs are higher for all paths if 1 meter sea level rise is reached at t = 10 compared to t = 90. This makes sense since the hydraulic loads on the system are higher, so the flood risk will be higher as well. Secondly, the investment costs at 1 meter sea level rise are a lot higher when the closure reliability of the barrier is not improved (path 1-3). This is a result of higher hydraulic loads within the Eastern Scheldt due to more frequent failure of the barrier than if the closure reliability is improved. However, the costs for improving the barrier are not included in this graph, therefore, it will only be optimal to improve the barrier reliability if the investment costs of this change are lower than the difference in flood risk + investment costs at the dikes. Thirdly, the dots at t = 100 indicate the total costs (flood risk + investment costs) when the flood risk is included for an infinite horizon beyond t = 100. The flood risk resulting from the infinite horizon is equal in both scenarios.

As shown in this chapter, the timing of sea level rise influences the total costs and the timing of reinforcements. If sea level rise occurs sooner, the total costs will be higher. Adaptation pathways can be drawn to visualise the possible options for the decision maker, after which the optimisation method can help in determining the optimal reinforcements for a scenario.

8 Discussion

This chapter will discuss the findings as set out in chapter 6. The assumptions made in this thesis will be considered as well as the usefulness and applicability of the results. Firstly, the optimisation method will be discussed, next the case study will be considered.

8.1. Optimisation method

In this section the optimisation method itself is discussed. In this thesis the same function of total costs is used as in Dupuits, Schweckendiek, and Kok [17]. Ciullo included the transfer of risk across the flood defence system his model, while this thesis assumes that the water body between the lines of defence is large enough not to influence the hydraulic loads on the first flood defence in case of reinforcements at the second flood defence. Additionally, the second flood defence consists of multiple parts which can be modified separately and independently from each other. Compared to Ciullo et al. [13], the optimisation approach of this thesis focuses more on the situation when a flood defence is already built, instead of justifying the initial construction of a new line of defence. This thesis will contribute to filling the knowledge gap and helping decision makers in constructing more effective reinforcements. The simplicity of the method increases its applicability.

Secondly, the optimisation balances the possible reinforcements with the resulting flood risk. The flood risk is based on the failure probability and the expected damage. In this thesis, the expected damage is constant, independent of which dike section fails. In reality, the location of the failure matters and determines how fast and to what extent the area behind it floods. Additionally, the inundation depth influences the damage and fatalities,. The expected damage used in this thesis is the damage related to the worst case scenario. Simplifying the damage to this value therefore overestimates the flood risk and promotes reinforcements. If different flood scenarios would be taken into account, less reinforcements would be needed and the outcome could be more accurate. Thirdly, the dependency between failure mechanisms is not taken into account in the optimisation. Taking the dependency into account could have reduced the failure probability, because the failure due to revetment failure or overtopping happens under similar conditions. However, since the failure probabilities of both revetment failure and overtopping are small, taking the dependency into account would only have a minor influence on the results.

8.1.1. Sensitivity to changing hydraulic loads

Since the reinforcements in the case study on the Eastern Scheldt were not sensitive to sea level rise, a second case study was performed. However since the same area was used as in the first case study, the results were only minimally dependent on sea level rise for the second case study as well. The dampening effect of the Eastern Scheldt barrier is too large for the reinforcements to be very dependent on sea level rise. Since there were minor differences due to sea level rise, it can be concluded that the method used in the two case studies does include the changes in hydraulic loads and responds to them. An interesting case study would be to research a flood defence system with multiple lines of defence with a first barrier that dampens the sea level rise less than the Eastern Scheldt barrier.

8.2. Case study

In this section, the case study results will be discussed. Assumptions and limitations to the results are included.

8.2.1. Morphology of the Eastern Scheldt

Morphology is kept out of scope in this thesis to simplify the project. In order to substantiate the results further, it should be investigated whether the morphology in the Eastern Scheldt changes significantly over time to the point where it influences the hydraulic conditions in the Eastern Scheldt. It is assumed in this thesis that the morphology remains constant, which is not necessarily true. The expectation is that the tidal flats will erode [52], therefore creating deeper waters in front of the dikes of the Eastern Scheldt, reducing the wave dissipation and resulting in higher waves on the dikes. If the tidal flats erode significantly, this could impact the optimisation outcome and more or other reinforcements than currently calculated could be optimal, regarding overtopping and revetment. Higher total costs can be expected in case of increased hydraulic loads on the dikes due to eroded tidal flats.

