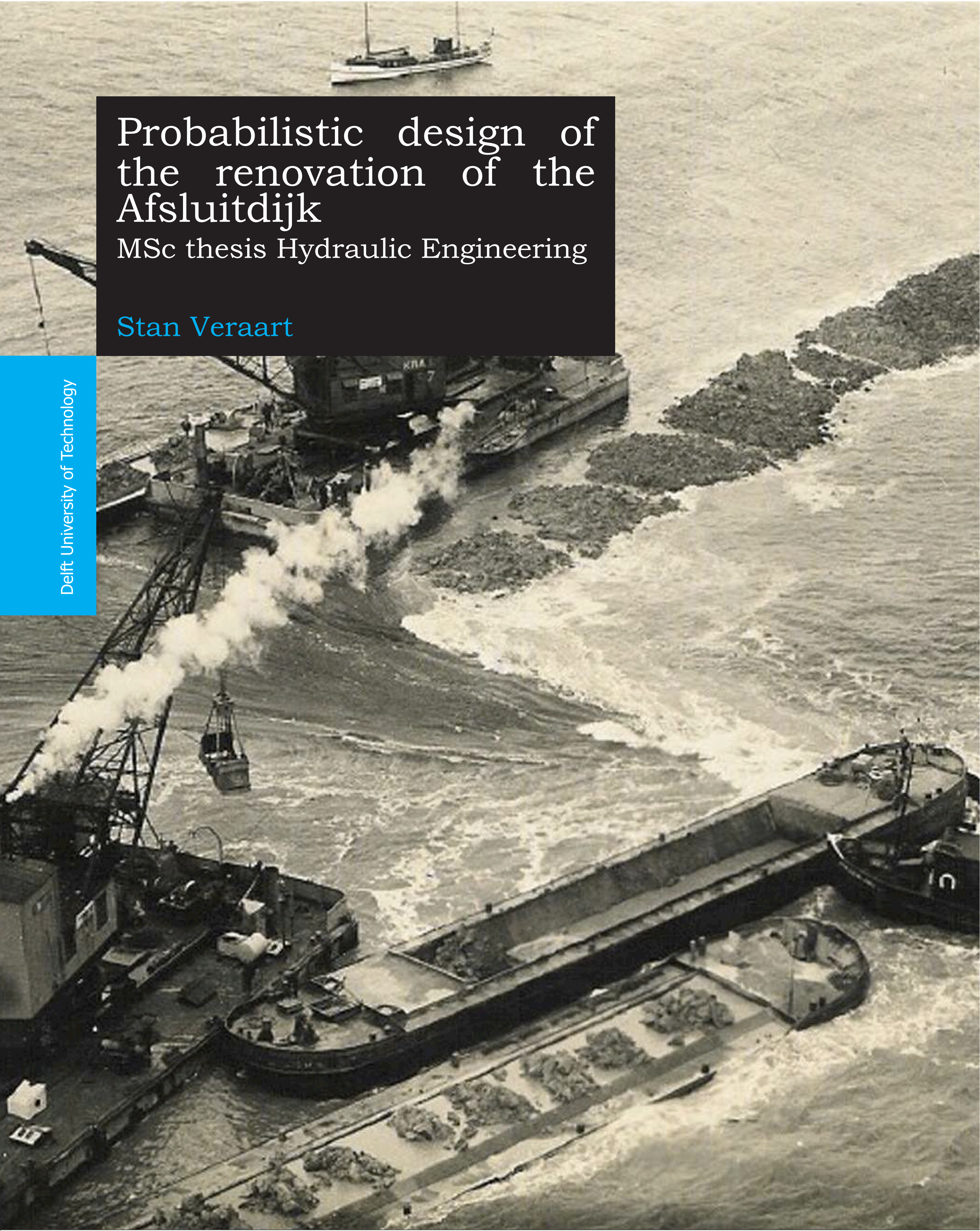

Probabilistic design of the renovation of the Afsluitdijk

MSc thesis Hydraulic Engineering

Stan Veraart

Delft University of Technology

Probabilistic design of the renovation of the Afsluitdijk

MSc thesis Hydraulic Engineering

by

Stan Veraart
1332503

in partial fulfillment of the requirements for the degree of

Master of Science
in Civil Engineering

at the Delft University of Technology,

Thesis committee

Chairman	Prof. dr. ir. M. Kok	Delft University of Technology
Supervisor	ir. P. Ravenstijn	Witteveen+Bos
	dr. ir. T. Schweckendiek	Delft University of Technology
	ir. H.J. Verhagen	Delft University of Technology
	ing. H.J. Regeling	Rijkswaterstaat

Preface

This thesis is the final step to fulfill my graduation as a Master of Science at Delft University of Technology. During my masters in Hydraulic Engineering I specialized in the track of Flood Risk, which really got me interested in the subject of this thesis. To do research on one of the most iconic Dutch hydraulic structures and to investigate the new flood safety standard that will be introduced in the near future is really amazing.

This research is done in cooperation with Witteveen+Bos. W+B is an engineering and consulting firm active in the field of water, infrastructure, environment and construction. W+B is currently working on the redesign of the Afsluitdijk and they are therefore interested in the effects of the implementation of the new safety standard. Without the help from the employees at W+B I would not have been able to complete this thesis. Special thanks go to my daily supervisor, ir. P. Ravenstijn Ravenstijn, for supporting me throughout my research and helping me to find the right people for the questions I had during my thesis. Furthermore I would like to thank ir. P.E.M. Schoonen from W+B for helping me starting up my MSc thesis and for determining my research subject and goals. The other employees from W+B I would like to thank for their help on my thesis are mw W.S. de Raadt MSc, ir. A.L. de Jongste, ir. S. te Slaa, P.T.G. van Tol MSc, ing. E Schulte Fischedick MSc, ir. G.R. Spaargaren, mw ir. M.L. Drost and F.M. Meins MSc.

I would like to thank the rest of my thesis committee, Prof. dr. ir. M. Kok, dr. ir. T. Schweckendiek, ir. H.J. Verhagen and ing. H.J. Regeling, as well for the reviewing and the comments on all the work throughout the process. The progress meetings were very pleasant and the discussions and comments really helped me to obtain the result I have achieved at this moment.

At the beginning of my thesis I had some interesting conversations about my research and I would like to thank R.B. Jongejan and W. Kanning for their time and help with the starting phase of my thesis. Also I would like to thank N. Kramer for her help about understanding the previous studies on the flood safety of the Lake IJssel region. Finally I would like to thank the VNK2-project team for giving me a license to PC-Ring which really helped me to obtain the results at the end of my thesis.

Stan Veraart
Delft, September 15, 2014

Contents

Summary

At this moment the concept of the safety standards for flood defences in the Netherlands is changing into a risk-informed approach. This new safety standard, which will be introduced in 2017, is expressed in terms of a probability of flooding instead of probabilities of exceedance of design loads. In the new safety standard the effects of a flood are taken into account, which leads to a risk-informed approach.

The implementation of the new safety standard introduces challenges because a fully risk-informed approach is new and extra uncertainties and more political discussions are introduced by this approach. In this thesis the focus is on the challenges during the transition period between switching from the current to the new standards. This transition period will especially introduce challenges for ongoing projects regarding strengthening of flood defences. The redesign in these projects is done according to the current standard but to prevent negative assessments in the future it is useful to take the future standard into account. This is for example the case for the renovation of the Afsluitdijk.

The Afsluitdijk is a separation dam between the provinces of North Holland and Friesland and separates the Wadden Sea and Lake IJssel. An assessment in 2006 showed that the Afsluitdijk does not meet the current standard and therefore a renovation of the Afsluitdijk is needed. In the design alternatives the new safety standards are not taken into account and the question rises how safe the new design has to be according to the new standard. The new standard for the Afsluitdijk, which is expressed as a probability of failure, must be known to determine the optimal redesign. At this moment it is uncertain what this standard is going to be. An economic optimization of the Lake IJssel region (Zwaneveld and Verweij, 2014) proposes a standard for the Afsluitdijk with a probability of failure of 1/9 400 per year. Although discussion remains about the proposed probability of flooding it is used in this thesis.

The goal of this thesis is to develop a probabilistic design for the renovation of the Afsluitdijk that takes the new risk-informed standard into account. In a probabilistic design both the distributions of the load and the strength of a flood defense are taken into account resulting in a (pre-defined) probability of failure. This differs from current design standards which make use of characteristic values for the strength and the load and safety factors to guarantee a safe design. To cope with future uncertainties it is recommended to make use of an adaptable design for the renovation of the Afsluitdijk. This main objective is translated into the following research question:

"*What is a probabilistic, cost-effective and adaptable design of the Afsluitdijk?*"

Before the renovation of the Afsluitdijk is investigated the current probability of failure of the existing Afsluitdijk is estimated. A fault tree is used to account for the different failure mechanisms of the Afsluitdijk. Failure of the Afsluitdijk occurs when the dike body fails or when a hydraulic structure fails. The hydraulic structures are outside the scope of this thesis because the main focus is on the optimization of the dike body. Therefore in the assessment of the current probability of failure the sluice complexes are not taken into account.

For the Afsluitdijk the following failure mechanisms are taken into account:

- Wave run-up and overtopping. If waves overtop the crest of the dike the inner slope can erode resulting in a breach.
- Instability of the revetment on the outer slope. If the revetment is washed away the dike body can erode which may result in a breach.
- Macro instability of the outer or inner slope. If the water pressures inside the dike body are too high sliding of the slope can occur which results in a possible breach.
- Piping. Seepage water erodes particles from underneath the dike if the hydraulic head is large enough. The erosion forms a pipe which grows from the inner to the outer side and this can lead to a settlement and failure of the dike.

With a model that is developed in this thesis these failure mechanisms are assessed on the probability of failure for a single cross section. This cross section is located in the middle of the Afsluitdijk where the hydraulic loads are the most severe. Following from these assessments it is concluded that the failure mechanisms of piping and macro instability have a negligible influence on the total probability of failure.

The failure mechanisms of overtopping and instability of the revetment have the most influence on the total probability of failure, which is typical for sea dikes. To assess the total probability of failure all the dike sections of the Afsluitdijk are assessed for these mechanisms with PC-Ring. PC-Ring is a software module that is developed to assess the probabilities of flooding and the consequences of flooding for all the dike rings in the Netherlands. In PC-Ring the correlation between failure mechanisms and the dike sections is taken into account as well. With this the results shown in Table 1 for the probability of failure for the Afsluitdijk are obtained.

failure mechanism	**Probability of failure**	**Return period**
overtopping	4.44E-03	225
instability stones	1.67E-03	600
total	**5.00E-03**	**200**

Table 1: Total probability of failure of the Afsluitdijk for the current design.

The results for the total probability of failure for the Afsluitdijk are close to the assumed probability of failure of the Afsluitdijk in the economic optimization study for the Lake IJssel region mentioned earlier. In this study it is assumed that the Afsluitdijk in 2012 has a strength of 1/250 per year, which is close to the 1/200 found in this MSc research. The current safety standard for the Afsluitdijk is safety against hydraulic loads with an exceedance frequency of 1/10 000 per year. According to this standard the current safety of the Afsluitdijk is insufficient and improvement in the design is needed.

In the current safety standard this exceedance frequency is chosen to comply with the adjacent flood defences, regardless of the possible consequences of failure of the Afsluitdijk. The consequences of a breach somewhere in the Afsluitdijk are not as severe as failure of a regular dike with valuable assets and inhabitants in the hinterland. In the new safety standards these consequences will be taken into account which could result in lower probabilities of failure. However it can be stated that the current configuration of the Afsluitdijk with a probability of failure of around 1/200 per year is insufficient. Therefore improvement of the Afsluitdijk is needed and the new designs by Witteveen + Bos are evaluated and a reference alternative is optimized.

In the assessment and the optimization of the renovation design only the failure mechanism of wave run-up and overtopping is considered. With the design instrument of OI2014[1] and earlier results the standard for a single dike section for a single failure mechanism is determined. With a standard of 1/9 400 per year the standard for a single dike section for the failure mechanism of wave overtopping is a probability of failure of around 1/22 500 per year.

The reference alternative B3+, shown in in Figure 1, that is developed by W+B is both calculated in the model developed in this MSc research and in PC-Ring. This reference alternative B3+ is not the final design but it is developed to show the feasibility and affordability of the renovation of the Afsluitdijk. The results are given in Table 2.

Model	**P_f [1/year]**	**R [years]**
Matlab	5.06E-05	19 750
PC-Ring	6.15E-05	16 250

Table 2: Probability of failure of the reference alternative B3+ for wave run-up and overtopping for dike section in the middle of the Afsluitdijk.

[1]OI2014 (Ontwerp Instrumentarion 2014 in Dutch) is a design instrument for flood defences especially developed for the transition period from the old standard into the new standard.

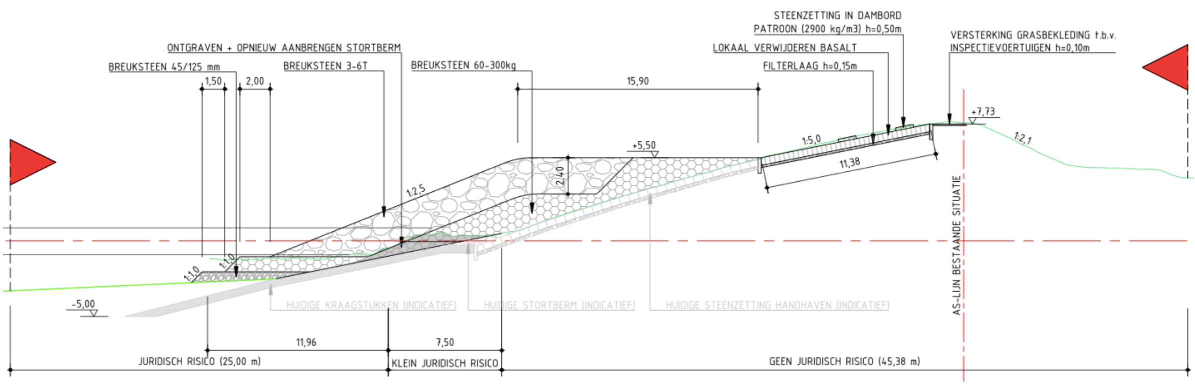

Figure 1: A cross section schematization of the reference alternative B3+ developed by W+B.

It is concluded that the probability of failure of this reference alternative B3+ is higher than the standard. With the assumed safety standard this design is not sufficient enough and the design must be improved. In the optimization of the reference alternative the probability of failure of 1/22 500 is set as the upper limit.

The aim of the optimization of the reference alternative is to minimize the investment costs for a design that meets the safety standard of 1/22 500 per year for the failure mechanism of overtopping. In the optimization five variables that have the most influence on the probability of failure are varied resulting in set of alternatives. The variables with the largest influence coefficients are:

- dike height (h_{crown})
- berm height (h_B)
- berm width (B)
- upper slope ($tan(\alpha_{up})$)
- lower slope ($tan(\alpha_{low})$)

For all the alternatives the investment costs are estimated. These costs are estimated with cost numbers given by W+B for the following activities:

- remove basalt and re-use in work (5 €/ton)
- transport and apply dike clay (25 €/m^3)
- transport and apply granular filter (25 €/ton)
- transport and installation concrete columns (160 €/m^2)
- transport and installation rubble (25-30 €/ton)

The possible alternatives with the lowest costs are found to be cheaper than the reference alternative B3+, while having a larger safety against flooding. The results of the reference alternative B3+ and the recommend optimized alternative are shown in Table 3. A schematization of the optimized alternative is shown in Figure 2. It is concluded that optimization of the design results in lower investment costs while improving the safety against failure due to overtopping.

alternative	h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
B3+	7.73	5.5	16	5	2.5	19 750	19 000
Optimized	8	5	7.5	4.5	3.5	31 250	15 250

Table 3: Cost estimation for the alternative with minimal costs and the reference alternative.

To increase the safety in the future the berm can be widened or the angle of the lower slope can be decreased. The investments costs for these two measures have the same efficiency up to extra investment costs of € 3 000 per meter, which improves the safety up to a probability of failure of 1/125 000 per year.

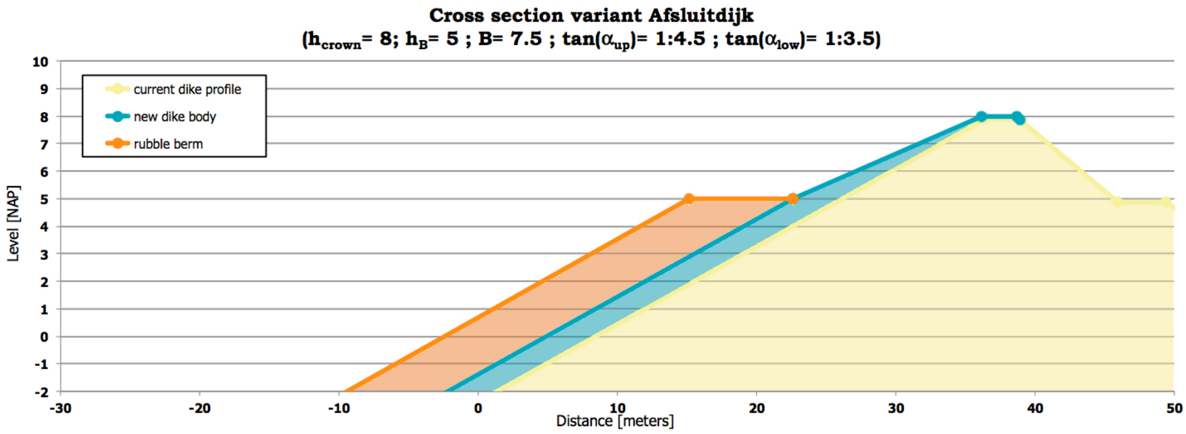

Figure 2: Schematization of the cross section of the recommended adaptable alternative.

To cope with uncertainties it is recommended to make an easily adaptable design for the renovation of the Afsluitdijk. A conclusion from the results is that the reference alternative B3+ that is developed by W+B is an easily adaptable design because in the design a berm is constructed off rubble, which can easily be added to widen the berm and/or decrease the lower slope. Therefore the main characteristics of the design of the reference alternative are good taking the new safety standard into account. Optimization of the dimensions of the reference alternative reduce construction costs of the renovation of the Afsluitdijk, as shown in the optimized alternative.

To answer the main research question the optimized alternative shown in Figure 2 is a cost-effective and adaptable design for the renovation of the Afsluitdijk. In the next years the flood safety standard for the Afsluitdijk is chosen and this design can easily be adapted to meet the future uncertain standard. The probabilistic design method used in this thesis is partly derived from the temporary flood defense design instrument OI2014 and it is shown that further investigation is needed before a final design can be recommended.

List of Acronyms

Abbreviation	Dutch	English
CPB	Centraal Planbureau	CPB Netherlands Bureau for Economic Policy Analysis
DP	Deltaprogramma	Delta Program
HR	hydraulische randvoorwaarden	hydraulic boundary conditions
IPO	Interprovinciaal overleg	Association of the Provinces of the Netherlands
KRW	Kaderrichtlijn Water	Water Framework Directive
MHW	Maatgevend hoogwater niveau	design water level
MIRT	Meerjarenprogramma Infrastructuur, Ruimte en Transport	
MKBA	Maatschappelijke kosten-baten analyse	social cost-benefit analysis
MLS	Meerlaags veiligheid	Multi-layered Safety
MSL		Mean Sea Level
NAP	Normaal Amsterdams Peil	
nHWBP	nieuw Hoogwaterbeschermingspro-gramma	Flood Protection Program
NWP	Nationaal Waterplan	National Water Plan
OI2014	Ontwerpinstrumentarium 2014	Design instruments for flood defences
ROR	Richtlijn Overstromingsrisico's	European Floods Directive
RWS	Rijkswaterstaat	-
SBW	Sterkte en Belasting Waterkeringen	Strenght and Loads Flood defences
SOD	Systeemontwerp Dijk	System design dike
STA	Structuurvisie Toekomst Afsluitdijk	National Structure Vision Afsluitdijk
TK	Tweede Kamer	House of Commons
UvW	Unie Van Waterschappen	Association of Regional Water Authori-ties
VNG	Vereniging van Nederlandse Gemeen-ten	Association of Netherlands Municipali-ties
VNK2	Veiligheidheid Nederland in Kaart	Flood Risk in the Netherlands
VTV	voorschrift toetsten op veiligheid	
WTI2017	Wettelijk Toetsinstrumentarium 2017	Legal assessment instruments 2017
WV21	Waterveiligheid 21e eeuw	21st-century Flood Protection
W+B	Witteveen + Bos	Witteveen + Bos

List of Figures

List of Tables

1 Introduction

At this moment the concept of the safety standards for flood defences in the Netherlands is changing into a risk-based approach. This new safety standard is based on a probability of flooding instead of a probability of exceedance of water levels. This way the effects of a flood are taken into account, which results in a more accurate risk-informed approach.