8.2.2. Climate change

Climate change is simplified severely in this thesis by only including sea level rise. The sea level rise considered in this thesis is limited to 1 m of sea level rise, while the projections of the IPCC [36] indicate that the last KNMI projections [28] should be adapted to even higher levels of sea level rise than currently calculated. Sea level rise could exceed 1 meter within the time frame considered (100 years). The water levels outside the storm surge barrier in such case would translate to higher water levels within the Eastern Scheldt. The conclusions drawn for 1 m sea level rise might not hold for an increased sea level rise. The higher hydraulic loads might require reinforcements for revetment failure or overtopping. Additionally, the response of the storm surge barrier to higher loads has not been considered. The storm surge barrier must be checked for structural failure when sea level rise of such magnitude of more than 1 meter arises.

8.2.3. Options for the Eastern Scheldt storm surge barrier

Currently only two options for the storm surge barrier are included in this thesis. Improving the closure reliability is purely theoretical. The question of how to actually improve the closure reliability is thus not thought out. The manner in which the closure reliability could be improved is an option worth of further research.

8.2.4. Overtopping

Several simplifications were applied to calculate the optimal reinforcements and flood risk for overtopping. Firstly, overtopping was calculated per dike segment, not per dike section. When considering dike segments, the scale does not introduce large errors in the hydraulic boundary conditions. However, this simplification prevents individual weak spots in a dike to be taken into consideration. Instead, the entire dike segment is simplified to a single cross section. The dike can be strengthened, but only if the entire segment is strengthened. Since partial strengthening is not possible in the model used in this thesis, the costs related to heightening the dike are also related to the entire dike segment length, which results in higher costs than when partial heightening would be possible. Secondly, only heightening the dike was included as reinforcement option, while several other option could be possible. Other options could be increasing the roughness of the dike slope or increasing wave dissipation in front of the dike (i.e. by placing vegetation). These reinforcements are likely to be less effective than heightening the dike, but can be implemented as well. The solution space that was included regarding overtopping is thus not complete. Thirdly, the dike height increase in the first case study uses increments of 0.5 meter, which is a reasonable assumption due to constructability. This assumption limits the solution space and improves the computation speed. As shown in the second analysis where increments of 10 centimeters were used, smaller increments can cause small differences in the optimal solution.

8.2.5. Revetment failure

The revetment failure probability is calculated using information from the WBI assessment. However, the revetment height was simplified to assume that along each dike segment the same height is applicable, which is not necessarily true. This choice was made because the revetment could only be calculated at the IMPLIC station, therefore it made sense to calculate the failure probability for that location. Secondly, the revetment failure probability in this thesis is only calculated for four locations in the Eastern Scheldt. Calculating the revetment failure for all barrier options and sea level rise scenarios would take too much time. Therefore, the choice was made to limit the number of locations. The locations were grouped according to wave height and revetment height to ensure similar results. The optimisation for revetment failure might be less accurate due to this simplification. Thirdly, some revetments had a failure probability smaller than 1 in 100,000 years. Hydra-NL cannot calculate with longer return periods and therefore these locations were given the return period of 100,000 years, which is thus an underestimation. However, it can be assumed that this has a negligible effect on the final outcome, because a return period of a 100,000 years is extremely large already.

9 Conclusions and recommendations

This chapter will discuss the conclusion of this thesis. The conclusion drawn from the case study on the Eastern Scheldt will be discussed, after which the research question and the conclusion on the general optimisation method will be discussed.

9.1. Optimisation method conclusions

The purpose of this thesis was to propose a method for optimising reinforcements in a flood defence system with multiple lines of defence. In this section the conclusions regarding the used optimisation method will be discussed.

The research question is:

How can reinforcements in a flood defence system with multiple line of defence be optimised under changing boundary conditions due to climate change?

The method for optimisation of reinforcements captures the interaction between multiple lines of defence by using the change in boundary conditions for different scenarios. The boundary conditions for the second line of defence are a result of the influence of the first lines of defence. Calculating the change in boundary conditions therefore connects the two lines of defence, making a system optimisation possible. Additionally, using the hydraulic boundary conditions as the connecting metric, the influence of sea level rise can be translated into this metric. This approach can be applied to other flood defence systems with multiple lines of defence. A drawback of the method is that it optimises conditionally on sea level rise. However, the amount of future sea level rise is unknown. Using adaptation pathways, the results from the optimisation can be used to identify possible strategies to take in the future. The timing of sea level rise determines the resulting total costs.

9.2. Case study conclusions

There were three options for the storm surge barrier at the Eastern Scheldt included in the optimisation: changing nothing, improving the closure reliability and heightening the closure water level. Failure due to macro-stability was found to be independent of water level. The reinforcements that would be required for macro-stability are therefore required in all possible scenarios. For overtopping and revetment failure at the dikes along the Easter Scheldt there are no reinforcements required. Possible explanations could be the low expected damage to the area in case of a flood, the dikes that were built at 'Delta hoogte' and the revetment has been reinforced in the period between 1997 and 2015. The case study reinforcements are found to be independent from sea level rise.

9.3. Recommendations

There are multiple recommendations that can be derived from this thesis. Firstly, it should be validated whether such a method works for other flood defence systems with multiple lines of defence. The method is only applied to the Eastern Scheldt and should be tested at other locations. A flood defence system with a barrier that dampens the sea level rise less would be interesting to consider. Secondly, a study should be done to investigate the effect of morphology on the hydraulic boundary conditions at the dikes. The study should investigate whether changes in the morphology influence the boundary conditions in the Eastern Scheldt significantly and should therefore be included in further optimisations, or whether the influence of morphology is minimal on these time scales and can be neglected.

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A

Dike assessment graphs

A.1. Current status dikes Eastern Scheldt

As introduced in section 2.1, all flood defences in the Netherlands had to be checked against the new safety standards. The assessments of the dikes can be found in *Informatiehuis water* [23]. A summary of the dike sections along the Eastern Scheldt is given in the table below. Section 219 in the table is also known as the Oesterdam.

Table A.1: Safety targets dikes Eastern Scheldt

Section	Length section[km]	Lowest Pf	Signal Pf
26-2	20.7	1:1000	1:3000
26-3	21.9	1:1000	1:3000
27-2	36.85	1:10 000	1:10 000
219	11.45	1:10 000	1:30 000
31-2	28.64	1:3000	1:10 0000
30-1	22.56	1:1000	1:3000
28-1	23.90	1:300	1:1000

The dike section are assessed according to the WBI2017. The failure modes are divided into categories, indicating how well the dike performs for that failure mode compared to the safety target. The categories are as follows:

1. I: the dike section suffices the signal Pf easily in the cross-section
2. II: the dike section suffices the signal Pf in the cross-section
3. III: the dike section suffices the lowest Pf and possibly the signal Pf in the cross-section
4. IV: section suffices possible the lowest Pf and/or signal Pf
5. V: section does not suffice the lowest Pf along the section
6. VI: Section does very clearly not suffice lowest Pf along the section

Table A.2: Assessment dikes according to new safety targets [23]

Failure mode	Code/Section	26.2	26.3	27.2	28.1	30.1	31.2	219
Height structure or gras cover erosion crest and inner slope	HTKW	I_t	-	II_t	-	II_t	II_t	I_t
Gras cover erosion	GEKB	III_t	III_t	II_t	I_t	II_t	II_t	IV_t
Piping	STPH	II_t	III_t	II_t	II_t	III_t	III_t	IV_t
Macro stability inner slope	STBI	V_t	V_t	VI_t	IV_t	IV_t	V_t	II_t
Gras erosion outer slope	GEBU	-	IV_t	V_t	II_t	III_t	IV_t	III_t
Macro stability outer slope	STBU	II_t	II_t	V_t	II_t	II_t	II_t	I_t
Micro stability	STMI	II_t	II_t	II_t	II_t	II_t	II_t	I_t
Stability stone cover	ZST	II_t	II_t	II_t	II_t	VI_t	II_t	VI_t
Gras cover stability outer slope	GABU	II_t	II_t	II_t	II_t	II_t	II_t	I_t
Gras cover stability inner slope	GABI	V_t	V_t	V_t	II_t	II_t	II_t	I_t

From Table A.2 the conclusion can be drawn that the failure modes occurring in the dike sections are very similar. The stability of the inner slope seems to be a problem in all sections, while micro instability and the stability of the grass cover are no problem in any of the sections. Table A.2 shows that the height of the dikes is a problem nowhere. The dikes along the Eastern Scheldt were built at 'Delta' level, because in the past they had to withstand much higher waves than the ones currently present in the Eastern Scheldt. The reason for this is that the dikes were built before the storm surge barrier. Due to fact that the dikes were built relatively high, the overtopping failure mechanism is not the dominating failure mode. However, if sea level rise were to continue, then overtopping could become relevant again for the dikes.

A.1.1. Spatial variation of failure mechanisms

The spatial distribution of the failure modes is shown in the figures below. It could be the case that a failure budget is exceeded due to two weak spots in a dike section, but it could also be the case that the entire dike section does not suffice.