The implementation of the new safety standard introduces challenges because a fully risk-based approach is new and uncertainties and political discussions are introduced by this approach. In this MSc thesis these challenges are analyzed. Focus will be on the challenges during the transition period between switching from the current to the new standards. This transition period will especially introduce challenges for current projects regarding reinforcement of flood defences. The renovation in these projects is based on the current standard but the new standard has to be investigated. This is for example the case for the renovation of the Afsluitdijk.

In this MSc thesis the consequence of the new safety standards on the design of the Afsluitdijk will be researched. First the problems regarding the renovation of the Afsluitdijk in the transition period are analyzed. Once these problems are formulated, the main objective is set and a research question including sub research questions for this MSc thesis are given. The method of approach describes how this problem can be tackled and through which steps the research question can be answered.

1.1. Problem analysis

At this moment the safety standards in the Netherlands are based on a probability of exceedance of water levels. These standards were introduced in the first Delta Plan after the major flood in 1953. Over the past decades the Netherlands have developed into a country with more inhabitants and more economic value. Also due to climate change the river discharges increase and the sea level rises. Together with the subsidence of the land the problems are increasing in the future.

At the end of 2013 a new Delta Plan (DP2014) was proposed in which the safety standard will have the form of a probability of flooding. Nowadays the knowledge about the different failure mechanisms is sufficient enough to use this in models to determine the probabilities of flooding for each failure mechanism. Therefore the probability of flooding can be used instead of probability of exceedance of water levels. In this approach the probability of flooding and the impact of a flood are both taken into account, which leads to a risk-informed approach.

In DP2014 it is proposed that every inhabitant of the Netherlands has to have a guaranteed basic safety. This basic safety means that the probability of loss of life is 10^{-5} (1/100 000) per year for every individual. This guaranteed basic safety is relatively high compared to other causes of loss of life, as shown in Table 1.1. For certain areas with a large population or a high economic value a higher safety level may be profitable. With the implementation of this new safety standard the expected number of loss of life and the economic damage will reduce in the long term and the whole population of the Netherlands will have the same basic level of safety.

Cause	Probability of loss of life per year
Mountaineering accident	10^{-2}
Illness	10^{-3}
Car accident	10^{-4}
Plane accident	10^{-5}
Factory accident	10^{-6}

Table 1.1: Probabilities of decease for different causes (source: Lecture notes CT4130).

Before this new safety standard is implemented there are a number of ongoing projects that are based on the current safety standard. One of those projects is the renovation of the Afsluitdijk. The Afsluitdijk is a closure dam between the provinces of Noord-Holland and Friesland. To guarantee safety in the hinterland it was designed to protect Lake IJssel (which was named Zuiderzee before closure of the Afsluitdijk) from storms on the Wadden Sea. In 1927 the construction of the Afsluitdijk started and it was finished in 1932. A year later a road and the finishing details were done and the Afsluitdijk was opened for public.

Since the safety assessment in 2006 it is clear that the safety of the Afsluitdijk is not up to date and that it has to be renovated to guarantee the safety against floods. To guarantee the safety against flooding it was decided that the Afsluitdijk has to meet the same safety standard as the adjacent dike ring in North-Holland. This standard is set at a probability of exceedance of a water level of 1/10 000 per year. The renovation of the Afsluitdijk that is mentioned in the MIRT 3[1] is based on this standard but the question is how the new safety standard will influence the design of the Afsluitdijk. At this moment the renovation of the Afsluitdijk is investigated by Witteveen + Bos and a concept for the renovation of the Afsluitdijk is proposed. This renovation is based on a exceedance probability of the water level and not on the probability of flooding which is the future standard. Therefore the influence of the new standard on the renovation design is investigated.

1.2. Problem formulation

1.2.1. Introduction

Before the new safety standard is implemented there is a transition period for the design of flood defences. During this transition period the design of flood defences is not based on the new safety standard but it takes this new standard into account. This is done by anticipating on the most probable future standard and making a design with a (semi-)probabilistic method. One of the key questions for the new safety standard is how flood defences can be designed and evaluated on a probability of flooding. Most designers do not have full knowledge of probabilistic methods and therefore simpler rules are derived from the safety target (i.e. the probability of flooding) to determine the properties (dimensions) of the flood defences. These semi-probabilistic methods were developed and published at the end of 2013 in the OI2014[2].

The goal in this MSc research is to develop a design for the Afsluitdijk with a probabilistic approach taking the new risk approach into account. Before a design is developed the consequence of the new safety standard on the design of the Afsluitdijk is investigated by determining the probability of failure of the Afsluitdijk, taking residual strength into account. First of all the probabilistic methods that are used must be evaluated and information about the new safety standards must be gathered. With the probabilistic knowledge the probability of failure is determined. With the probabilistic methods and with information about the safety standard the renovation design of the Afsluitdijk can be optimized. The new probabilistic safety standard is less conservative than the current methods and probably results in lower costs for the renovation of the Afsluitdijk.

Currently a lot of investigation on the new safety standard is done and with a social cost-benefit analysis (MKBA) an economic optimization can be made to determine the probability of flooding for all the flood

[1]MIRT (Dutch acronym: Meerjarenplan Infrastructuur, Ruimte en Transport) is a program with plans for infrastructure and environment in the Netherlands for the coming years.
[2]Design instruments for flood defences (Ontwerpinstrumentarium 2014 (OI2014) in Dutch)

defences in the Netherlands. Thorough investigation on these probabilities was done by Deltares (Kind, 2011). For the Lake IJssel region the proposed probabilities of flooding depend on a lot of different factors. One of the main problems in the Lake IJssel region is that it is assumed that the Afsluitdijk will not fail. This means that the proposed probabilities of flooding may change if failure of the Afsluitdijk is taken into account.

1.2.2. Research goals

The proposed probabilities on the Lake IJssel region, including the Afsluitdijk, are further investigated by the CPB Netherlands Bureau for Economic Policy Analysis (CPB). In this analysis (Zwaneveld and Verweij, 2014) the Lake IJssel region is analyzed and an improved proposal of the probability of flooding for the Afsluitdijk is done. From this analysis a probability of failure of 1/9 400 per year is found. Still a lot of questions remain unanswered and after an assessment (van Ierland et al., 2014) it is clear that further research needs to be done before a proposed probability of failure for the Afsluitdijk can be chosen.

At this moment new plans are made for the renovation of the Afsluitdijk. In the design alternatives the new safety standards are not taken into account and the question rises how strong the new design has to be according to the new standard. The new standard probably results in a less conservative design, which costs less and it may even be the case that the Afsluitdijk does not need improvement at all. To determine the optimal design the assigned probability of failure for the Afsluitdijk must be known. As mentioned earlier the proposed probability of flooding depends on a lot of factors and it might change a lot depending on the choices that are made.

Another challenge is how the new safety standard is taken into account in the design of the Afsluitdijk. For the design based on a probability of flooding simplified approaches are developed. In this semi-probabilist approach safety factors are determined for different failure mechanisms that take these into account. However these semi-probabilistic methods are not developed for the Afsluitdijk. In this MSc thesis it is aimed to use fully probabilistic methods as much as possible. In case of insufficient knowledge or data a semi-probabilistic method will be used.

The start of the renovation of the Afsluitdijk is planned to start in 2017. By then the new safety standard for the Afsluitdijk is known so in the renovation design this new standard can be taken into account. In this thesis the goal is to anticipate on the new flood standard and propose a renovation design that can be adapted to the uncertain future standard. Also the probabilistic method and the determination of semi-probabilist factors in the design process is a goal in this thesis.

1.2.3. Research question and sub-questions

The main objective of this MSc research is to optimize the renovation design of the Afsluitdijk according to the new safety standard. From the problem formulation and the main objective a research question can be defined. The research question for this MSc thesis is formulated as follows:

"*What is a probabilistic, cost-effective and adaptable design for the renovation of the Afsluitdijk?*"

To reach the main objective the research question is divided into sub-questions:

1. Which failure mechanisms are of importance for the dike body of the Afsluitdijk?
2. What is the probability of failure of the existing dike body of the Afsluitdijk?
3. What flood safety standard (probability of failure) for the Afsluitdijk is most likely to be introduced with the new standard?
4. What is the probability of failure of the reference alternative that is developed at this moment?
5. How can the design of the reference alternative be optimized?
6. How can the uncertainty of the new standard be taken into account in the design of the Afsluitdijk?

1.3. Research methodology

This section describes the research method of this MSc thesis. In this MSc thesis qualitative research methods are used. At the beginning a literature study and interviews will provide the necessary information on the current safety standard and the different ideas of the new safety standard. Also this method will be used to gather information about probabilistic and semi-probabilistic methods for the design and safety assessments of flood defences.

After the general information is gathered focus is placed on the Afsluitdijk. The first step in this focus is to get acquainted with the development of the Afsluitdijk and the current plans for the Afsluitdijk (MIRT 3). Also the functioning of the Afsluitdijk and the interaction with the flood defences along the Lake IJssel region are described. This general information is kept in mind during further research to prevent to lose the bigger picture out of sight while going deeper into research.

After gathering this general information on the Afsluitdijk and probabilistic methods the next step is to calculate the current probability of flooding of the Afsluitdijk. Because the new safety standard will be based on the probability of flooding it is useful to know the current strength of the Afsluitdijk. This is useful because the current configuration of the Afsluitdijk may be strong enough. In this calculation a probabilistic method is used with the use of failure mechanisms budgets, that are developed in OI2014. A simple calculation on the residual strength of the boulder clay core is done to address the influence of this residual strength on the probability of flooding. Investigation on the residual strength is an ongoing process but in the design and assessment rules residual strength is still not incorporated. The influence of the residual strength is shown to emphasize that it has indeed a significant influence on the probability of failure of the Afsluitdijk.

Once the probability of failure is known the impact of this probability of flooding on the design will be investigated. However, it is unclear what the new standard for the Afsluitdijk will be. In a recent analysis of the Lake IJssel region (Zwaneveld and Verweij, 2014) a proposal is done for the probability of flooding. This derived probability of failure of 1/9 400 per year depends on uncertainties and this must be further investigated. In this MSc research the proposed standard will not be investigated. Because of the uncertain factors and the complex system of interaction of the flood defences along Lake IJssel and the Afsluitdijk the determination of the standard is outside the scope of this MSc research.

The future standard of the Afsluitdijk is unknown at this moment. However the probabilities of flooding for the reference alternative developed by W+B can be investigated. With a semi-probabilistic approach, which defines safety factors based on a probabilistic approach, the probability of failure will be determined. This semi-probabilistic approach is developed for the OI2014 and this approach is evaluated to see if it is applicable for the Afsluitdijk.

If necessary this semi-probabilistic approach is improved. The goal of this improvement is to derive failure mechanism budgets for the Afsluitdijk depending on the total probability of flooding. With this relation between these budgets and probability of flooding it is a lot easier to make a design if the standard (e.g. a certain probability of flooding) is chosen. Also an economic optimal design can be determined for different probabilities of flooding.

The final step of this MSc research is to compare all the results with each other and with the renovation design of the Afsluitdijk, according to the latest system design of the dike (Dutch acronym: SOD) by W+B. Different alternatives are developed by W+B and it is the question if it is profitable keeping the new safety standards in mind. After the comparison of these results conclusions are drawn on the current plans of the Afsluitdijk and recommendations are given for the future design.

2

Design and safety assessment of flood defences

2.1. New safety standard

The safety standard for flood defences in the Netherlands will be based on a risk approach. In DP2014 (Ministerie van IM and Ministerie van EZ, 2013) the concepts of the new standard are described. This new standard is aiming for three goals and these goals are reached through the concept of Multi-Layered-Safety (MLS).

The desired level of safety will be defined by the probability of flooding and the effects of a flood. Today the technical knowledge is available to understand these probabilistic calculations. Insights in what influences the strength of flood defences and what the impacts are of a flood for the hinterland are the base for a risk approach for the safety standard for flood defences. The new safety standard is based on three goals:

- Basic safety for everyone. Starting point is that for every Dutch civilian that is protected by any kind of flood defences the probability of loss of life due to a flood is less than 10^{-5} (1/100 000) per year. This basic safety is not taken into account in the current safety standard.
- Reduce social disruption due to a flood to a minimum. Social disruption appears when a large number of lives is lost in the area which is flooded and/or if large economic damage is the result of a flood.
- Prevent the fallout of vital infrastructure and vulnerable facilities like utilities and hospitals as much as possible. These facilities and infrastructure are during and after a flood of great importance for the functioning of the area of impact, the surrounding region or even the whole country.

All these goals are relevant and determine together the desired level of safety of the flood defences. After analysis of safety programs for specific areas these goals can be introduced in an advice for new safety standards. These goals can be reached by three kinds of measures:

- **Level 1:** New safety standards for the flood defences and preventive measures to reduce the probability of flooding.
- **Level 2:** Include the effects of a flood in the spatial planning and try to reduce those effects if possible. In some specific areas adjustments to the spatial planning can contribute to the desired level of safety.
- **Level 3:** Effective evacuation and disaster management after a flood to reduce the effects of a flood to a minimum.

The desired level of safety can be reached by a certain combination of the three levels. This is the concept of MLS. Improvement of the flood defences accordingly to level 1 will be the starting point in the new safety standard. However, if the improvement of the flood defences is too expensive or other reasons do not allow for improvements, a solution with a combination of level 2 and 3 may be the best option. The question is what the economic optimum is and if a combination with integration of level 2 and 3 is sufficient to guarantee the desired level of safety.

One of the main problems for this new safety standard is how these different goals are taken into account in the design of flood defences. For every dike trajectory and other structures like storm surge barriers the current probability of flooding can be estimated. On the other hand the spatial planning and evacuation fraction in the hinterland also have an influence on the design of the flood defences to reach the desired level of safety.

The spatial planning influences the effects of a flood by changing the way of a flood. But also in disaster management after a flood it is of importance what part of the vital facilities, networks and infrastructure is still functioning. This is influenced by the spatial planning and depends on where the flood starts.

The first goal in the new safety standards is that the probability of loss of life is 10^{-5} per year for every inhabitant that is protected by flood defences. This probability of loss of life is the basic safety. The part of the population that can be evacuated in an area that is flooded has influence on this basic safety. The problem is that the evacuation fraction is estimated but it depends on so much factors that this evacuation fraction is very uncertain. The amount of time between the start of the evacuation and a flood, at what time the evacuation starts (mid-day is different than during the night), the population, the weather conditions, the infrastructure, the evacuation plans, etc. This evacuation fraction can be estimated and can be checked with earlier floods but the uncertainty in estimating it for the future remains.

Another goal in the new safety standards is that social disruption due to a flood must be reduced to a minimum. This social disruption appears when a large number of lives is lost due to a flood or when the economic damage is very large. It is hard to implement such an un-quantified goal into a safety standard and it becomes a political question.

The economic damage can be estimated and with this it is possible to find an optimum between investment costs for the strengthening of flood defences and the costs of the total economic damage if a flood occurs. However, this may lead to huge investment costs and then the question rises if the government is willing and is able to invest in the strengthening of flood defences in certain areas. This can be calculated with a social cost-benefit analysis (Dutch acronym: MKBA (Maatschappelijke Kosten Baten Analyse)).

The total number of lives that is lost must also be taken into account. The evacuation fraction and population have an influence on the first goal of the standard; a basic safety level of 10^{-5} per year. With this basic safety the total number of loss of life in a certain area is taken into account but this may still lead to an undesirable situation. If for instance an area with a large population and a very low evacuation fraction is protected by a system of flood defences that has a probability of flooding of 10^{-5} per year the basic safety is guaranteed. However, if this area is flooded a large number of lives will be lost and this may be undesirable. So the question is how the population of an area has impact on the probability of flooding of the flood defences.

The impact on the environment due to a flood is not mentioned in the main goals for the new standard but this must also be taken into account in the design of flood defences. In a MKBA the investments costs for flood defences are compared to the benefits. An economic optimization results in the optimal level of safety. In this method the costs and benefits are expressed in monetary values. Just like the loss of life it is hard to express ecological damage into a monetary value. To take these intangible damage values into account in a MKBA is hard. It is a political discussion on how much the nature, landscape and ecological values are worth.

2.1.1. Water and flood management in the Netherlands

The largest water administration in the Netherlands is Rijkswaterstaat (RWS) which manages the general water system in the Netherlands[1]. For all the regional water systems smaller water boards are assigned to manage these waters. Besides these water boards also the Government and municipalities play a role in the water management in the Netherlands. As cross-border measures commitments are

[1]source: http://www.rijksoverheid.nl/onderwerpen/water-en-veiligeid/overheid-en-waterbeheer

made in the European Union. In the Water Act is described what the responsibilities off these different organizations are.

The Government of the Netherlands is responsible for the national policy and strategic goals for the water management in the Netherlands. Besides this the Government is responsible for the national measures and the safety standards for primary flood defences.

A Province[2] transforms the national goals to regional policy. Also a Province is responsible for the secondary flood defences. A province also issues permits for the extraction of ground water (if it is a large amount).

The municipalities do have some specific tasks in the water management. They are responsible for the ground water level in urban areas, the drainage of wastewater and excess rainwater.

The water administrations (Rijkswaterstaat and water boards) are responsible for the overall water management:

- The prevention of floods.
- Taking care of sufficient ground and (fresh) surface water.
- Taking care of the water quality.

Rijkswaterstaat is responsible for the management of the main water system in the Netherlands. The main water system consists of the large water systems like the sea and rivers. Also RWS is responsible for the warning system of storm surges on the North Sea. RWS is part of the Ministry of Infrastructure and the Environment.

The water boards are responsible for the regional water systems. These are the smaller water systems. There are in total 24 water boards in the Netherlands that cooperate in a the Association of Regional Water Authorities (Dutch acronym: UvW (Unie van Waterschappen)).

In 2007 the European Floods Directive (Dutch acronym: ROR (Richtlijn Overstromingsrisico's)) was introduced. The main goal of this guideline is to reduce the negative consequences of a flood. The guideline mandates the European members to gather information, communicate (inter)nationally and make plans for the national and cross-border flood management. The obligations in the ROR are taken into account in the Water Act.

Since 2010 the responsible organizations work together on the implementation of ROR. Maps are developed which show flood risks. These maps are useful for the public and (local) management to gain insight on the source and the size of the risks. The maps also are a base for the goals and measures in the flood risk management.

2.1.2. Flood management programs

The goal is to legally implement the new safety standard for flood defences in the Netherlands in 2017. At this moment a lot of programs are dealing with the flood management system in the Netherlands and these programs give insight on the future of flood management. Many different parties are involved in these programs and to prevent un-clarity an overview of the different programs is given in this section. In Figure 2.1 an overview of the organization of flood management in the Netherlands including the different programs is given.

Delta Program (DP) 2014 - subprogram water safety

The DP is a national program that is led by a governmental commissioner (the Delta commissioner). Within this program the Government, Provinces, Municipalities and Water Boards cooperate with social organizations, companies and research institutes. The main goal is to protect the Netherlands against floods and provide sufficient fresh water taking future scenarios into account.

After the major flood in 1953 the safety standards for the flood defences were defined. Over the past decades the population of the Netherlands and the economic value have grown significantly. Also

[2] The Netherlands is divided in twelve Provinces. Cooperation between the Provinces is in the form of the Association of the Provinces of the Netherlands (Dutch acronym: IPO (Interprovinciaal Overleg)).