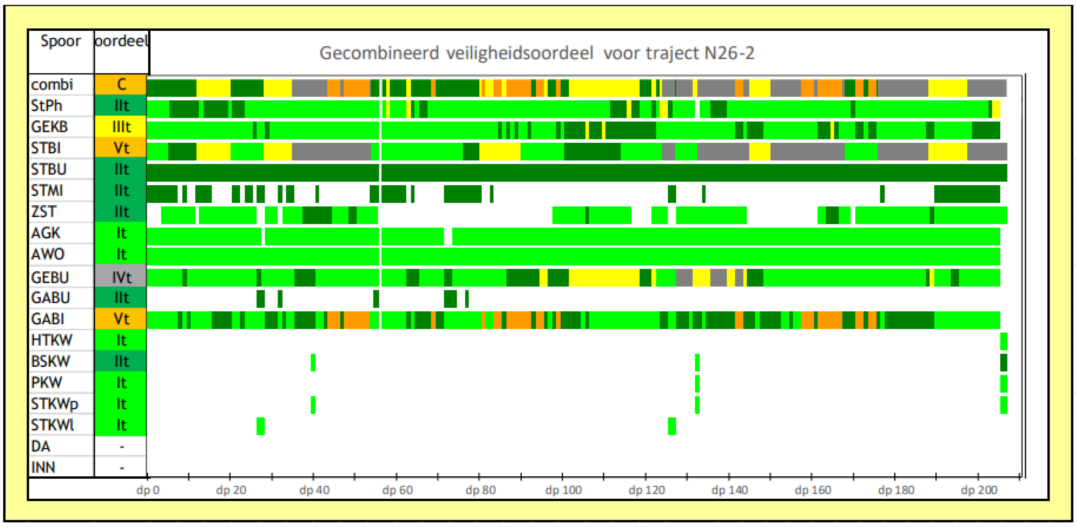

Figure A.1: Failure mechanisms assessment dike 26-2 [23]

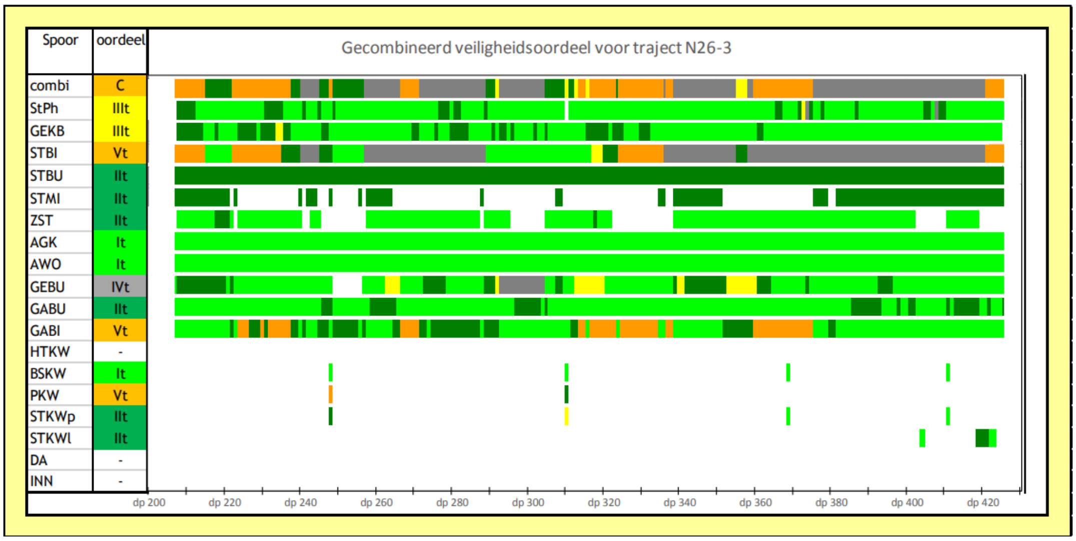

Figure A.2: Failure mechanisms assessment dike 26-3 [23]

Figure A.3: Failure mechanisms assessment dike 27-2 stretch one [23]

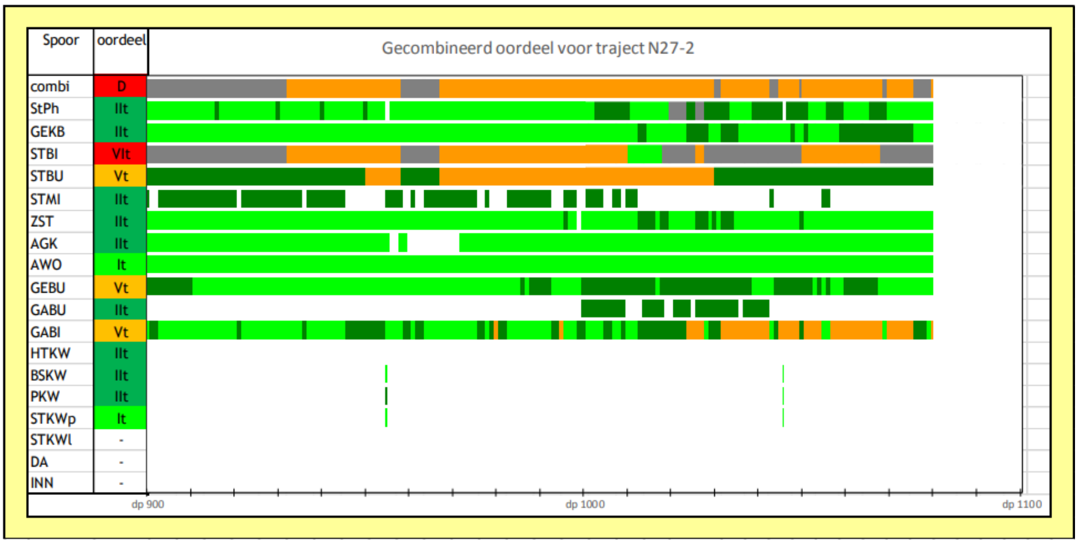

Figure A.4: Failure mechanisms assessment dike 27-2 stretch two [23]