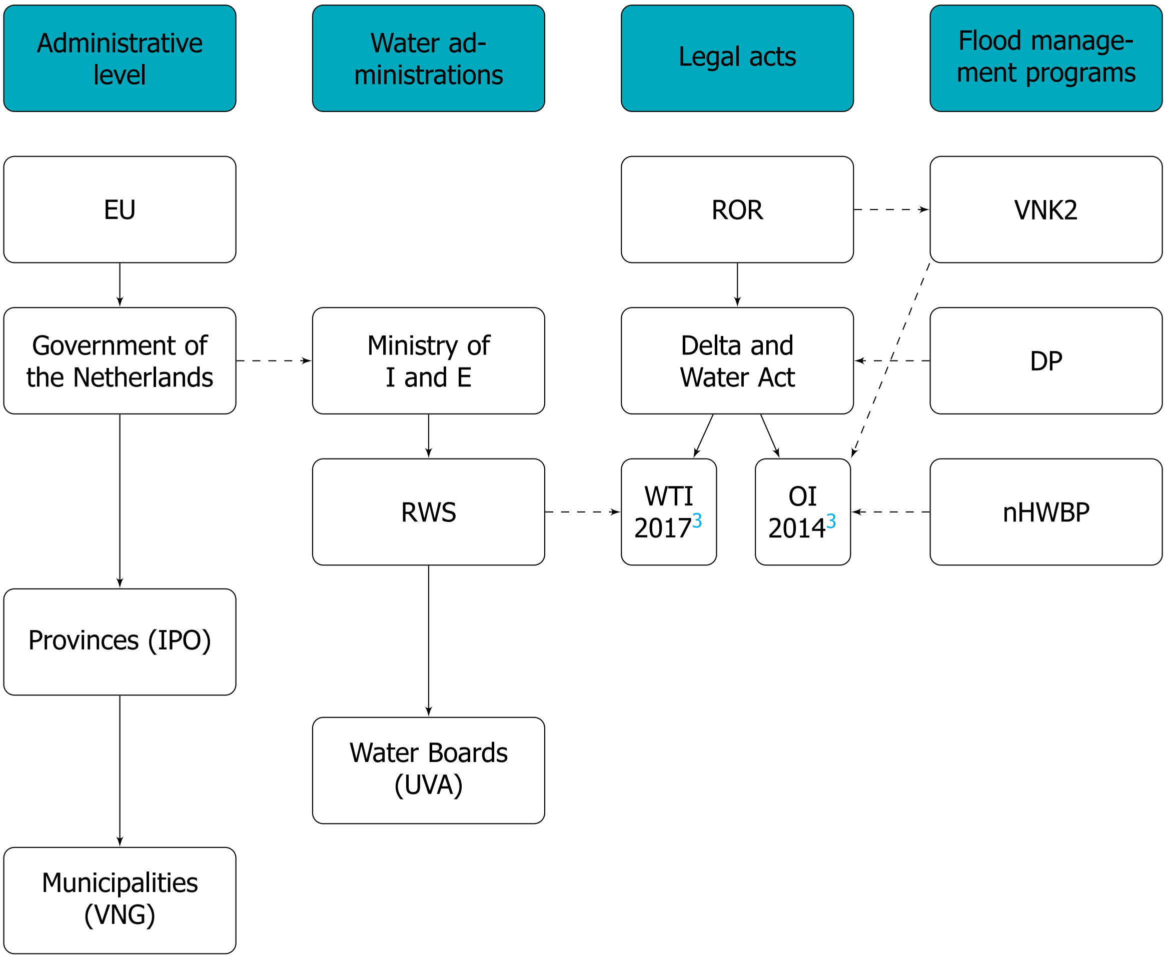

Figure 2.1: Overview of the water management in the Netherlands, including water administrations and the flood management programs and legal acts.

the sea level is rising and the ground-level is subsiding. This taken into account together with more extreme weather conditions in the future was the inducement to develop a new DP. In 2010 the initial phase of the program started and the goal was to have a full and implementable program ready before 2015.

The DP consists of three national sub-programs and six regional sub-programs. The three national sub-programs are:

- Safety
- Fresh water
- Spatial planning

The sub-program safety deals with the question how to protect the Netherlands against floods on a social acceptable risk level. The sub-program spatial planning is initiated to see what the influences are of spatial planning on the effects of a flooding, heavy rainfall and drought. The sub-program fresh water takes a closer look at the current policy of fresh water management to find (future) problems and provide solutions. Because this MSc research is focussed on flood management the fresh water program of the DP will not further be mentioned, details of the fresh water program and the regional sub-programs can be found in DP 2014 (Ministerie van IM and Ministerie van EZ, 2013).

In 2012 a new Delta Act was introduced in which the purpose of the DP is legally fixed. Besides the DP the Delta Act also describes the role of the Delta commissioner and the financials of the DP in a Delta

[3] Legal design and safety assessment instruments for flood defences, see Section 2.2 for further explanation.

fund. Also this Water Act describes that the Delta commissioner is obliged to present the DP every year on Prinsjesdag[4], together with the budgets for the associated ministry.

In the DP2015 a proposal for five delta decisions is done. Delta decisions are the main choices that have to be made for the water safety and fresh water supply in the Netherlands. One of those delta decisions is about the flood safety. The current thoughts about the water safety result in a new approach for the safety standard for flood defences. The main idea is to transform the basic approach of the safety standard from a probability of exceedance of water levels to a probability of flooding. Also the flood risk, including the effects of a flooding, will be taken into account. The currently proposed safety standard will be based on three earlier mentioned goals:

- Basic safety for everyone.
- Reduce social disruption due to a flood to a minimum.
- Prevent the fallout of vital infrastructure and vulnerable facilities like utilities and hospitals as much as possible.

All these goals are all relevant and determine together the desired level of safety of the flood defences. After analysis of safety programs for specific areas these goals can be introduced in an advice for new safety standards. These goals can be reached by the earlier mentioned MLS concept. In this MLS concept these goals are reached by improvements on three levels that include improvement of flood defences, water safety in spatial planning and investment in disaster management.

With this new approach the expected number of mortality and economical damage will reduce in the future. The proposal for this approach is made in DP2015 and will be implemented in the follow up of the National Water Plan (NWP) (first design in December 2014). In the final version of the NWP in 2015 the delta decisions are pinned down in the policy of the water safety in the Netherlands. This procedure of implementation of the policy in the law starts in 2015 and the goal is that the new safety standard for flood defences is legally bounded in 2017. This also means that in 2017 the methods for design and safety assessment of flood defences must be ready.

This risk-based approach will be put into practice before it is legally bounded in 2017. This is done in order of the House of Commons (Dutch acronym: TK (Tweede Kamer)) and will be put into practice in the large flood defense reconstruction program (Dutch acronym: nHWBP (nieuwe Hoogwaterbeschermingsprogramma)). This implementation is done in three phases and after 2017 dike trajectories that do not comply with the new safety standard are automatically part of the nHWBP. In the nHWBP a prioritization is made to see on where improvements of dike trajectories are most necessary. The goal is that in 2050 all the flood defences in the Netherlands comply with the new safety standard.

Flood defense reconstruction program (nHWBP)

The water boards and the Infrastructure and the Environment execute the measures to make sure the flood defences are in compliance with the new safety standard. These measures are part of the new flood defense reconstruction program (nHWBP). The nHWBP will be actualize every year and will be composed for a period of six years, starting in 2014. The cooperation of the water boards and the government is based on agreements that are fixed in the Administrative Agreement Water of May 2011. The most important agreements is that the water boards and the government are both responsible for the flood defences and that both parties contribute 50% to the investment costs.

The design of the nHWBP is new in comparison with previous programs. The nHWBP focusses on a closer cooperation between the water boards and the Ministry of Infrastructure and the Environment, an ongoing program (actualized every year) and new starting points. More time is invested in the preparation of the program and projects, the new safety standard for flood defences is implemented and innovation and knowledge sharing is stimulated.

Since 1996 safety assessments of the flood defences are done in the Netherlands. Since 1996 every five years an assessment was done to gain actual insights on the status the flood defences. These assessments are the basis for decision-making on flood management. In 2011 a third assessment was done to see if the flood defences were in compliance with the safety standards. The results of this third assessment are the starting point of the nHWBP.

[4]Prinsjesdag is the opening-day of the Dutch parliament.

After these results the water boards had the chance to indicate which part of the flood defences needed improvement. The different trajectories are combined in several projects. The water boards had to indicate which dike trajectories and/or structures are part of the different projects and which failure mechanisms have to be tackled. With a prioritization a choice is made for the order of execution of the different projects. This prioritization is based on the probability of flooding and the effects of a flood. To determine the economical damage of a flooding the flood calculations of the project Flood Risk in the Netherlands (Dutch acronym: VNK2 (Veiligheid Nederland in Kaart)) and the research 21st-century Flood Protection are used. With this a list of the projects is produced, ranked on order of urgency.

With the available budget for the coming five years (2014-2019) the ranking is re-arranged. The most urgent projects are planned to be executed one by one. The execution of these projects is done in a desired timeframe, determined by the Water Boards, of two years for three phases that are distinguished in the execution of a project. These phases are analyzing, plan development and realization. The first projects on the list are ranked in such a way that as many projects as possible can start in the first six years. The result of these described steps is in essence the nHWBP.

Flood Risk in the Netherlands (Dutch acronym: VNK2)

To gain insight on the flood risks in the Netherlands the project Flood Risk in the Netherlands (VNK2) started in 2006. VNK2 is an initiative of the Ministry of Infrastructure and the Environment, the UvA and the IPO. The execution of this project is done by Rijkswaterstaat in cooperation with research institutions and engineering companies.

The flood defences in the Netherlands are separated in 58 dike rings. A dike ring is a consecutive stretch of a dike which protects a bounded area. A dike ring consists of different dike sections and hydraulic structures but these sections and structures have to protect the same hinterland, therefore they are combined in the same dike ring.

In the first phase a system assessment is done based on the risk analysis of three dike rings. After this a start is made on calculations of results for all the dike rings. The first published report in 2012 included the results for 27 dike rings and in 2014 the analysis on the flood risk for all the dike rings will be done.

The flood risk is determined by the probability of flooding and the consequences of a flooding. In this risk-based approach the failure mechanisms and their corresponding probability of failure are taken into account to calculate their influence on the total probability of flooding and flood risk. By distinguishing the possible failure mechanisms insight is gained on possible strength improvements and the reduction of the flood risk of those improvements.

The results of VNK are a solid base for the prioritization of projects in the nHWBP. Also the results of VKN2 can be used to define measures which are optimized in terms of safety, given the investment costs in MLS.

2.2. Design and safety assessment instruments for flood defences

The current safety assessment method in the Netherlands is clearly described and is a legal standard. With the new safety standard the safety assessment method also has to change. In this section the current method is described followed by the possible future safety assessment methods.

2.2.1. Current method

The safety assessment of flood defences in the Netherlands is unique because it is prescribed by law[5]. The things that are prescribed are the safety assessment itself, the safety level, the hydraulic boundary conditions and the assessment method.

The safety assessment of flood defences in the Netherlands is described by the following steps:

[5] Water Act, Articles 2.12, 2.2, 2.3 and 2.6

- The flood defense manager assesses the safety and reports for the provincial (regional) authority per dike ring area or per connecting flood defense.
- The provincial authority reports to the minister per province.
- The minister reports to the national parliament.

To assess the safety of a flood defense information and methods are needed:

- A legel standard
  - Criteria per type of flood defense
  - Criteria per failure mode
- Data of the flood defense
  - Dimensions, geometry and composition, properties, etc.
- (Hydraulic) boundary conditions
  - Water level, waves
  - Other loads
- Safety assessment method
  - Per type of flood defense
  - Per failure mode

2.2.2. Future method

According to the Water Act the flood defences must be assessed every six years. This assessment will probably evolve into a continuous process[6]. For this safety assessment legal instruments are developed that are used by the water administrations. This instrument is developed by RWS and Deltares in order of the Ministry of Infrastructure and the Environment. This development is based on knowledge, insights from research and experience from previous assessments.

For the assessment of the safety of flood defences all the water administration must use the legal assessment instruments (Dutch acronym: WTI (Wettelijk Toetsinstrumentarium). The WTI consists of:

- Hydraulic boundary conditions (Dutch acronym: HR (Hydraulische Randvoorwaarden)): The water levels, wave heights, currents, etc. that administrations must use in the assessments.
- Instructions on safety assessments (Dutch acronym: VTV (Voorschrift toetsen op veiligheid)): The methods and calculation methods that administrations must use in the assessments.
- Supportive (calculation) software for the safety assessments.

Because the new safety standards are based on the probability of flooding instead of probability of exceedance of water levels the WTI must change as well. The planning is to introduce the new safety standards in 2017. Therefore the WTI2017 must be developed. Insights from the VNK2 and a research program on the Strength and Loads Flood Defences (Dutch acronym: SBW (Sterkte en Belastingen Waterkeringen)) must help the development of WTI2017.

In the Flood Protection Program (nHWBP) the WTI2017 must be taken into account to guarantee that design are future proof. For this transition period from the current safety standard to WTI2017 a design instrument (Dutch acronym: OI2014 (Ontwerpinstrumentarium)) is developed, based on results from VNK2. The OI2014 is developed in such a way that it follows the current design instruments as much as possible.

2.3. Failure mechanisms of flood defences

To calculate the probability of flooding all possibilities of failure must be taken into account. In this paragraph a general introduction of the failure mechanisms of flood defences is given. After this brief introduction the most important mechanisms for the Afsluitdijk are described in more detail.

Flood defences are categorized in four categories:

[6]Discussion about the frequency of assessments is still going on at this moment.

1. Dunes.
2. Soil structures (dams, dikes).
3. Special water retaining structures (cofferdam, retaining wall, sheet piling).
4. Water retaining hydraulic structures (locks, barriers, pumping stations).

The Afsluitdijk is a large dam with navigation locks and outlet sluices to let the surplus of water out of the Lake IJssel. The name Afsluitdijk suggests it is dike but there is no direct hinterland protected by the Afsluitdijk. The main purpose of the Afsluitdijk is to retain the storm surge from the Wadden Sea to prevent high water levels and wave loads on the IJssel Lake. Flood defences like the Afsluitdijk that do not protect direct hinterland but retain outside water are called b-type flood defences. Because this MSc research is focussed on the Afsluitdijk and not on flood defences in general only categories 2 and 4 will be described in this Paragraph. For more information on dunes and special water retaining structures see (Weijers and Tonneijck, 2009).

2.3.1. Soil structures

In contrary to dunes soil structures like dikes and dams are manmade. These soil structures are typically build of a combination of sand and clay with a certain type of cover. Because unlike dunes the slopes are designed to prevent erosion, different hydraulic conditions demand different types of covers. For river dikes this is mostly grass, for lake and sea dikes and/or dams mostly a revetment of stone or asphalt is used. A typical cross section of a dike is trapezium-shaped with an optional berm on both sides of the main structure. The main reason why these soil structures are used a lot is because the soil material is cheap and widely available and construction is rather easy.

The main function of soil structures is the retention of water. The structural design defines the water-retaining capacity (strength) of the dike. Stability of the soil body and the crest height are the two factors that determine this strength. This stability depends on the geometry of the dike body and the materials that are used. A general overview of a dike is given in Figure 2.2.

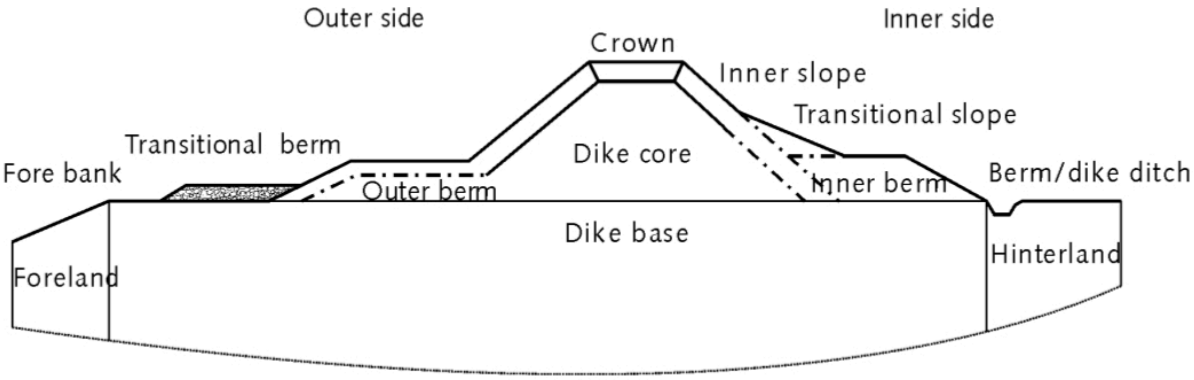

Figure 2.2: Standard cross section of a dike without special structures.

The main load on dikes is caused by water. As the water level increases the hydraulic head[7] over the dike increases which can cause instability. Also with rising water levels waves can overtop the crest and if the water rises even further the water can flow over the crest. Waves attack the outer slope of a dike and the can overtop the crest, which can cause erosion on the inner slope. The different failure mechanisms are shown in Figure 2.3 and an explanation is given for the different types of the failure mechanisms.

(A) Overflow: The water level on the outer side reaches a higher level then the crown of the dike. The water can flow over the dike into the hinterland. Also the flow of water will erode the inner slope of the dike which results in instability and finally into breaching of the dike.

(B) Wave overtopping: The water level is lower than the crown of the dike but the waves can cause water to flow over the dike. This phenomenon is called overtopping. If this overtopping becomes to large this results in the same as failure due to overflow (erosion inner slope, instability, breach).

[7] The hydraulic head is defined as the difference in water levels between the inner and outer side of the dike.

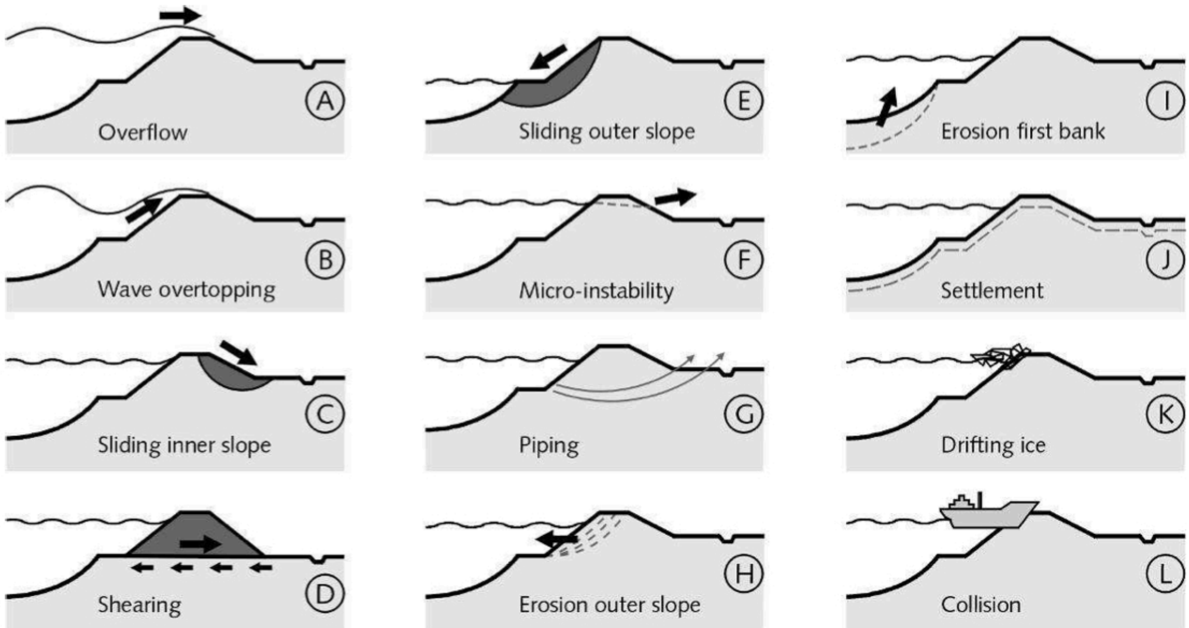

Figure 2.3: Failure mechanisms of dikes.

(C) Sliding inner slope: During a period of high water the inner slope can slide and this leads to instability. The water retaining function of the dike may be intact but if overtopping or overflow starts a breach is inevitable.

(D) Shearing: This mechanism is not often observed. If the friction in a layer of the dike is not sufficient it can slide horizontally. This failure occurred in 2003 in Wilnis (the Netherlands), where the dike was constructed of peat.

(E) Sliding outer slope: If the water level on the outer side of the dike drops rapidly the water inside the dike is not able to follow. The pressure inside the dike becomes to high which results in a sliding of the outer slope. Because this only occurs when water levels are dropping this failure mechanism does not immediately result in flooding. However if the water levels rise again before the dike is repaired this may be a threat.

(F) Micro-instability: Seepage of water that reaches the inner slope of a dike can transport small particles, this is why it is called micro-instability. The seepage transports the particles out of the inner slope and in this way the dike erodes from the inside. The damage is usually minor but if the pressure builds up inside the dike this may lead to an unstable situation.

(G) Piping: Piping is a similar process as micro-instability but it occurs underneath a cohesive layer in the subsoil. The seepage water erodes particles from underneath the dike if the hydraulic head is large enough. The erosion forms a pipe which grows from the inner to the outer side and this can lead to a settlement and failure of the dike. This mechanism can only occur underneath a cohesive layer (clay) on top of an erodible and permeable soil (sand).

(H) Erosion outer slope: Wave attack can cause erosion of the outer slope. This erosion can result in collapse of a dike. To protect the dike against wave attack it is protected with a stone or asphalt revetment. If the wave attack is not too big a grass cover on a clay layer is often used.

(I) Erosion first bank: This failure mechanism can occur if there is a steep underwater slope in front of the dike. If the sand in the subsoil is loose packed it can transform into a denser state which results in high water pressures. The sand becomes a liquid and starts to flow. This shifted soil can start the instability of the outer slope of a dike.

(J) Settlement: Settlement is a long and time consuming process. In comparison with a collapse the deformation due to settlement is very slow and takes years/decades. However in a lifetime a dike can settle a lot and therefore this settlement must be taken into account in the design.

(K) Drifting ice: Drifting ice caused around half of the total number of breaches of river dikes in the

Netherlands. Because the heat of the surface water of the rivers increased this has not been a threat for over a century. This does not mean that ice will never be a threat again, but it is not taken into account in the current safety standards.

(L) Collision: The collision of a vessel into a dike can cause severe damage and this can result in flooding. However during high water on the rivers shipping is forbidden and during a storm surge on sea it is also not likely that ships will sail. Therefore this failure mechanism is not taken into account in the safety standard.

2.3.2. Water retaining hydraulic structures

Water retaining hydraulic structures can be part of a flood defense. The most common hydraulic structures that have a water-retaining function are locks, outlet sluices, storm surge and tidal flood barriers, weirs and pumping stations. For these hydraulic structures the failure mechanisms are different than for soil structures. The main failure mechanisms of these hydraulic structures are:

1. Constructive failure
2. Strength and stability of the foundation and subsoil.
3. Strength and stability of transitional structures. Piping erosion.
4. Failure to close gates (in time). sluices, locks, barriers.
5. The state of maintenance.
6. Overtopping and overflow.

2.3.3. Failure mechanisms Afsluitdijk

The Afsluitdijk can be divided in different sections. These sections mainly consist of dike bodies, navigation locks and outlet sluices. Therefore the failure mechanisms of the Afsluitdijk can be separated in two categories:

- Failure of a dike body.
- Failure of a hydraulic structure.

The most important failure mechanisms in the failure of a dike body are:

- Wave overtopping/overflow.
- Erosion outer slope.
- Macro instability.

When a breach develops in the dike body of the Afsluitdijk it has some residual strength, especially because the dike body is build on and next to a boulder clay core. This residual strength is not taken into account in the assessment of the probability of failure of the Afsluitdijk but the influence of this residual strength is evaluated separately from this assessment.

Besides a failure of the dike body the hydraulic structures can fail. The most important failure mechanisms of the hydraulic structures in the Afsluitdijk are:

- Constructive failure.
- Failure to close gates (in time).
- Geotechnical instability.

2.4. Risk-based approach

After the major flood in 1953 the Dutch Government already made a first flood risk inventory. This was done by the first Delta Committee who defined probabilities of exceedance of water levels for different dike rings, based on this risk inventory. For instance the design water level for Central Holland, which has the most economic value, was determined at a probability of exceedance of 1/10 000 per year. Nowadays our knowledge on risk management and probabilistic design is developed even further and it can be used in a new safety standard with a full risk-based approach, including a probability of flooding. In this chapter the concept of the risk-based approach is explained and how it is used in the current safety standard and how it will be used in the future standard.

2.4.1. Risk, probability and consequences

There are a lot of different definitions of risk. In engineering risk is defined as:

$$risk = P_f * consequence \tag{2.1}$$

In which P_f is defined as the probability of failure. This definition of risk is used to be able to quantify risk so it can be used in design and decision-making. The probability of an undesired event or failure (in this case flooding) is generally expressed as a frequency (1/year). This undesired event can lead to unwanted consequences like damage to objects or loss of life. The consequences can be defined as a value. The different values that are used in flood risk management in the Netherlands are explained in subparagraph 2.4.2.

In the flood risk management two types of probabilities are distinguished. The first probability that is used is the probability of exceedance of a certain water level. In the current safety standard this probability is used as a starting point for the design of flood defences. With safety factors for different failure mechanisms a design is made based on this design water level. The second probability that is used is the probability of flooding/failure of flood defences. In the new safety standard this probability of flooding will be used. With the use of the probability of flooding the design is not based on one normative water level, that is assumed to cause failure, but on the continues water load.

In the current safety standard the assumption is that the design water level causes failure. The typical failure mechanisms that depend on this probability of exceedance of water level are overtopping and overflow, causing erosion of the inner slope and finally failure of the flood defense. History has shown that other failure mechanisms play a big role in the probability of flooding as well, like piping and micro- and macro-stability of the slopes (geotechnical failure mechanisms). In the new safety standard the probability of flooding is based on all occurring water levels. The less extreme water levels will not lead to overtopping or overflow but may cause failure due to these geotechnical failure mechanisms. Because these less extreme water levels occur more often the probability of the geotechnical failure mechanisms becomes more significant in the design and safety assessment of flood defences.

2.4.2. Risk dimensions

In flood risk analysis we define three different types of risk that are all taken into account in the safety standard. The dimensions of those risks are expressed in a certain type of value. This can be the total number of casualties or a monetary value. The three dimensions that are used in flood risk management in the Netherlands are:

- Individual risk
- Group/societal risk
- Economic risk

Individual risk

A commonly used indicator for the individual risk is the location bounded risk (Dutch acronym: PR (Plaatsgebonden Risico)). This indicator is defined as the probability of loss of life due to a flooding for a person that would be located at one place for a whole year. In this indicator evacuation and self-reliance is not taken into account. Because in many cases a flooding can be predicted and preventive evacuation can reduce the risk of loss of life this indicator is not used in the Netherlands. Instead of this indicator the local individual risk (LIR) is used. In the LIR a correction for the effect of preventive evacuation is taken into account with respect to the PR.

Group risk

As in the safety standard one of the main goals is to prevent social disruption the group risk is taken into account. A flooding that causes a lot of fatalities has more impact than a lot of small incidents and the social disruption is therefore larger. To prevent this social disruption the group risk is considered. This group risk means the risk that a significant amount of people loose their lives due to a flooding. A commonly used indicator for the group risk is a FN curve, shown in Figure 2.4. A FN curve shows the probability of N or more fatalities. This FN curve can be made for specific dike rings or it can be used on a bigger scale, for instance the whole population of the Netherlands. In this FN curve a certain boundary can be defined that may not be exceeded. To reach this goal measures can be taken that either reduce the probability of flooding or the consequence of flooding.

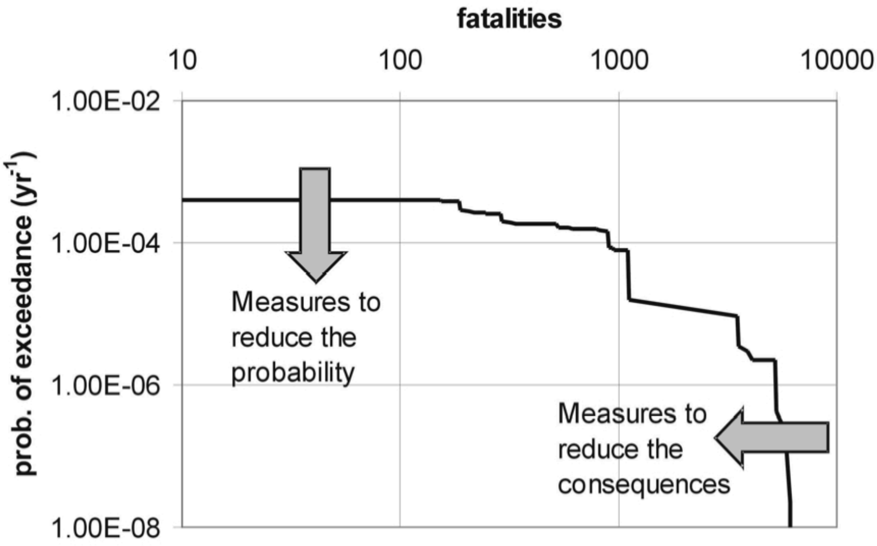

Figure 2.4: Example of a FN curve.

Economic risk

The previous risk dimensions are combined the casualty risk. Besides this casualty risk an economic optimization can be made with a social cost-benefit analysis (MKBA). A MKBA focusses on the costs and benefits of flood protection measures, in which the economic efficiency of investments in flood defences is the point of interest. In a MKBA the investments on improvement of flood defences (costs) are compared to the reduced consequences of a flooding (benefits). In a MKBA the reduced consequence of flooding due to improvement on disaster management (evacuation plans, early warning systems) is not taken into account.

The economic risk in this MKBA is used to determine the benefits. For a certain area the economic damage can be determined if a flooding occurs. If this consequence, a monetary value, is multiplied by the probability of flooding the economic risk is defined. So an improvement on a flood defences reduces the probability of flooding which reduces the total economic risk. Because an economic risk is expressed in a monetary value the intangible values are transformed to monetary values. The intangible values that are taken into account are the casualties (and related damage) and damage to landscape, culture and nature. However this is a complex problem and in a recent study Kind (2011) on the economic optimized safety levels for the Netherlands an extra added 50 % on the tangible damage is used to take the intangible values into account.

In the First Delta Committee already a model was used to define the economic optimum for the design of flood defences. This model was introduced by Prof. van Dantzig and it is shown in Figure 2.5. In this figure the investment costs (I) and the consequences of a flooding (C), expressed in monetary value, are added to get the total costs. At the minimum of this total costs the economic optimum is found. From this economic optimum the accepted failure probability and the cost for investments can be determined.

However in this MSc research the economic optimum is not investigated but an adaptable design for the renovation of the Afsluitdijk is optimized in terms of costs. In this optimization different values for some dike characteristics are varied and a alternative with minimum costs and above a certain

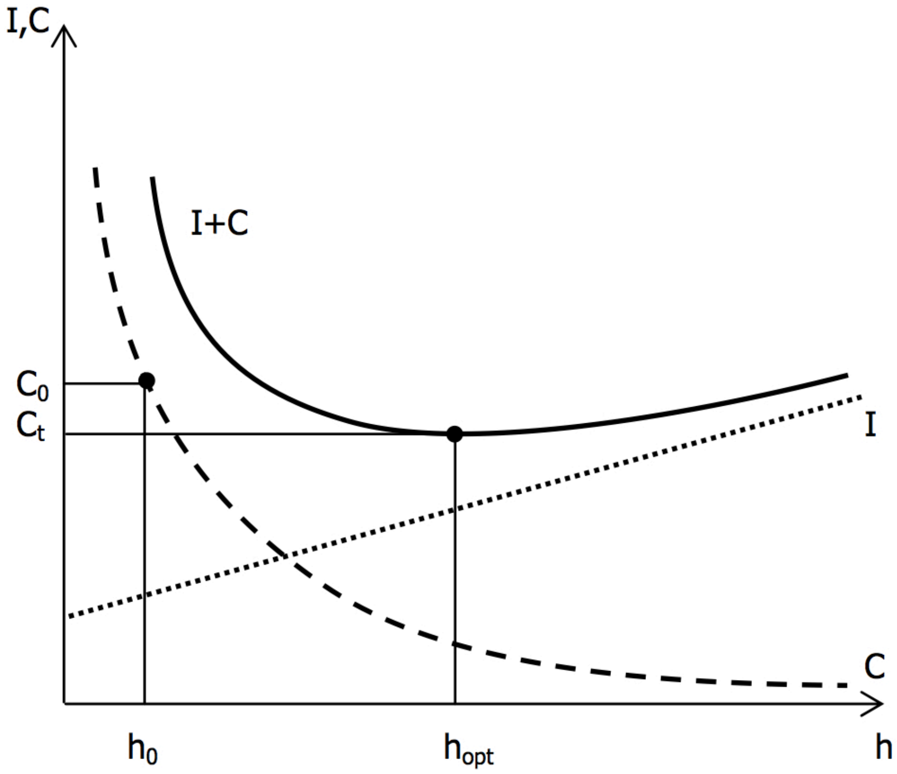

Figure 2.5: Costs as a function of the accepted probability of failure by Prof. van Dantzig.

safety standard is recommend. To find the economic optimum is outside the scope of this MSc thesis because of insufficient time and knowledge. A recent study (Zwaneveld and Verweij, 2014) on the economic optimum results in a safety level for the Afsluitdijk with a probability of failure of 1/9 400 per year.

3
(Semi-)probabilistic models

In this chapter a brief introduction on probabilistic and semi-probabilistic methods is given. In more detail the semi-probabilistic method which is used as a design instrument for the coming years is described in Appendix D. This design instrument is developed at the moment and will be used as a legal flood design instrument in the Netherlands for the coming years. Besides this the fully probabilistic model PC-Ring, which is used in the VNK-project, is described. PC-Ring is a software module that can assess the probability of flooding of a dike ring with a fully probabilistic method. PC-Ring is also used in this MSc research to assess the current probability of failure of the Afsluitdijk and to optimize the design of the reference alternative. Finally the derivation of the hydraulic loads is described. These hydraulic loads are defined as stochastic values and are used as an input in the model that is developed during this MSc research.

3.1. Semi-probabilistic method (OI2014)

In 2017 the safety assessments and design procedures for flood defences will be described legally by the WTI2017. In the Flood Protection Program (nHWBP) the WTI2017 must be taken into account to guarantee that designs are future proof. For this transition period from the current safety standard to WTI2017 the OI2014 is developed, based on results from VNK2. The OI2014 is developed in such a way that it follows the current design instruments as much as possible.

From a given standard (probability of flooding) for each dike trajectory a probability of failure for each cross section and each failure mechanism is determined. With this probability of failure the hydraulic loads and the safety factors are derived which are both part of the OI2014. With these hydraulic loads and the safety factors a flood defense can be designed with the current guides and technical reports.

One of the core ideas in the OI2014 is that the design instrument is practical, useful and applicable for all the flood defences. It aims to prevent flood defences being disapproved during the first safety assessment with the new safety standard. In practice this means that designs made according to OI2014 are conservative, but the chances of disapproval during the first safety assessment are small.

In the design method OI2014 the probability of failure standard is a given. With the results of VNK2 probability budgets are assigned to every failure mechanism and these budgets are used in WTI2017. This probability budget is also used in OI2014 to derive the failure requirements for every mechanism on cross-section level. The result are shown in Table D.1.

In this MSc research it is necessary to develop a semi-probabilistic approach for the design and safety assessment of the Afsluitdijk. Fully probabilistic methods are preferable but to take the correlation and length-effect into account a semi-probabilistic method with failure mechanism budgets is needed. Therefore the probability failure budgets that are chosen in OI2014 can be compared with the budgets and the corresponding partial safety factors that are derived in this MSc research.

Table 3.1: Probability of failure budget (maximum allowable probabilities of failure as percentages of the safety standard).

Type of flood defence	Failure mechanism	Type of trajectory	
		Sandy coast	**Remainder (dikes)**
Dike	Overflow and overtopping	0%	24%
	Piping	0%	24%
	Macro-stability inner slope	0%	4%
	Damage and erosion	0%	10%
Hydraulic structure	Non-closure	0%	4%
	Piping	0%	2%
	Constructive failure	0%	2%
Dune	Dune erosion	70%	0% / 10% [a]
Remainder		30%	30% / 20%
Total		**100%**	**100%**

[a] For trajectories that partly consist of dunes, the dune erosion contributes a relatively small percentage of failure. Proposed is to shift 10% from the remainder to dunes. This prevents that in these situations a completely new probability budget has to be used.

3.2. Probabilistic methods

3.2.1. General probabilistic methods

The state just before failure occurs is called the limit state. The probability of failure can be expressed as the exceedance probability of this limit state. To describe the probability of failure in a probabilistic way a limit state functions is used which is formulated as follows:

$$Z = R - S \tag{3.1}$$

in which R is the strength (R of resistance) and S is the load (S of solicitation). The limit state is described by $Z = 0$ so failure occurs if Z is negative. The probability of failure P_f is described by the following formula:

$$P_f = P(Z \leq 0) = P(S \geq R) \tag{3.2}$$

The reliability is when Z is positive, so the reliability function is the inverse of P_f and is given by:

$$P(Z \geq 0) = 1 - P_f \tag{3.3}$$

There are different calculation methods to determine the total probability of failure. These calculations are separated in three different categories:

- level III: P_f is calculated by considering the probability density functions of all strength and load variables.
- level II: In this level a number of methods for determining the probability of failure can be used. The general procedure is linearization of the reliability function in a carefully selected point. These methods approximate the probability distribution of each variable by a standard normal distribution.
- level I: At this level P_f is not calculated. The calculation of this level is a design method according to standards which uses representative values for the strength and the load. A safety margin is created by the use of partial safety factors.

Without getting into the details of the different methods for all the levels some examples are given:

- level III
  - Analytically
  - Numerical integration
  - Monte Carlo simulation
- level II
  - FORM

    - SORM
    - AFDA
- level I
    - Safety factors and characteristic values

In this MSc research the aim is to use fully probabilistic methods, which is level III. In this MSc research the crude Monte Carlo simulation is used in the development of a Matlab model which assesses the probability of failure of the Afsluitdijk. In this method the possibility of drawing random numbers form a uniform probably density function between zero and one is used. The non-exceedance probability of an arbitrary random variable is uniformly distributed between zero and one, regardless of the distribution of the variable:

$$F_X(X) = X_u \tag{3.4}$$

in which X_U is the uniformly distributed variable between zero and one and $F_X(X)$ is the non-exceedance probability $P(X < X)$. So for the variable X:

$$X = F_X^{-1}(X_u) \tag{3.5}$$

In which $F_X^{-1}(X_u)$ is the inverse of the probability distribution function of X. A random number X can be generated from an arbitrary distribution $F_X(X)$ by drawing a number X_u from the uniform distribution between zero and one.

With this method for every variable a random value can be drawn from their corresponding distribution functions, leading to a vector which is used as input in the limit state function. The resulting value of the vector is calculated and if this limit state function is smaller than zero failure occurs. By repeating this procedure a large number of times the probability of failure can be estimated as follows:

$$P_f \approx \frac{n_f}{n} \tag{3.6}$$

in which:

n_f	is the number of simulation for which the limit state function is smaller than zero
n	is the total number of simulations

To obtain a result with a reliability of 95% the total number of simulations is:

$$n > 400(\frac{1}{P_f} - 1) \tag{3.7}$$

3.2.2. PC-Ring

In the VNK-method a software module is used to asses the probabilities of flooding of the flood defences in the Netherlands. This software module is PC-Ring which calculates the probability of flooding of dike rings with a probabilistic method. PC-Ring is used in this MSc research to determine the total probability of failure of the Afsluitdijk and partly to optimize a new design for the Afsluitdijk.

PC-Ring is owned by the Ministry of Infrastructure and the Environment and it is developed by Deltares. A complete description of the software is given in a user manual, a theoretical manual, a programming manual and a system documentation. For the use in this MSc research only the user and theoretical manual are of interest. The user manual (van der Wouden and Grashoff, 2009) describes the calculation possibilities, the method of schematization of a dike ring, the control of the program and the composition of the input and output files. The theoretical manual consists of three parts:

- A: Mechanism description (Steenbergen et al., 2007). For all the mechanisms that are programmed in PC-Ring a description and the corresponding formulas are given.
- B: Statistical models (Steenbergen and Vrouwenvelder, 2003b). Description of the modeling of stochastic values used in PC-Ring in general. Besides this also recommended values for dike parameters are given and the derivation of the hydraulic boundary conditions and models are described.
- C: Calculation methods (Steenbergen and Vrouwenvelder, 2003a). Description of calculation methods used in PC-Ring.