Figure A.5: Failure mechanisms assessment dike 31-2 stretch one [23]

Figure A.6: Failure mechanisms assessment dike 31-2 stretch two [23]

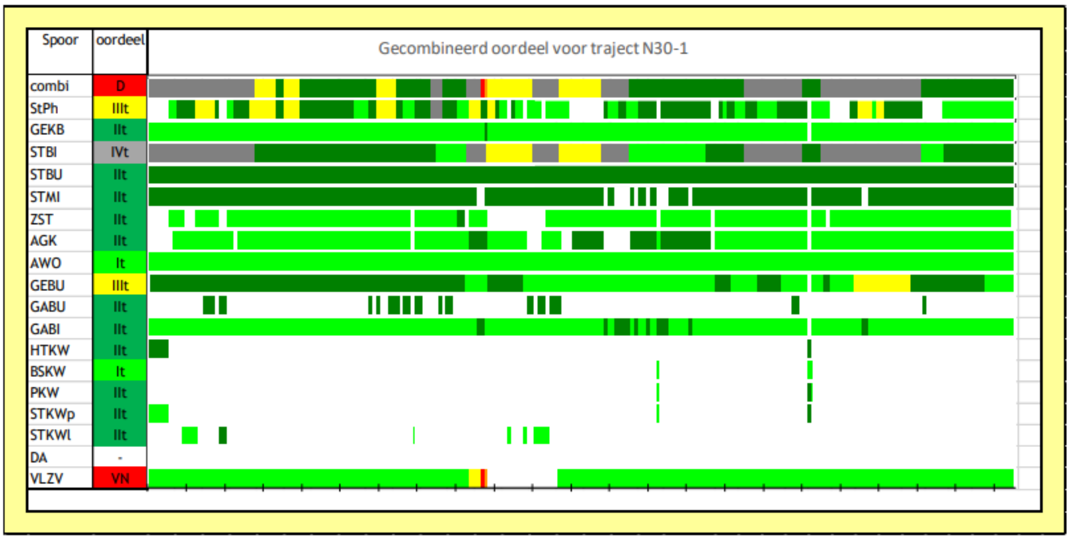

Figure A.7: Failure mechanisms assessment dike 30-1 [23]

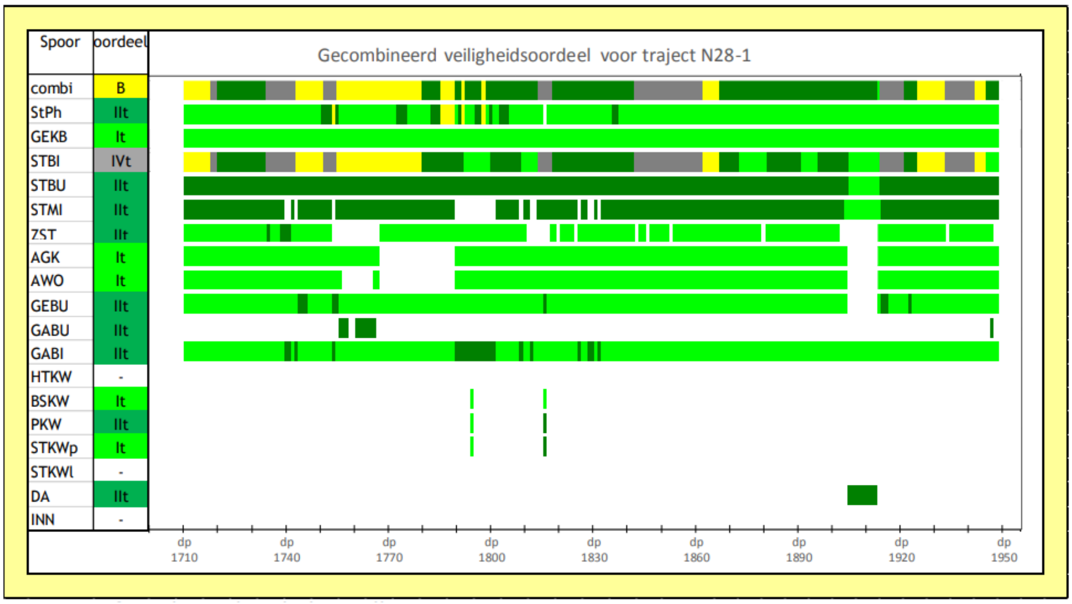

Figure A.8: Failure mechanisms assessment dike 28-2 [23]

B
Input Hydra-NL

The input that was fed into the Hydra-NL program can be found in this chapter.