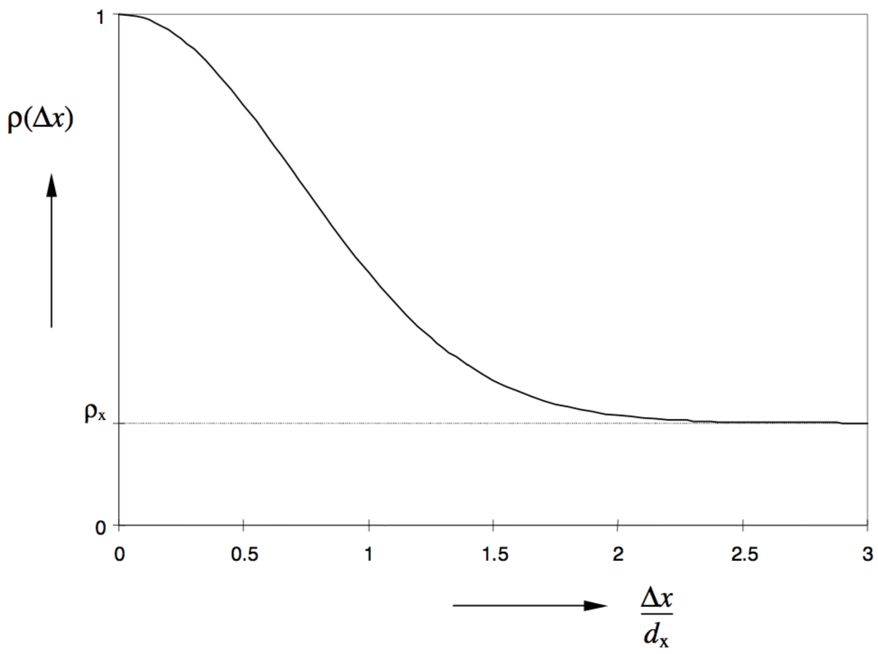

Figure 3.1: The spatial correlation starting at a value of 1 and decreasing to the constant correlation for an increasing distance.

In PC-Ring the flood defense can consist of dikes, dunes and/or hydraulic structures. The failure mechanisms described in paragraph 2.3 can be calculated with PC-Ring. The main method which is used in PC-Ring follows these steps:

- The flood defense is around the dike ring area is divided into sections. In PC-Ring a section is a hydraulic structure, a dike stretch or a dune stretch. For a section a schematization of a cross section is defined for the relevant failure mechanisms.
- The contribution to the probability of flooding of the sections is determined with a probabilistic failure probability analysis for the representative cross section. The probability of failure of a cross section is translated into a probability of failure for the section taking correlation and the length-effect into account.
- In the end the total probability of flooding for a dike ring is calculated with the failure mechanism budgets for each section and combining all the sections taking correlation and the length-effect into account.

In PC-Ring the correlation inside a section consists of spatial and time correlation. The spatial correlation is defined with a constant correlation ρ_x a correlation distance d_x and a correlation distance for the spatial spread Δ_x. This spatial correlation is given by the following formula:

$$\rho(\Delta x) = \rho_x + (1 - \rho_x) exp \left\{ -\frac{\Delta x^2}{d_x^2} \right\} \tag{3.8}$$

The function starts at a correlation of 1 and converges to the constant correlation with an increasing distance (see Figure 3.1).

For the time correlation the processes in the time domain are discrete in time intervals Δ_t according to a Borger-Castanheta-model. Inside these time intervals Δ_t there is complete correlation ($\rho = 1$) and in between the time intervals there is a constant correlation ρ_t.

The stochastic parameters in PC-Ring consist of the following values:

- distribution type
- mean value (μ)
- standard deviation (σ) or variation coefficient (V)

- correlation function for spatial spread (d_x and ρ_x)
- correlation function for time spread (Δ_t and ρ_t)

With these stochastic parameters the probability of failure for each mechanism and each section is calculated, taking the length of the section into account. To calculate the total failure probability of a dike ring the probability of failure for all the mechanisms and for each cross section is combined. Failure occurs if one section fails due to one of the failure mechanisms. To calculate the total probability a method for the combination of a random serial system is used.

If a random serial system with n elements is considered the total probability of failure of the system is given by:

$$P_f = P(Z_1 < 0 \cup Z_2 < 0 \cup ...Z_n < 0) \tag{3.9}$$

For instance this formulation can be used to calculate n dike sections for the failure mechanism over-topping. In this formula Z_i is the limit state function of section i and the event $Z_i < 0$ corresponds with the failure of section i. To do this calculation a probabilistic analysis for each section i is done resulting in a reliability index β and influence coefficients $\alpha_{i,k}$ for every basic variable k in section i.

The next step is to combine this to a total probability of failure. The upper limit for this total probability of failure is given by the sum of the probabilities of all the sections n. The lower limit is given by the section i with the maximum probability of failure. The combined probability of failure is in between these bounds:

$$P_f < \sum_{i=1}^{n} P\{Z_i < 0\} \tag{3.10}$$

$$P_f > max\, P\{Z_i < 0\} \tag{3.11}$$

Because the difference between these limits is usually too large an approximation method is used to determine the total probability of failure. The parameters of importance in the calculation of the total probability of failure are the correlation coefficients of the lint state functions Z_i. A correlation coefficient is defined for two stochastic values and it expresses the combined spread of the variables. If the correlation coefficient between the variables Z_i equals zero there is total in-dependancy and the upper limit equals the total probability of failure. For an increasing correlation coefficient the total probability of failure will reduce until the lower limit is reached (with a fully correlated system where the correlation coefficient equals one).

The correlation coefficient of two functions Z_i and Z_j is calculated with the influence coefficients $\alpha_{i,k}$ and the correlation of the stochastic variable x_k in element i with the corresponding stochastic variable x_k in element j:

$$\rho(Z_i Z_j) = \sum \alpha_{i,k} \alpha_{j,k} \rho_{i,j,k} \tag{3.12}$$

With the known reliability indexes β_i and β_j and with the correlation coefficient the combined probability of failure $P(Z_i < 0 \cup Z_j < 0)$ is calculated. An equivalent reliability index and equivalent influence factors are determined (for a full description see (Steenbergen and Vrouwenvelder, 2003a). The probability $P(Z_i < 0 \cup Z_j < 0)$ is replaced by an equivalent probability of $P(Z_e < 0)$ with corresponding β^e and α_k^e. In Figure 3.2 a global overview of this method is shown.

The total number of sections is now reduced to $(n-1)$. By repeating this procedure $(n-1)$ all the sections are combined resulting into the total probability of failure. As mentioned earlier this method is based on equivalent influence factors. Because these are not calculated exactly this method is always an approximation. The accuracy of this method is influenced by the order in which the different sections are combined.

The most accurate solution is found if for every step all the correlation coefficients are calculated and that the two sections with the highest correlation coefficients are combined. Take for instance a system which consists of four elements (Figure 3.3) with limit state functions Z_1, Z_2, Z_3 and Z_4. If Z_1 and Z_2 have the strongest correlation these are first combined to Z_2^e. Then the correlations between Z_2^e, Z_3 and Z_4 are calculated. If Z_3 and Z_4 have the strongest correlation these are combined to Z_4^e. Finally the equivalent elements are combined to get to the total probability of failure.

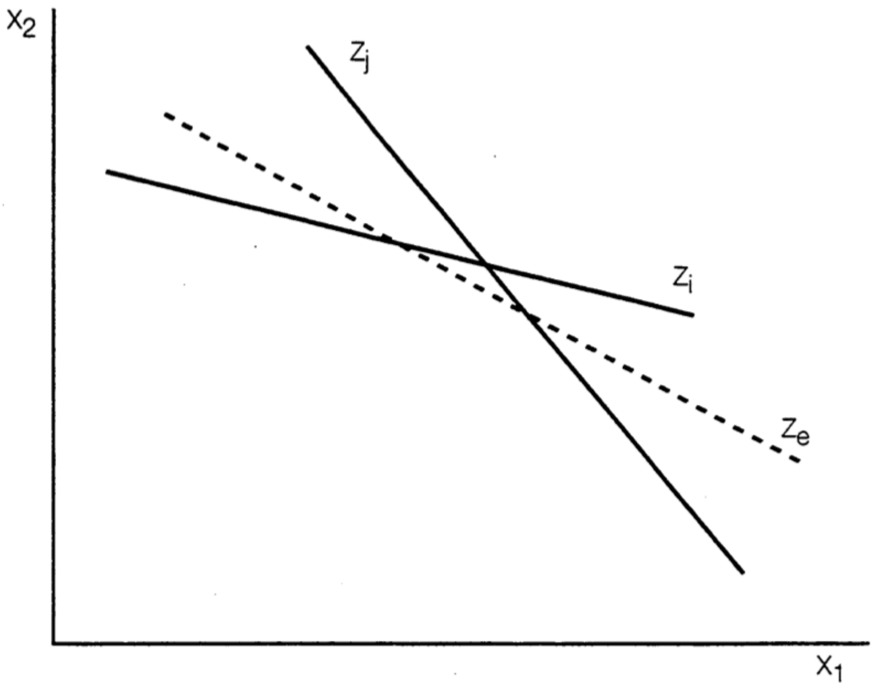

Figure 3.2: Equivalent limit state function Z_e of the combined probability of Z_i and Z_j.

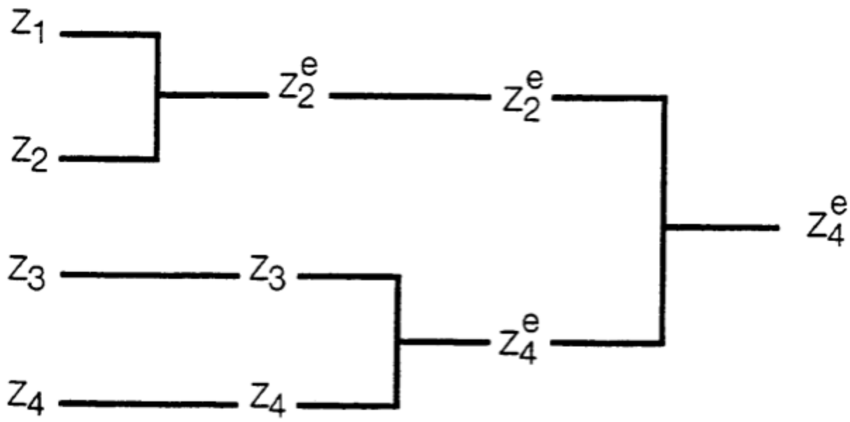

Figure 3.3: Combination of probability of failure for a system with four elements.

For a dike ring we have n sections and each sections has i failure mechanisms. To combine this to a total probability of failure there are two possibilities:

- Combine all the mechanisms i per section. After this combine all the sections n to a total probability of failure.
- Combine all the sections n per failure mechanisms. After this combine all the failure mechanisms i to a total probability of failure.

After the first step in the second method there are limit state functions for each mechanism with different basic variables and influence coefficients. The correlation between these different basic variables is not defined and therefore this method is not suitable. In the first method after the first step the basic variables of the different limit state functions are fully correlated so the problem with the second method does not occur. Therefore the first method is used in PC-Ring.

3.3. Hydraulic loads

In PC-Ring the hydraulic loads are derived from a database (with measurements offshore) and used in the calculations. However the hydraulic boundary conditions have to be derived for the model developed in this MSc research, because the hydraulic boundary conditions can not be extracted from PC-Ring directly. For the previous assessment of the Afsluitdijk in 2011 the hydraulic boundary conditions were derived with Hydra-K. Hydra-K is also used in this Msc research to derive the hydraulic boundary conditions. A full description of the derivation of the hydraulic boundary conditions is given in Appendix A.

Hydra-K is a software module which can be used to assess the primary flood defences along the Dutch coast. For the failure mechanisms wave run-up, wave overtopping and instability of the revetments the current flood defences can be assessed. Hydra-K can also be used to generate water levels and wave conditions for the hydraulic boundary conditions. Also Hydra-K is used to derive the hydraulic boundary conditions (HR2011) for the legal assessment of the flood defences in the Western Scheldt and the Wadden Sea.

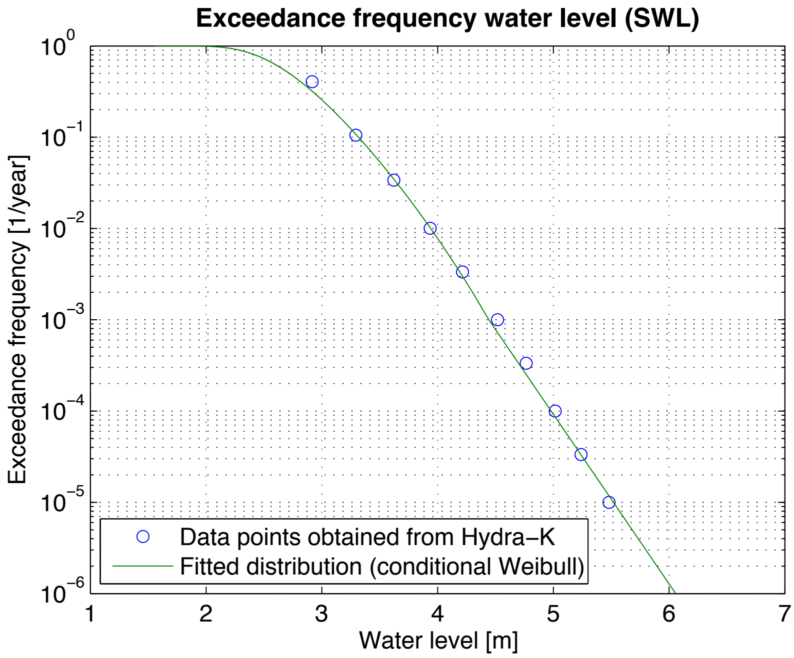

Figure 3.4: Fitted conditional Weibull distribution according to the exceedance frequency values given by Hydra-K.

To determine the hydraulic boundary conditions for the different dike sections of the Afsluitdijk Hydra-K is used. With Hydra-K for different design exceedance frequencies the corresponding variables are calculated. This is done at the different locations that represent the dike sections of the Afsluitdijk. Trough these points a distribution is fitted which is used as an input in the model used in this MSc research. For the water level a conditional Weibull distribution is used which results in the best fit with the data points from Hydra-K. In Figure 3.4 the data points obtained from Hydra-K and the conditional Weibull fit are shown.

Because the extreme water levels have a high correlation with the significant wave height a relationship between the water level and the significant wave height is used in the model:

$$H_s = C_1 log(SWL) + C_2 \tag{3.13}$$

In these extreme conditions the mean energy wave period can be derived from the significant wave height. For this relationship the following formula is used:

$$T_{m-1,0} = \sqrt{\frac{H_s}{C_3}} \tag{3.14}$$

3.4. Method for the assessment of the Afsluitdijk

The failure of the Afsluitdijk can be caused by many different things. Different dike sections and hydraulic structures and multiple failure mechanisms must be taken into account. Besides this the correlation and the length effect must be taken into account. A general procedure to obtain the probability of failure for the Afsluitdijk is shown in Figure 3.5.

This flowchart (Figure 3.5) is split up in two different parts. To get acquainted with the different failure mechanisms a model is developed to get insights in the different failure mechanisms (see Appendix B). With the information from this model the failure mechanisms that are most relevant are selected and these are evaluated in PC-Ring, which is described in the next paragraph. In PC-Ring the length effect and correlation of the different failure mechanisms and cross section is taken into account, resulting into a total probability of failure.

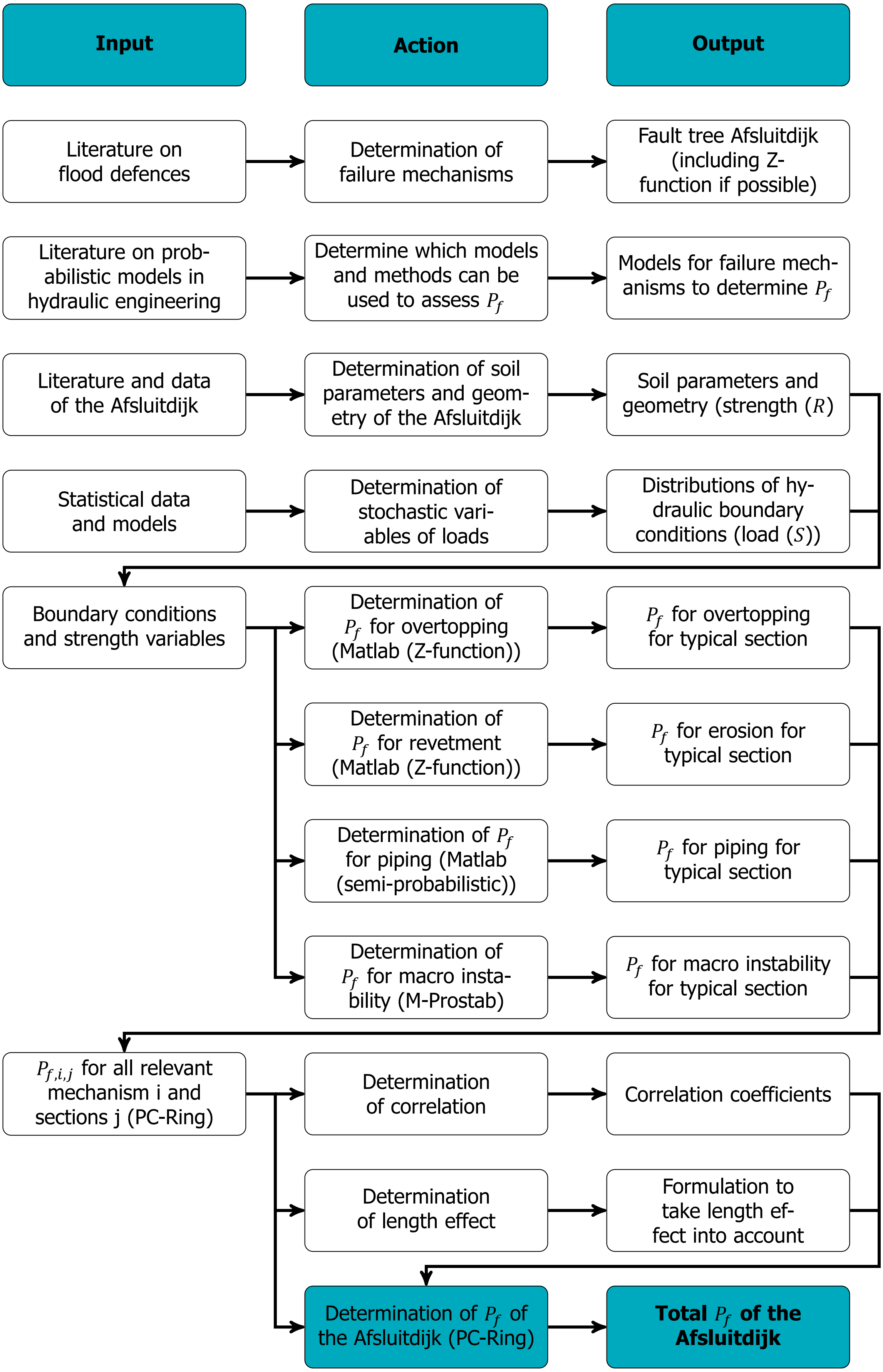

Figure 3.5: Flow chart model to assess the probability of failure of the Afsluitdijk.

4

Current design of the Afsluitdijk

4.1. General introduction to the Afsluitdijk

4.1.1. History of the Afsluitdijk

In 1891 hydraulic engineer Cornelis Lely presented the first feasible plans for the closure of the Zuiderzee and reclamation of land (Flevoland). As Minister of Waterstaat Lely succeeded in 1913 to implement his plans into the governmental program. In 1918 his plans were approved by the Parliament. He succeeded partly due to the food crisis during World War I which showed the importance of land reclamation for agricultural activities. However the biggest influence was the flood in 1916 that effected the whole area around the Zuiderzee.

In 1927 the first beginning of the Afsluitdijk was constructed. The design was at that time still based on the plans Lely made 1891, in which he proposed to include locks and outlet sluices in the Afsluitdijk. The final decision to build two sluice complexes, at Den Oever en Kornwerdezand, was based on a military point of view. Only one complex would increase the vulnerability during an attack.

Another change in his plans was the connection at the coast of Friesland. A commission, with Lorentz as chairman, calculated that the tidal range would increase significantly if the connection was located at Piaarn (Commissie Lorentz, 1926). This increasing tidal range would also cause larger flow velocities that would complicate the construction. Therefore the location of the connection was shifted North to Kornwerderzand.

Also the bend in the Afsluitdijk was not in the original plans. At the location of the bend at Kornwerderzand a deep gully was present. Due to this bend the Afsluitdijk is perpendicular to this gully. The expectation was that this would enhance the sluicing of water out of the IJssel Lake. Also less material was required to construct the Afsluitdijk through the gully.

The Afsluitdijk was constructed from four different locations: at both the shorelines and two construction islands Breezand and Kornwerderzand. The final closure of the Zuiderzee was in 1932 and at this location a monument designed by Dudok and a statue of a stoneworker are placed. In 1933 the Afsluitdijk was officially opened for traffic.

4.1.2. Current state and future of the Afsluitdijk

Since the opening of the Afsluitdijk it has been modified at certain locations. Some changes have been made to revetments en the height of the dike was increased at some locations. Also the section between the shoreline of Friesland and Kornwerderzand has been widened. The biggest change is the road that is converted (in the 1970's) from a two-lane road to a four-lane highway and that the original plan to add a railway has been abandoned. At this moment the Afsluitdijk still is an icon of the Dutch hydraulic engineering and it is seen as a unique landscape appearance that is famous for the spatial quality.

The main function of the Afsluitdijk is retaining water. The Afsluitdijk is a primary flood defense that protects against high water from the Wadden Sea to provide safety in the IJssel region. Besides this main function the Afsluitdijk has several other functions:

- Water management. The Afsluitdijk separates the fresh water in Lake IJssel from the salt water of the Wadden Sea. The large fresh water basin of Lake IJssel is important for the water management, drinking water supply, agriculture, industry and the refreshment of the bordering lakes. Also the outlet sluices play a role in the water management because the regulate the water level in the IJssel Lake.
- Mobility. The highway A7 is a connection for road traffic. The part of the A7 on the Afsluitdijk connects the Provinces of North-Holland and Friesland. Every day around 20.000 vehicles make us of the highway on the Afsluitdijk. Also a separate cycling trail is located on the Afsluitdijk. Navigation locks are located at Den Oever and Kornwerderzand to provide the crossing for both recreational and professional vessels.
- Housing. 15 houses are located on the island of Kornwerderzand and one in Breezanddijk.
- Recreation. The Afsluitdijk attracts local and foreign tourists and one of the reasons for that is that it is part of a large hiking and cycling trail. Besides this there is small campsite located at Breezanddijk and at Kornwerderzand a museum attracts tourists. The Monument, including a bridge for pedestrians over the A7, is a popular stopping area.
- Defences. There is a shooting range located on Breezanddijk. Also military flying routes cross the Afsluitdijk. Between Breezanddijk and Den Oever a radar distortion area over the IJssel Lake is issued. In this area it is not allowed to build constructions that distort radar signals because of their size.
- Small business. Besides the camping there are some fishing and surfing facilities located on the Afsluitdijk. Other small businesses on the Afsluitdijk are the restaurant near the Monument and a gas station in Breezanddijk.

4.1.3. New standard for the Afsluitdijk

Flood safety Lake IJssel region

Uncertainty in the current proposed probability of flooding

The proposed probability of breaching of 1/9 400 per year depends on a lot of factors. In the research by the CPB (Zwaneveld and Verweij, 2014) this probability of flooding is proposed but it is also mentioned that a lot of factors influence this proposed design standard. A few other problems that also came up are described in an assessment (van Ierland et al., 2014). From both these reports it is clear that the future standard remains uncertain and investigation is still needed to decrease the uncertainty in the new standard for the Afsluitdijk. In this subparagraph a few of those factors are mentioned to give an idea about the uncertainty in this proposed probability of flooding.

One of the main influencing factors on the proposed probability of flooding is the choice of the design water level of the Lake IJssel. In the analysis it is assumed that the water level is kept at the current design level in the future by pumping water out into the Wadden Sea. Normally the surplus of water in Lake IJssel is sluiced out by the gradient in water levels but due to the rising sea level this becomes a problem in the future. The design is based on pumps with a capacity of 2 000 m^3/s to guarantee the design water level in the Lake IJssel. This capacity could also be lowered and this can influence the probability of flooding of the Afsluitdijk.

In the current design according to the STA a pump is being installed with a much lower capacity than this proposed 2 000 m^3/s. If research shows that this leads to a significant difference in safety compared to the 2 000 m^3/s the question rises how this problem must be dealt with. There are three options:

- Reinforce dikes earlier.
- Reinforce dikes according to the pumping scenario with 2 000 m^3/s and accept a temporary lower safety level.
- Increase the pumping capacity on short notice so a comparable safety is guaranteed.

If the design water level is rising with the sea level the water can be sluiced out Lake IJssel by the gradient in water levels. This results in higher water levels in Lake IJssel and so the probability of flooding for flood defences along the Lake IJssel increases. As a result these dikes will be stronger in comparison to the needed strength with the current design water level. So in this case failure of the Afsluitdijk will have a smaller impact on the flood defences along Lake IJssel because they are stronger. If rising with the sea level is chosen as a future design water level the probability of flooding of the Afsluitdijk can be higher (i.e. less strength so a weaker and cheaper design).

The failure of the functioning of structures is not taken into account. The probability of failure of pumps and sluices may have an influence on the choice for managing the water level in the Lake IJssel. This choice has a large influence on the design standard for the Afsluitdijk. More attention has to be paid on the risks of pumping and sluicing and on measurements to reduce these risks.

The method that is used to determine probabilities of failure for every dike ring has to be investigated. The CPB analysis is based on probability of flooding of one or more representative locations for every dike ring, in which the probabilities of flooding for these locations are completely dependent or independent of each other. The question is how realistic this is. Research has to be done to see if the results depends on the number of assessment locations and if the length of dike rings and trajectories have to be taken into account.

Another question is how useful it is to work with the standards according to the method described in the CPB analysis. The proposed standards strongly depend on multiple aspects:

- The choice for the year 2050 as assessment year.
- The question to what extent these standards meet the standards according to the law.
- The mutual dependencies that say that the used probability of flooding, under the assumption that the Afsluitdijk will not fail, is just a part of the total probability of flooding.
- The difference between this used probability of flooding, under the assumption that the Afsluitdijk will not fail, and the total probability of flooding is hard to explain.

With this kept in mind it is advised to do extra research on the proposed safety standard. Focus must be on the frequency and method of assessment, the ability of legal implementation of the standard, the communicability of the standard, the choice between the total probability of failure and probability under assumption that the Afsluitdijk will not fail and if one target year is enough.

Also extra attention has to be paid to the possible danger for the city of Amsterdam due to a flood. If the Markermeer dikes fail this could lead to increased danger for the city of Amsterdam and this has to be investigated.

Another method that could reduce the impact of a flood is disaster management. In this case it might be useful to have spare material ready to temporarily repair a breach in the Afsluitdijk. This may result in added safety and must therefore be investigated.

In the CPB analysis it is assumed that the Afsluitdijk currently has a safety level of 1/250 per year. If it turns out that (certain parts of) the Afsluitdijk are stronger it might be profitable to not implement the design that is made according to the STA.

4.2. Current design of the Afsluitdijk

4.2.1. Configuration of the Afsluitdijk

In Figure 4.1 an overview of the Afsluitdijk is shown. The main part of the Afsluitdijk consists of a standard dike body. In the West the Afsluitdijk starts at Den Oever. At Den Oever the Stevinsluices are located, which consists of outlet sluices and navigation locks. From West to East we cross the Monument, the working island Breezanddijk (designed as a construction island for the Afsluitdijk) and at the end Kornwerderzand with the Lorentzsluices (both outlet sluices and navigation locks).

The Afsluitdijk can be separated into dike sections. According to (Witteveen + Bos, 2013) the Afsluitdijk is divided into 17 sections. Because some of those sections experience a substantial different hydraulic loads these are split up into two sections (A and B). An overview of the dike sections is given in Table 4.1.

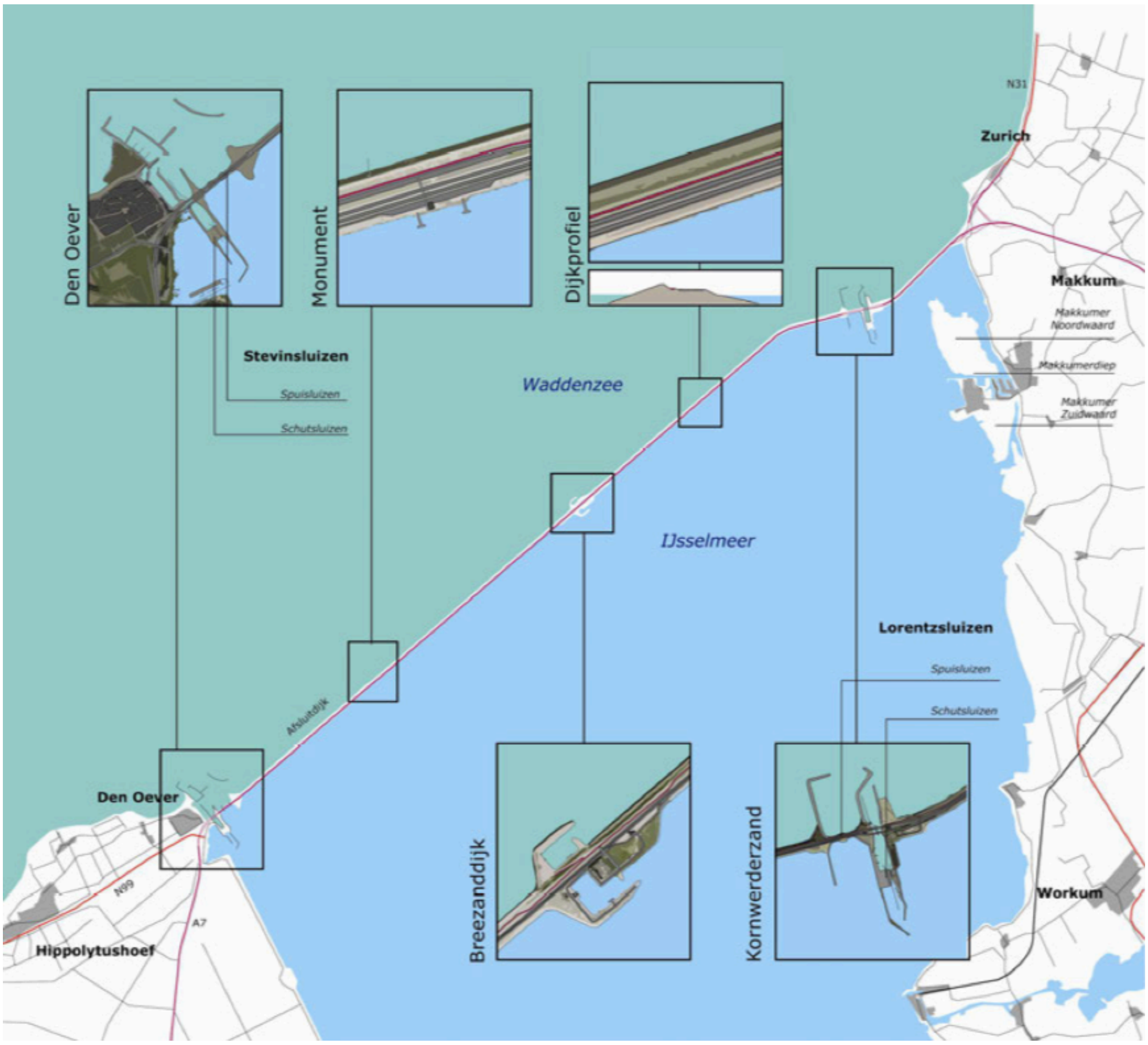

Figure 4.1: An overview of the Afsluitdijk with the dike body and special structures (Ministerie van IM, 2011).

4.2.2. Standard cross section dike

The core of the dike body consists of boulder clay, that was dredged close to the construction site of the Afsluitdijk. On top and on the side of this boulder clay core sand is placed. The dike body is covered with rumble, a stone revetment and a clay cover with grass. This cover provides protection against erosion and provides stability. A standard cross section is shown in Figure 4.2.

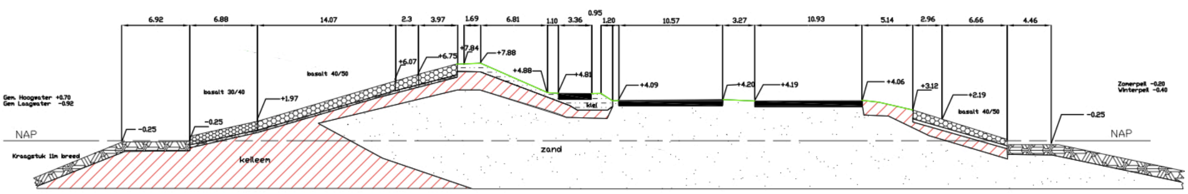

Figure 4.2: A typical cross section of the Afsluitdijk.

dike section	from [km]	to [km]	length [km]	description
1	0.3	0.91	0.61	outer port Den Oever (West)
2	0.91	1.47	0.56	outer port Den Oever (East)
3	1.47	1.6	0.13	between bridge and outles sluices Den Oever
4	1.6	2.07	0.47	outlet sluices Den Oever
5	2.07	2.5	0.43	connection dike and outlet sluices Den Over
6A	2.5	4.4	1.9	Afsluitdijk
6B	4.4	6.9	2.5	Afsluitdijk
7	6.9	7.6	0.7	Monument
8A	7.6	11	3.4	Afsluitdijk
8B	11	15.05	4.05	Afsluitdijk
9	15.05	17.53	2.48	Working port Breezanddijk
10A	17.53	19.5	1.97	Afsluitdijk
10B	19.5	21.03	1.53	Afsluitdijk
11A	21.03	23.9	2.87	Afsluitdijk
11B	23.9	25.9	2	Afsluitdijk
12	25.9	26.22	0.32	Kornwerderzand (West)
13	26.22	26.49	0.27	Kornwerderzand
14	26.49	26.7	0.21	outlet sluices Kornwerderzand (East)
15	26.7	27.3	0.6	outer port Kornwerderzand (West)
16	27.3	27.9	0.6	outer port Kornwerderzand (East)
17	27.9	31.92	4.02	section Friesland Kornwerderzand

Table 4.1: Dike sections of the Afsluitdijk including locations for the determination of the hydraulic boundary conditions.

The shape of the dike body is almost the same along the Afsluitdijk. In general the top of the Afsluitdijk has a height of +7.75m NAP[1]. The cross-section of the Afsluitdijk can be separated in 13 different zones:

1. Protection zone Wadden Sea. The exact dimensions of all the flood defences is described by a water administration. The most recent definition of the Afsluitdijk is from November 2009. In this definition a protection zone of 150 m is chosen that is the necessary length to provide geotechnical stability of the Afsluitdijk. This is including possible future reinforcements of the dike body.
2. Core zone. The whole dike body itself is located in the core zone.
3. Outer toe. The outer toe is the transition from the sea bed to the edge of the dike body. This is also the border of the core zone.
4. Rubble-mound berm. This berm is horizontal plate around 0m NAP with a rubble-mound cover. This berm is the transition from the underwater slope to the higher stone revetment of the outer slope. The rubble-mound berm protects the toe against forces from the waves and currents.
5. Outer slope. The outer slope is defined as the area between the rubble-mound berm and the crown of the dike. The outer slope of the Afsluitdijk is mainly covered with a placed basalt revetment.
6. Crown. The crown is the top of the dike body and is on average 2 meters wide. It is covered wit clay and grass.
7. Inner slope upper dike. The inner slope of the upper dike is also covered with clay and grass.
8. Cycling path. At the bottom of the inner slope of the upper dike a cycling path of asphalt is located.
9. A7. The A7 consists of two asphalt roads with each two traffic and one emergency lane. A small inner wayside is constructed of clay with grass.
10. Inner slope. The inner slope on the IJssel Lake side is steep. The top part is covered with clay

[1]Standard water level that is used in the Netherlands (Normaal Amsterdams Peil in Dutch), approximately equal to Mean Sea Level (MSL)

and grass and the bottom part mainly consists of placed basalt stones.

11. Rubble-mound berm. Also on the IJssel lake side a horizontal plate is constructed to protect the inner toe against waves and currents.
12. Inner toe. The inner toe is the transition from the bed of the IJssel Lake to the edge of the dike body. The inner toe marks the other border of the core zone.
13. Protection zone IJssel Lake. Also on this side of the Afsluitdijk a protection zone of 150 meters is defined.

4.2.3. Hydraulic structures

The assessment and the design of the hydraulic structures are outside the scope of this MSc research. Because the main focus is to optimize the design for the dike body the hydraulic structures are not taken into account. Also in the assessment of the current configuration of the Afsluitdijk it is assumed that the Afsluitdijk will fail due to failure of the dike body and not due to failure at one of the sluice complexes. Because the sluice complexes do have an influence on the total probability of failure of the Afsluitdijk a brief introduction to the sluice complexes, their current state and the future developments are described in Appendix C.

4.3. Probability of failure of the Afsluitdijk

4.3.1. Fault tree analysis Afsluitdijk

Before the probability of failure can be calculated it must be clear how failure of the Afsluitdijk can be caused. To get insight in the possible causes of failure of the Afsluitdijk a fault tree is made. In this fault tree (Figure 4.3) the different failure mechanisms for the dike sections and the hydraulic structures are given. The turquoise boxes are part of the MSc thesis and the failure mechanisms that are investigated in this MSc thesis are shown in the orange boxes.

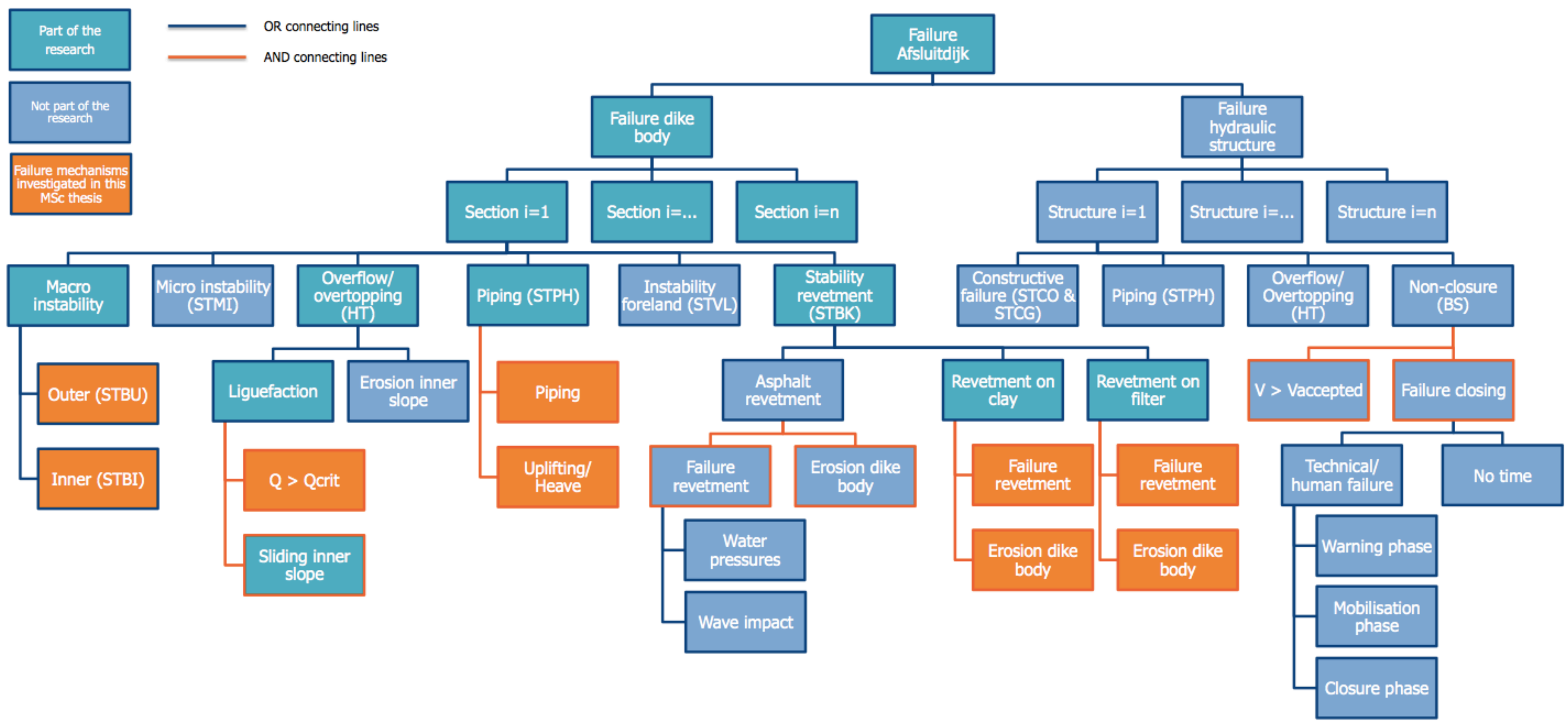

Figure 4.3: Fault tree for failure of the Afsluitdijk. The lila boxes are failure mechanisms that are not taken into account in this MSc thesis, the turquoise boxes are part of the investigation and the orange boxes are the failure mechanisms that are researched in this MSc research. The boxes connected with blue lines to the upper event are OR lines which means that one of the underlying events has to occur before the upper event occurs. The orange connecting lines are AND lines which means that all the underlying events must happen before the upper event happens.