For the input for the Eastern Scheldt barrier, multiple input table are used. The failure probability per closure is described by the probability that a number of gates fail. The table that is used as input is shown in Figure B.1:

*Falende schuiven	Bemand	Onbemand
0	0.9864754288	0.9395069053
1	0.0118000000	0.0541000000
2	0.0003810000	0.0018300000
5	0.0001880000	0.0020000000
10	0.0005880000	0.0009720000
16	0.0003770000	0.0006070000
31	0.0001700000	0.0002310000
47	0.0000000712	0.0000000947
62	0.0000205000	0.0007530000

Figure B.1: Input failure probability per closure

The first column states how many gates fail, the second column is the scenario for which the barrier is closed manually and the third column is the scenario in which an automatic emergency closure is performed.

C
Macro-stability reinforcements

This appendix includes the maps of optimal macro-stability reinforcements per dike segment.

Figure C.1 shows the optimal reinforcements for macro-stability for dike segment 26-2. The dike segment requires multiple berms and a stability screen.

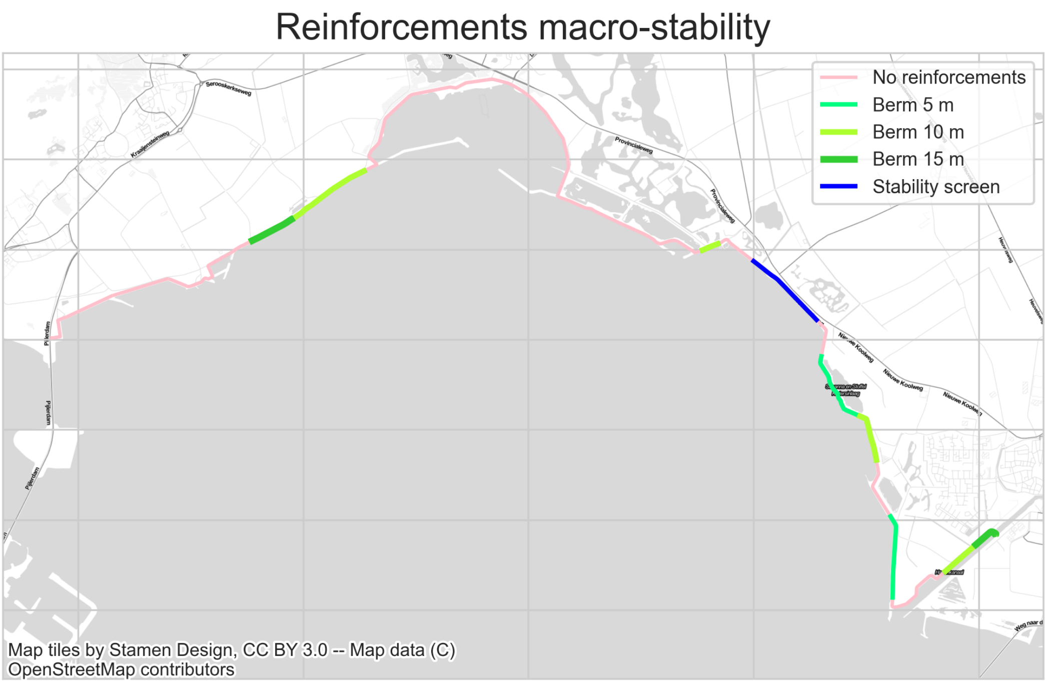

Figure C.1: Macro-stability reinforcements dike segment 26-2

Figure C.2 shows the optimal reinforcements for macro-stability for the dike segment 26-3. There are multiple berms applied, as well as stability screens. The stability screens in the eastern part of the dike segment are located in front of a town: Bruinisse.The stability screen does not require solving any building conflicts, therefore it is cheaper to apply instead of a berm in densely populated areas.

Figure C.3 shows the optimal reinforcements for macro-stability for dike segment 27-2. Multiple stretches need a berm, up until 25 meters berm length. Stability screens are also applied in three locations. Similar to 26-3, the stability screens are applied in locations where buildings are present near the dike.