4.3.2. Probability of failure of a single cross section

To assess the probability of flooding of the Afsluitdijk the process described in paragraph 3.4 is followed. First of all the probability of failure for the dike section is assessed and the probability of flooding of the hydraulic structures is for so far not included in this assessment. First all the failure mechanisms are tested for one dike section of the Afsluitdijk. The hydraulic boundary conditions for the Afsluitdijk are derived from Hydra-K. The method to derive the hydraulic boundary conditions and the results are shown in Appendix A.

First of all the probability of failure is calculated for different failure mechanisms for one dike section. The first failure mechanisms that is investigated is wave run-up and wave overtopping.

Wave run-up and overtopping
A full description of this failure mechanism and how this is calculated is given in Appendix B. The probability of failure depends on the critical overtopping discharge, which is the erosion resistance of the inner slope (strength) of the Afsluitdijk. Because this critical overtopping discharge is not easily calculated the probability of failure is calculated for different overtopping discharges. The results are shown in Table 4.2.

Critical discharge [l/s/m]	Return period failure [years]	Probability of failure [1/year]
0.1	13	7.69E-02
1	35	2.86E-02
5	103	9.71E-03
10	182	5.49E-03
25	464	2.16E-03
50	977	1.02E-03
100	2 770	3.61E-04
150	5 447	1.84E-04

Table 4.2: Probabilities of failure for different critical overtopping discharges for dike section 8B.

The probability of failure due to overtopping is strongly dependent on the critical overtopping discharges. In the VNK manual (Steenbergen et al., 2007) a method to determine this critical overtopping discharge is given, which is described in more detail in Appendix B. This critical overtopping discharge is dependent on the percentage of overtopping waves during a storm. In this MSc research this number is set at 100 % which is conservative and further research would be required to investigate this percentage. This results in a critical overtopping discharge with a lognormal distribution with a mean value μ of 51.5 l/s/m and a standard deviation σ of 10.1, the distribution is shown in Figure 4.4.

This distribution for the critical overtopping discharge is used to determine the total probability of failure. In a previous MSc thesis (Landa, 2014) extended research is done on the critical overtopping discharge. In this thesis it is concluded after tests that the Afsluitdijk is able to withstand a critical overtopping discharge of 30 l/s/m. With this critical discharge the probability of failure is larger than calculated with the VNK method.

Another critical overtopping discharge that can be assumed is the critical overtopping discharge that is used for the design of the alternatives by Witteveen + Bos. According to the starting point document (Witteveen + Bos, 2013) the critical overtopping discharge is 10 l/s/m. This results in a even larger probability of failure.

The VNK method assumes residual strength of the clay cover and the grass. If we do not take this into account we get a smaller critical discharge resulting in a higher probability of failure. The different results for these three critical overtopping discharges are shown in Table 4.3. The distribution for the critical overtopping discharge is given in Figure 4.5.

Because there are still many uncertainties about the critical wave overtopping and the erosion process itself further research is needed which can lead to new standards for the critical overtopping. An

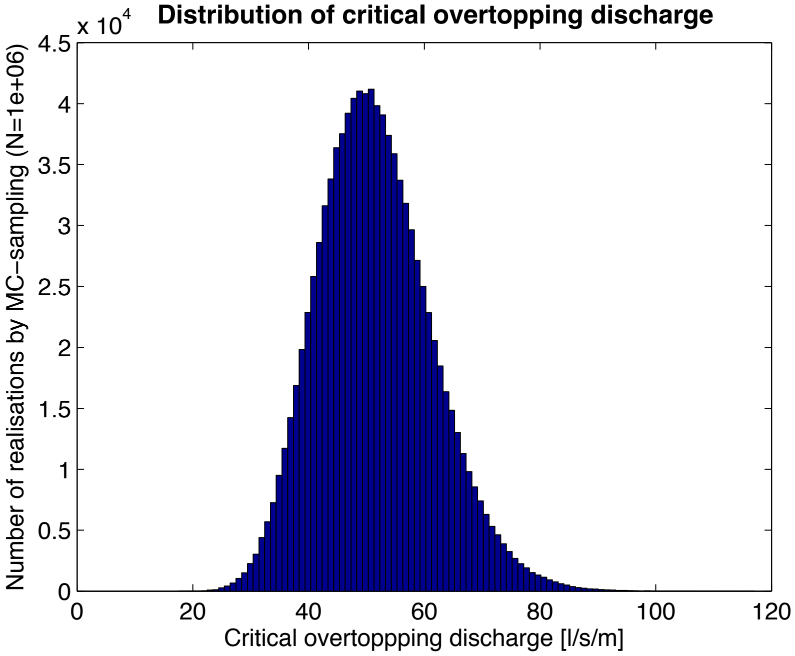

Figure 4.4: Distribution of the critical overtopping discharge with residual strength for dike section 8B.

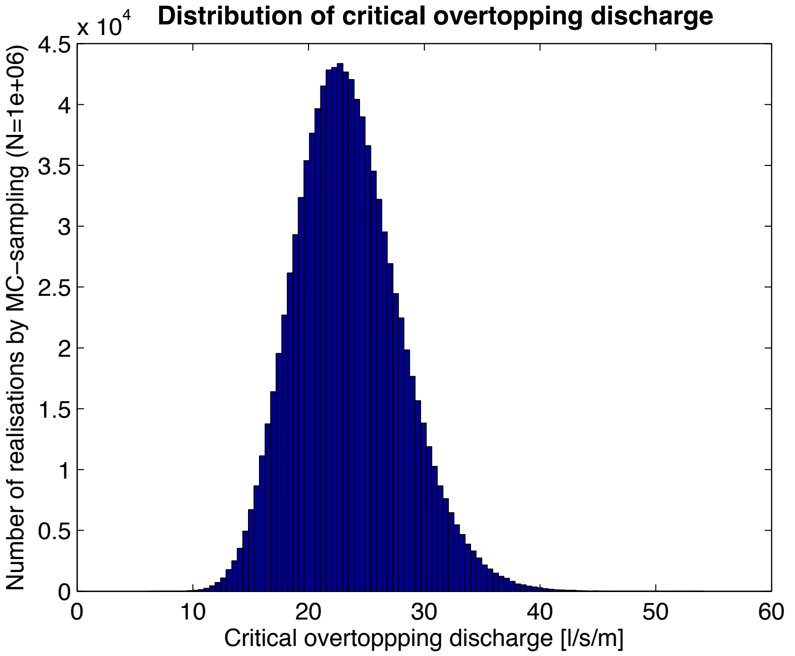

Figure 4.5: Distribution of the critical overtopping discharge without residual strength for dike section 8B.

example of this recent research is (Hai Trung, 2014), a recent Phd thesis in which the overtopping on grass covered dikes is investigated. In the future and especially with the new safety standards in 2017 in mind research like this is useful. However in this thesis the followed method will be the VNK method, so for the failure mechanism of overtopping this means that the probability of failure for dike section 8B is 1/1 605 per year.

Method	Critical discharge [l/s/m]	Return period failure [years]	Probability of failure [1/year]
Witteveen + Bos	10	182	5.49E-03
Afsluitdijk tests	30	580	1.72E-03
VNK (without residual strength)	μ=23.4 σ=4.6	397	2.52E-03
VNK (with residual strength)	μ=51.5 σ=10.1	1 157	8.64E-04

Table 4.3: Probabilities of failure for the three different approaches for the critical overtopping discharges for dike section 8B.

Failure stone block revetment

The failure of the outer slope revetment depends on the wave attack, the layout of the protection of the slope and the layout of the dike core. Because there are a lot of different types of protections a lot of different approaches determine the probability of failure. The different protection types (with their own corresponding failure mechanisms) are divided into main categories:

- grass cover
- stone blocks
- stone columns
- rubble
- asphalt cover
- concrete plates

A dike cover can consist of multiple covers, so each different cover segment has to be assessed. The current assessment method that is used in the Netherlands is a spreadsheet, which is called Steentoets. Steentoets is developed by Deltares and it is used to assess if the dike cover is strong enough, too weak or if more detailed investigation is needed. For a full description of Steentoets see (Klein Breteler, 2012).

A probabilistic calculation for the failure of the slope cover is not possible in Steentoets. To calculate the probability of failure in a probabilistic way there are three options:

1. Use the formulas that are embedded in the spreadsheet as an input in a Matlab script.
2. Make a link between Matlab and Steentoets.
3. Use a simplified assessment method in a Matlab script.

The main reason to choose the first option is that all the underlying calculations in Steentoets could be transformed into a Matlab script. The spreadsheet is based on VBA[2] and a conversion from VBA to Matlab is needed. After research on the internet it is clear that this conversion is possible but still requires a lot of manual labour and adaptions. Because the main advantage would be that this could be done automatically this is not the best option.

The second options requires a link between Matlab and Steentoets. If we could produce variables in Matlab and use this as an input in Steentoets a Monte Carlo simulation can be done which results in a probability of failure. A Matlab script can be generated that follows these steps:

- create a set of variables
- load this set of variables as an input in Steentoets
- run Steentoets
- retrieve the results from Steentoets and store them
- create a new random set of variables according to their distribution and do this process all over again

If this is done a lot of times the total numbers of a negative assessment can be divided by the total number of runs, resulting in a probability of failure. The downside is that this method requires a lot of calculation time and a lot of manual effort to produce the right script in Matlab.

[2]Visual Basics for Applications is a programming language that is used to automate and extend application programs (mostly used in Microsoft Office).

The third option is the simplest option. The probabilistic tools that are used for the failure mechanism of overtopping can be used in Matlab and with simplified formulas the probability of failure can be calculated. In the VNK method (Steenbergen et al., 2007) general formulas are described for four types of protections:

- grass cover
- placed stone blocks on clay
- placed stone blocks on a filter
- asphalt cover

As these formulas are simplifications and not all the different types of protection can be calculated the assessment of the current flood defences in the Netherlands is done with Steentoets. Because the layout of the dike section whit the highest hydraulic loads consists of stone blocks on a filter and grass the VNK method can be used to make a probabilistic calculation.

Taking the advantages and disadvantages of the three methods into account the third method is chosen for the probabilistic calculation. This method can be used for the assessment of the current configuration of the Afsluitdijk. If one of the alternatives consists of type of cover that is not described in this method a different approach needs to be found. The complete method can be found in Appendix B.

As mentioned above we look at the dike section that is exposed to the largest hydraulic loads. This is dike section 8B. The normative cross section consists of placed stone blocks on a filter. This failure mechanisms can results in a breach. This is only the case if the cover fails and the dike body will erode away during the storm duration. This residual strength of the dike is also taken into account. The residual strength is given as a conditional probability of failure which means that the probability of complete erosion of the dike body is given the fact that the revetment will fail. Therefore the probabilities of failure can be multiplied to come to a total failure probability with the residual strength taken into account. The results for dike section 8B are given in Table 4.4.

Failure mechanism	**Return period failure [years]**	**Probability of failure [1/year]**
stone blocks and filter	6 024	1.66E-04
conditional failure dike core	2.64	3.79E-01
total failure	15 903	6.29E-05

Table 4.4: Probability of failure of the placed stone blocks on a filter including residual strength.

Macro instability

During a period of high water the inner slope can slide and this leads to instability. The water retaining function of the dike may be intact but if overtopping or overflow starts a breach is inevitable. If the water level on the outer side of the dike drops rapidly the water inside the dike is not able to follow. The pressure inside the dike becomes to high which results in a sliding of the outer slope. Because this only occurs when water levels are dropping this failure mechanism does not immediately result in flooding. However if the water levels rise again before the dike is repaired this may be a threat. Both these failure mechanisms are investigated for the Afsluitdijk.

To investigate the failure mechanism of macro instability the Bishop method is used. The slip circle method of Bishop is based on a moment equilibrium of the whole slip circle and vertical force equilibrium of the individual slices. The Bishop method considers the driving moments of the slices by soil weight, water pressures and loads around the center of a slip circle. Stability requires that the sum of these driving moment is equal to (or less than) a certain resisting moment. A more detailed description of this method can be found in Appendix B.

A probabilistic calculation is done with M-Prostab, which is a software module that is also used in the VNK method (Trompille et al., 2011). M-Prostab is a Bishop probabilistic random field module of Mstab, a geotechnical analysis software module developed by Deltares. The input variables in M-Prostab (the average values and standard deviations) have to be adapted before a calculation is done. A more detailed description of Mstab[3] and the adaption of the input variables can be found in (Bakker, 2004).

Another option for a probabilistic calculation is according to the same method which is used for the previous failure mechanisms. For every water level the normative slip circle can have a different center and a different radius. To find this normative slip circle an iterative process is used which takes time. If a Monte Carlo simulation is used to determine the total probability of failure then for every calculation step this iterative process is done, which results in large calculation times. Also to set up this script in Matlab requires effort and time. Because the calculation of the macro instability is already developed in Mstab it does not make sense to create a new method, especially because there are no benefits.

M-Prostab can calculate the probability of failure for different water levels. The results for the instability of the inner and outer slope for different water levels are shown in Table 4.5 and 4.6. A reliability index can be derived from the probability of failure for each water level and with this a relationship between the reliability index and the water level can be found. With the probability density function of the water level and the reliability index function the total probability of failure can be found.

In this MSc thesis the total probability of failure is not derived because these probabilities of failure found for the different water levels are very low compared to the failure mechanism of wave run-up and overtopping. Therefore this failure mechanism is negligible. In the calculations it is assumed that the water levels have penetrated the whole dike completely. Also for the instability of the outer section it is assumed that the water level will drop rapidly to NAP, which is not the case in real life. Therefore the calculated probabilities of failure are even lower in reality, especially for the instability of the outer slope.

water level [m]	Pf STBI	P water level	Pf total	Return period
5.57	1.30E-03	1.00E-04	1.30E-07	7.69E+06
4	6.18E-07	1.10E-02	6.80E-09	1.47E+08
3	1.70E-16	0.33	5.61E-17	1.78E+16
2	3.61E-19	1	3.61E-19	2.77E+18
1	3.61E-19	1	3.61E-19	2.77E+18

Table 4.5: Probability of failure for the inner slope for a normative cross section.

water level [m]	Pf STBU	P water level	Pf total	Return period
5.57	1.78E-04	1.00E-04	1.78E-08	5.62E+07
4	2.92E-05	1.10E-02	3.21E-07	3.11E+06
3	6.45E-06	0.33	2.13E-06	4.70E+05
2	1.02E-06	1	1.02E-06	9.80E+05
1	9.28E-08	1	9.28E-08	1.08E+07

Table 4.6: Probability of failure for the outer slope for a normative cross section.

Piping and heave

Piping is a micro-instability process as small particles are eroded from underneath a cohesive layer in the subsoil. The seepage water erodes particles from underneath the dike if the hydraulic head is large enough. The erosion forms a pipe which grows from the inner to the outer side and this can lead to a settlement and failure of the dike. This mechanism can only occur underneath a cohesive layer (clay) on top of an erodible and permeable soil (sand).

The assumption is that the probability of failure of the Afsluitdijk due to failure is so small it can be neglected. The assessment method for piping has changed and older rules that have been used in the

[3] Mstab is an older version of D-Geo Stability

past are no longer valid. Therefore it is desirable to assess the Afsluitdijk on piping and heave (without detailed calculations) with the new assessment standard.

To assess a soil structure or hydraulic structure on piping several formulas are used. All these different formulas are based on three main variables:

- The head (difference in water level) over the structure.
- The length of the aquifer from the high water entrance point to outflow point (mostly were the clay cover is the smallest).
- The flow resistance of the aquifer.

A description of these different formulas and the current piping assessment method is described in more detail in Appendix B. A deterministic calculation with characteristic values is done for piping to show the safety factor:

$$SF = \frac{\Delta H_c/(\gamma_n \cdot \gamma_b)}{\Delta H - 0.3d} \approx 5$$

in which:

ΔH_c	[m]	critical hydraulic head over structure
ΔH	[m]	occurring hydraulic head over structure
d	[m]	thickness of cover layer
γ_n	[-]	safety factor
γ_b	[-]	schematization factor

With this safety factor it is shown that piping is not an issue for the Afsluitdijk because it will fail due to other failure mechanisms before piping will occur. Therefore the assumption that piping is not a relevant failure mechanisms for the Afsluitdijk is valid.

4.3.3. Total probability of failure including correlation and length effect

In (Projectbureau VNK2, 2012) a screening method is proposed that defines which dike sections have to be analyzed and for which failure mechanisms. This selection is made on assessment results, insights of the flood defense manager and insights from VNK. A proposal is given in Table 4.7 which is a minimum requirement, more calculations can always be done if necessary.

Failure mechanism	**Sections**	**Comments**
Wave run-up and overtopping	All	By calculation of all sections we have a probability of failure for all the dike sections
Piping and heave	Selection, around 30%	All the sections in which piping/heave is a relevant failure mechanism must be calculated based on indicator and spacial spread
Slope protection and erosion	Selection, around 30%	All relevant sections based on assessments and insights from manager and VNK
Macro stability	Selection, around 10%	Selection based on SF assessments, insights manager and spacial spread.

Table 4.7: Selection criteria for the calculation of dike sections.

With the results from this chapter it is concluded that the failure mechanisms piping and macro stability do not have a major influence on the total probability of failure. To calculate the total probability of failure of the dike body of the Afsluitdijk these failure mechanisms are not part of the calculation in PC-Ring.

The correlation and length effect is calculated with PC-Ring according to the method described in paragraph 3.2.2. For the mechanism wave run-up and overtopping all the non-protected dike sections are calculated in PC-Ring. The reason why the protected dike sections are not taken into account is because the hydraulic boundary conditions in those locations strongly depend on the protection of

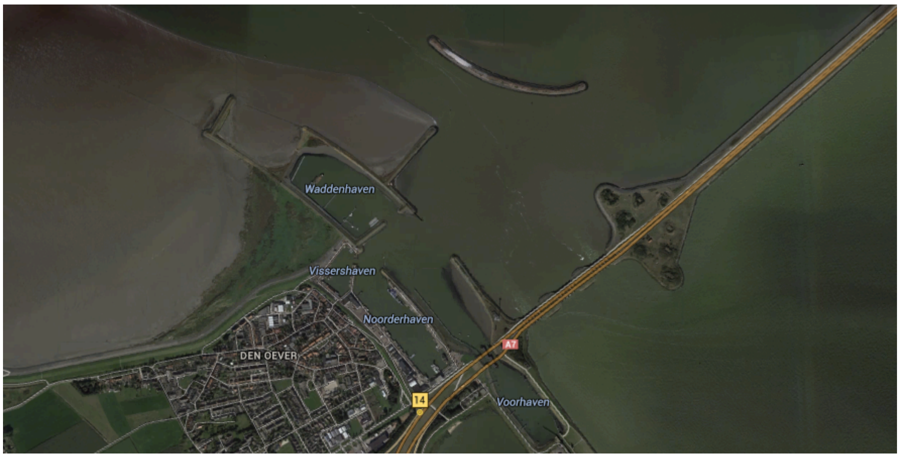

Figure 4.6: Top view of the sluice complex at Den Oever with the breakwaters North of the Afsluitdijk (source: Google Maps).