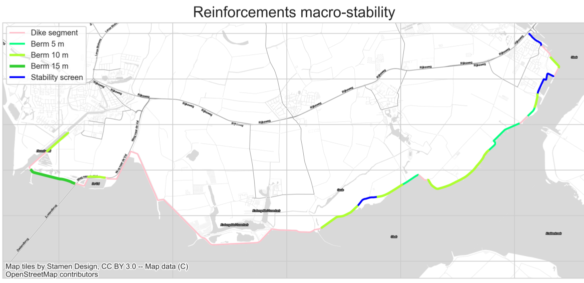

Figure C.2: Macro-stability reinforcements dike segment 26-3

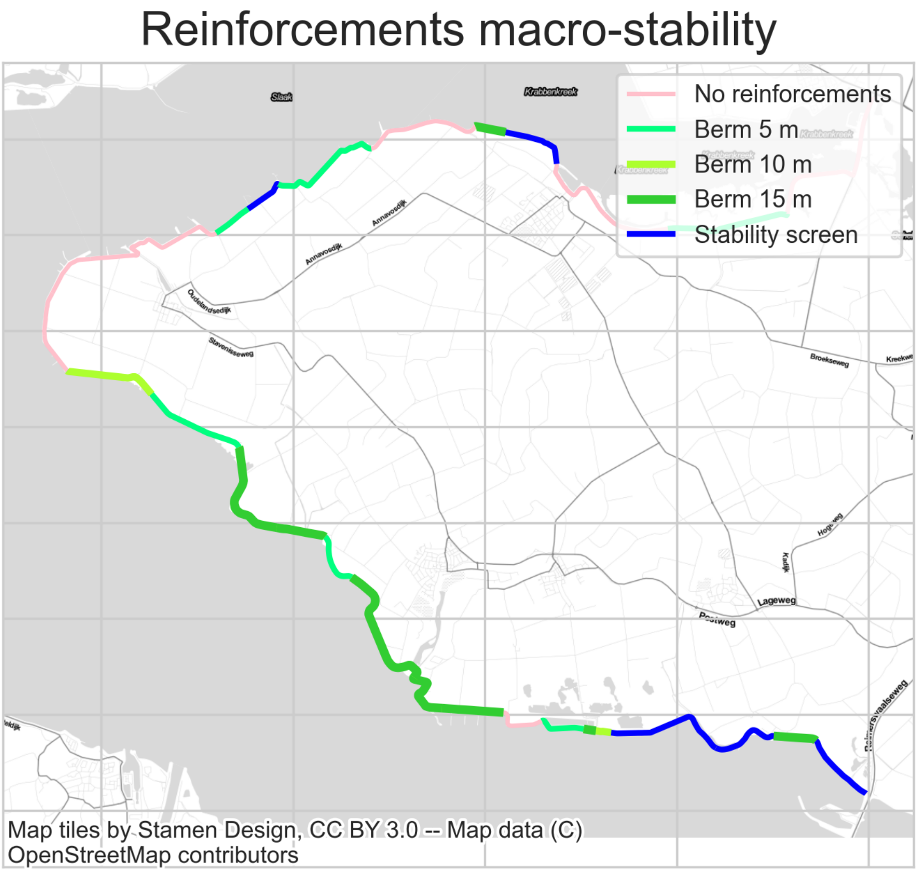

Figure C.3: Macro-stability reinforcements dike segment 27-2

Figure C.4 shows that the dike segment does not need many reinforcements for macro-stability. Again the stability screen is located at a town: Colijnsplaat.

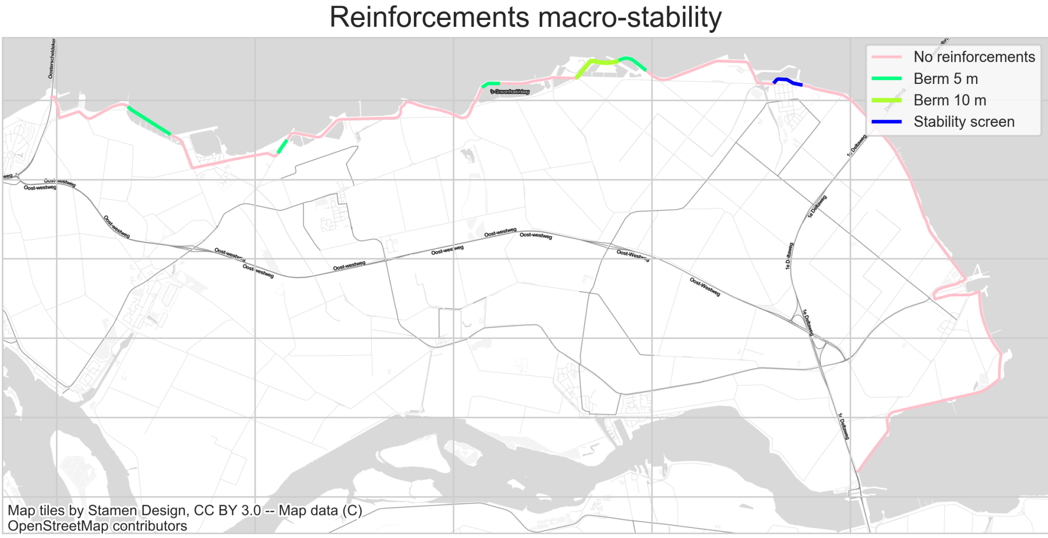

Figure C.4: Macro-stability reinforcements dike segment 28-1

In Figure C.5, only small berms of 5 meters are shown. The buildings in the area are not within this range of the dike, therefore the berms are the optimal reinforcements.

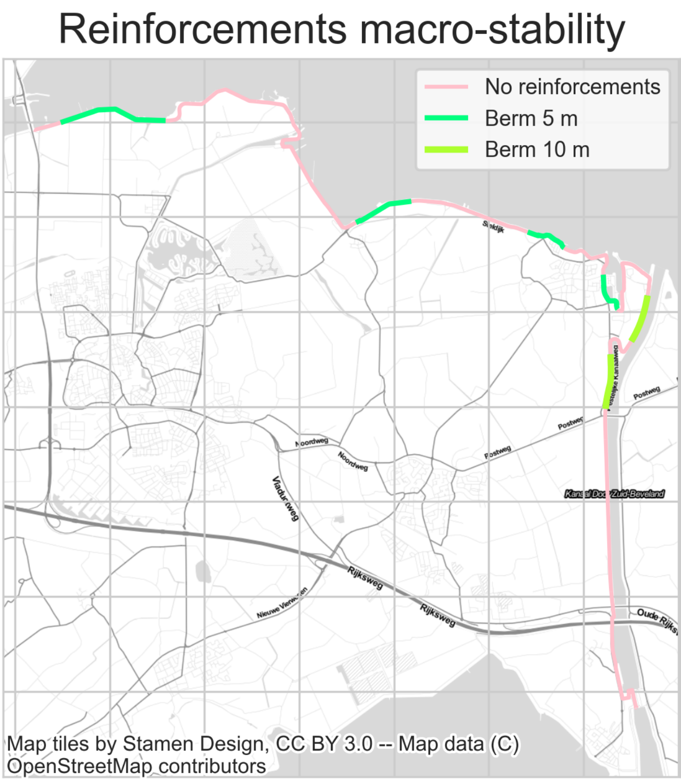

Figure C.5: Macro-stability reinforcements dike segment 30-1

Figure C.6 shows that dike segment 31-2 requires multiple berms. These berms are not large enough to reach the town next to the dike. Therefore these are the optimal solution.

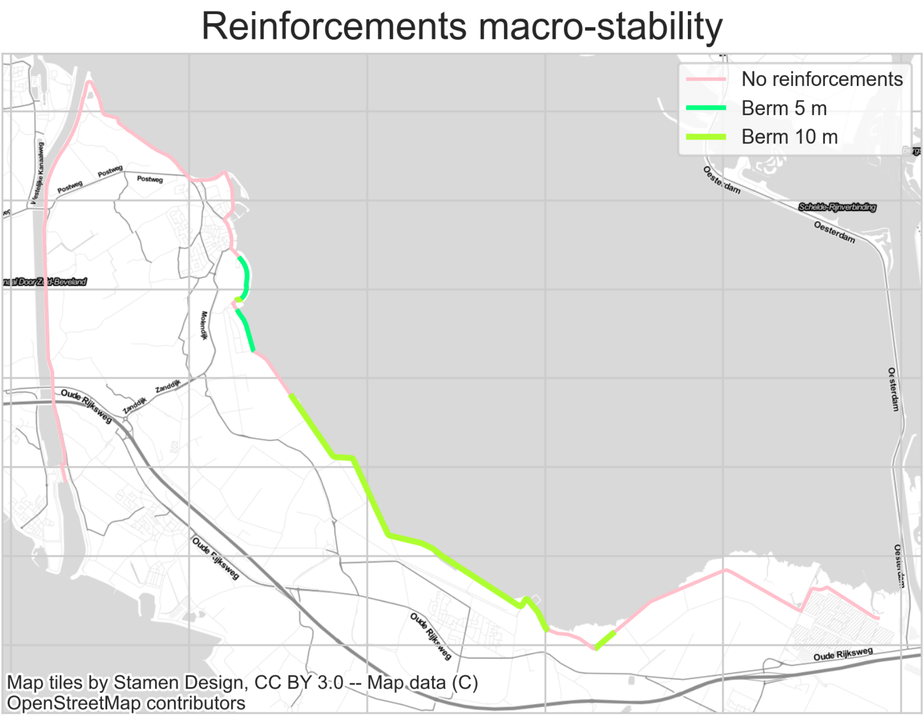

Figure C.6: Macro-stability reinforcements dike segment 31-2