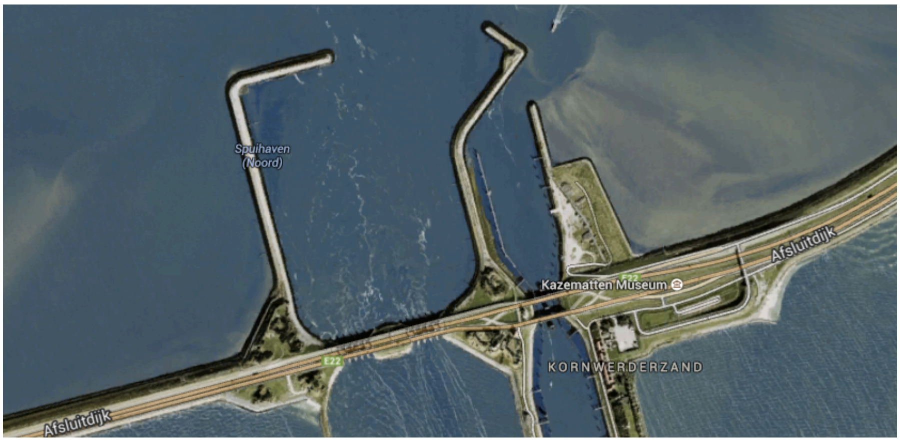

Figure 4.7: Top view of the sluice complex at Kornwerderzand with the breakwaters North of the Afsluitdijk (source: Google Maps).

the breakwaters in front of the Afsluitdijk. The breakwaters at Den Oever and Kornwerderzand are respectively shown in Figure 4.6 and 4.7.

Symbol	Description	Unit	Spatial spread		Time spread	
			dx	ρx	Δt	ρt
H_{crown}	dike crown height	m	300 m	0	-	-
H_{toe}	dike toe height	m	300 m	0	-	-
$tan(\alpha_{up})$	upper slope	-	150 m	0	-	-
$tan(\alpha_{low})$	lower slope	-	150 m	0	-	-
B	berm width	m	300 m	0	-	-
H_b	berm height	m	300 m	0	-	-
m_{q0}	model factor overtopping	-	section	0.7	-	-
mH	model factor H_s	-	900 m	0.7	-	1
mT	model factor T_s	-	900 m	0.7	-	1
Δh_{loc}	error in local water level	m	6000 m	0.5	-	1
t_s	storm duration	hour	-	1	12 h	0

Table 4.8: Spread in space and time of stochastics used to determine failure due to overtopping.

The influence of these breakwaters is not calculated in PC-Ring and therefore the hydraulic boundary conditions at these locations are more severe than in reality. If these sections were taken into account this would lead to a probability of flooding that is higher than in reality. These dike sections may still

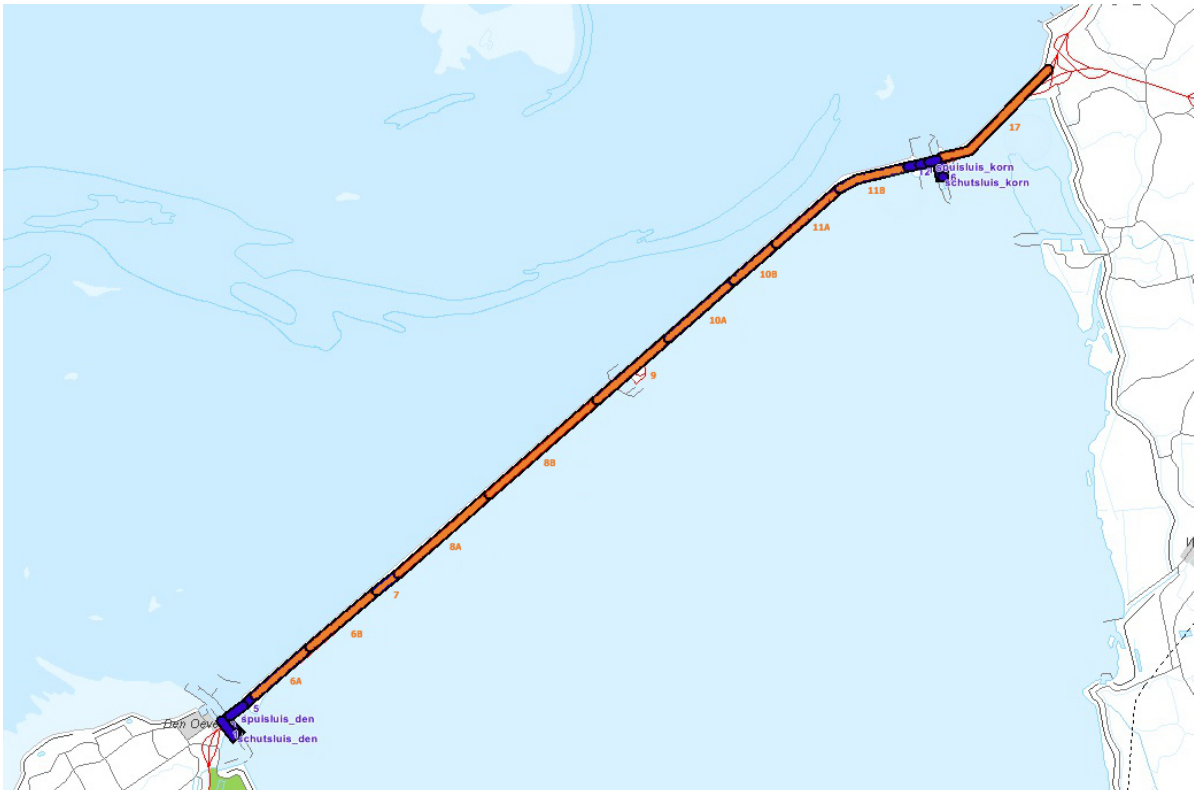

Figure 4.8: Top view of the Afsluitdijk with the selected dike sections for the calculation in PC-Ring in orange.

fail in reality so it is recommended to investigate the hydraulic loads and the strengths of these dike sections as well. Also the sluice complexes are not taken into account. An overview of the dike sections for which the mechanism of wave run-up and overtopping are calculated is shown in Figure 4.8.

As shown in Table 4.7 it is recommended to calculate around 30% of the dike sections for the mechanism slope protection and erosion. The dike sections with the most severe hydraulic boundary conditions are selected to ensure that the weakest sections are calculated in PC-Ring. These dike sections are from dike section 8A until 10B.

To calculate the total probability of failure of the dike body the correlation and length effect is combined with PC-Ring. The coefficients that are used for the different variables are shown in Table 4.8 for the failure mechanism overtopping and in Table 4.9 for the instability of the protection.

Symbol	**Description**	**Unit**	**Spatial spread**		**Time spread**	
			dx	ρx	Δt	ρt
L_k	clay cover width	m	900 m	0	-	-
$L_b k$	dike core width at crown height	m	1500 m	0	-	-
$tan(\alpha_{out})$	outer slope	-	150 m	0	-	-
$tan(\alpha_{in})$	inner slope	-	150 m	0	-	-
$c_r k$	factor erosion resistance clay cover	ms	-	-	12 h	0
β_r	angle in reduction factor r	∘	-	-	-	-
D	stone thickness	m	1500 m	0	-	-
Δ	relative density stones	-	1500 m	0	-	-
c_k	factor strengt of stones on clay	-	section	1	-	-
d_f	granular filter thickness	m	300 m	0.7	-	-
D_{f15}	grain size 15% fractile of filter layer	m	300 m	0.5	-	-
s	width of the splits between stones	m	300 m	0.5	-	-

Table 4.9: Spread in space and time of stochastics used to determine failure due to instability of the slope.

With these correlation coefficients and the lengths of the dike sections taken into account the results for the probabilities of failure are shown in Figure 4.9 for overtopping and in Figure 4.10 for instability of the revetment. The exact numbers and the combined probability of failure are given in Table 4.10.

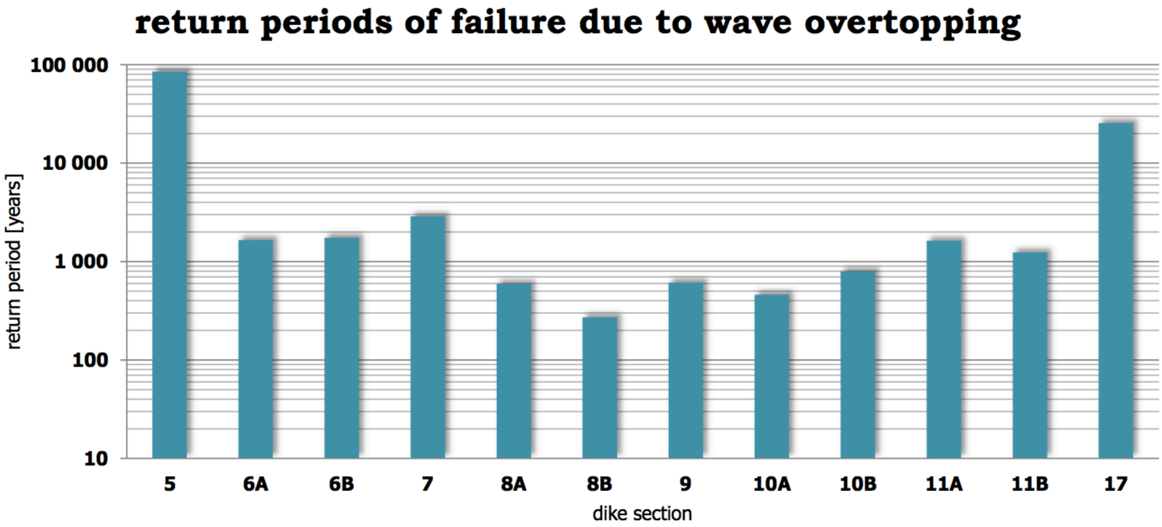

Figure 4.9: Probability of failure for the dike sections due to the failure mechanism of wave run-up and overtopping.

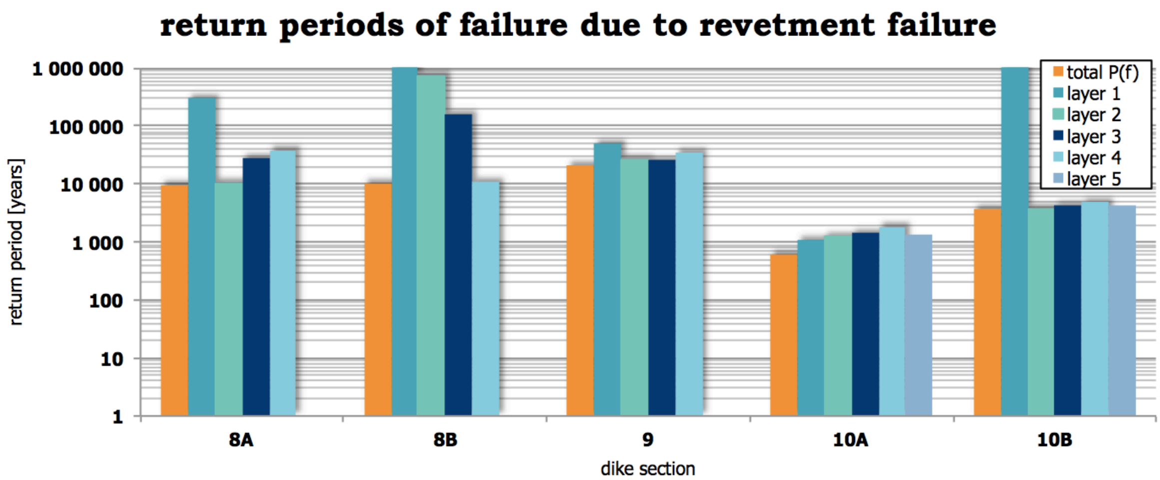

Figure 4.10: Probability of failure for the dike sections due to the failure mechanism of instability of the outer slope protection.

With the numbers for the failure mechanisms of wave run-up and overtopping and instability of the outer slope revetment the total probability of failure is calculated in PC-Ring. The results are shown in Table 4.11 The total probability of failure for the Afsluitdijk is high compared to the current standard. The current standard for the Afsluitdijk states that the Afsluitdijk must be able to withstand a storm with an exceedance frequency of 1/10 000 per year, which is a factor 50 higher than the calculated probability of failure. The reliability of the calculations is questionable because the length effect has a large influence on the total probability of failure, which is discussed further in this paragraph. What the new safety standard is for the Afsluitdijk is not decided so far, however it is concluded that the Afsluitdijk needs improvement to prevent the occurrence of a breach in the near future.

The cumulation of failure mechanisms is not taken into account in the determination of the probability of failure. The cumulation of failure mechanisms implies that if one mechanism starts to fail it may introduce a higher chance of failure of the other mechanisms as well. For example if stone blocks are eroded from the outer revetment the macro instability of the outer slope becomes less which could

Dike Section	Wave run-up and overtopping			Instability stone revetment		
	β	P_f	R	β	P_f	R
5	4.23	1.18E-05	84 897			
6A	3.24	6.06E-04	1 650			
6B	3.16	7.96E-04	1 256			
7	3.39	3.47E-04	2 882			
8A	2.93	1.68E-03	594	3.71	1.02E-04	9 773
8B	2.68	3.68E-03	272	3.73	9.59E-05	10 431
9	2.94	1.63E-03	613	3.90	4.74E-05	21 091
10A	3.07	1.09E-03	920	2.94	1.63E-03	613
10B	3.24	6.03E-04	1 658	3.46	2.69E-04	3 715
11A	3.23	6.16E-04	1 622			
11B	3.41	3.26E-04	3 067			
17	3.95	3.92E-05	25 498			
total	**2.62**	**4.36E-03**	**229**	**2.94**	**1.65E-03**	**607**

Table 4.10: Reliability indexes, probabilities of failure and return periods for all the dike sections for overtopping and instability of the stones.

failure mechanism	Reliability index	Probability of failure	Return period
overtopping	2.62	4.36E-03	229
instability stones	2.94	1.65E-03	607
total	**2.56**	**5.21E-03**	**192**

Table 4.11: Total probability of failure of the Afsluitdijk for the current design.

result in sliding of the outer slope. This effect is not taken into account in the design and assessment standards but it is recommended to investigate this for a more precise determination of the probability of failure.

As stated before the length effect has a large influence on the results from PC-Ring. The length of the different dike section is shown in Figure 4.11. Some of the results for the mechanism of wave overtopping can be explained by the length of the dike sections. If we look at all the dike sections that are split up in an A and B part, we see difference between the two parts for the probability of failure. Because the cross sections that are used as an input are the same the difference in the results is caused by something else. Besides the different input locations for the hydraulic boundary conditions it is clear that the parts with smaller lengths also have a smaller probability of failure.

Another result is the probability of failure due to overtopping for dike section 5. Compared to the other dike sections it has a probability of failure which is up to 300 times smaller than the other sections. This is explained by the less severer hydraulic boundary conditions at this location and the fact that it has a very wide berm which reduces the amount of wave run-up and therefore overtopping. The cross section of section 5 is shown in Appendix F.

As expected the section with the highest probability of failure is section 8B. The probability of failure of this section has a large influence on the combined probability of failure for the failure mechanism of wave run-up and overtopping. The amount of influence of the weakest link, dike section 8B, on the total probability of failure is discussed further in this paragraph taking the length effect into account.

For the failure mechanism of instability of the stones dike section 8B is not the weakest link. The probabilities of failure for the dike sections 10A and 10B are much higher with a factor of 3 up to a factor of 17. This is explained by the fact that at these sections the weakest revetment parts consist of stones placed directly on clay while at the other dike sections they are placed on a granular filter. The values for these revetment sections are from an assessment for the Afsluitdijk which is done with the spreadsheet Steentoets (Klein Breteler, 2012). An overview of the revetment blocks from dike section 8A up to 10B in the spreadsheet Steentoets is shown in Appendix G.

The results for the total probability of failure for the Afsluitdijk are close the assumed probability of

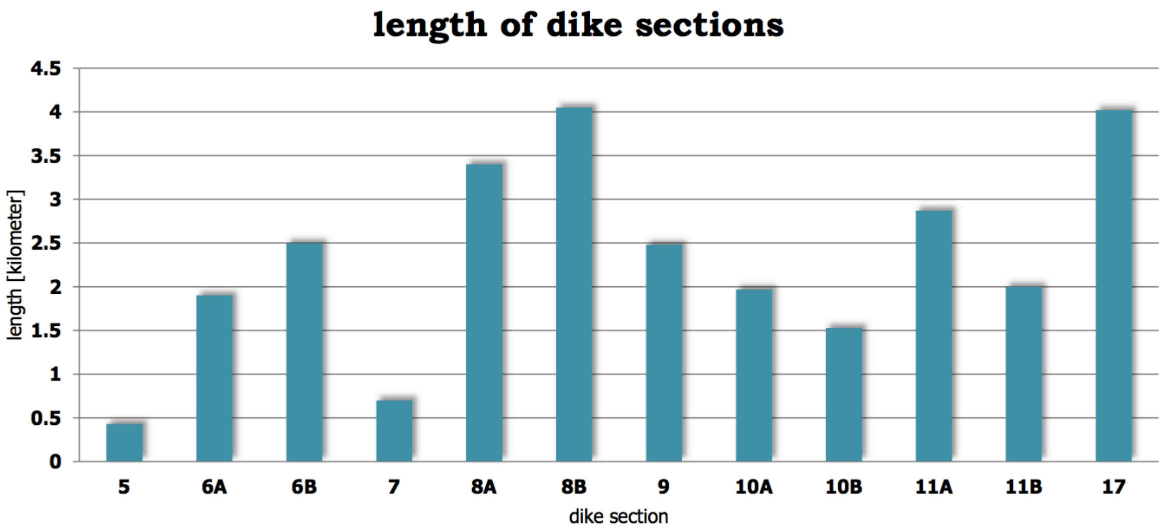

Figure 4.11: Length of the dike sections of the Afsluitdijk.

failure of the Afsluitdijk in an economic optimization study for the Lake IJssel region (Zwaneveld and Verweij, 2014). In this study it is assumed that the Afsluitdijk in 2012 has a strength of 1/250 per year, which is close to the 1/192 found in this MSc research.

The results from the calculation with PC-Ring differ a lot from the results that are obtained with the model used in this MSc research. In Table 4.12 the difference between the results for some dike sections are shown. The difference between the results from PC-Ring and the Matlab model is also shown in Figure 4.12.

Dike Section	PC-Ring		Matlab model	
	P_f	R	P_f	R
7	3.47E-04	2 882	2.96E-04	3 378
8B	3.68E-03	272	7.08E-04	1 412
11A	6.16E-04	1 622	2.45E-04	4 081
11B	8.09E-04	1 237	1.54E-04	6 493

Table 4.12: Difference between the models for 4 dike sections including the length effect.

A maximum difference with a factor of around 5 between PC-Ring and the model can be explained by the length effect in a dike section. To show this effect some calculations are done with PC-Ring for different lengths of the dike sections. The results are shown in Table 4.13.

From Table 4.13 it is observed that the length of a dike section has a large influence on the total probability of failure. If we compare the results from a calculation in PC-Ring for short dike sections with a length of 100 meter with the results from the model the differences are much smaller, shown in Table 4.14 and Figure 4.13.

With a maximum difference factor of 0.5 between PC-Ring and the model it can be concluded that the length effect has a major influence on the total probability of failure. Also the results from PC-Ring and the model used in this MSc research are more reliable as they are close to each other.

The length effect between the different dike sections has also an influence on the total probability of failure of the Afsluitdijk. For the critical dike section 8B (with the most severe hydraulic boundary conditions) a probability of failure for the mechanism overtopping is 1/272 per year. The combined probability of failure of all the dike sections is a factor 1.2 higher. This result can be used in the optimization of the alternative in Chapter 5.

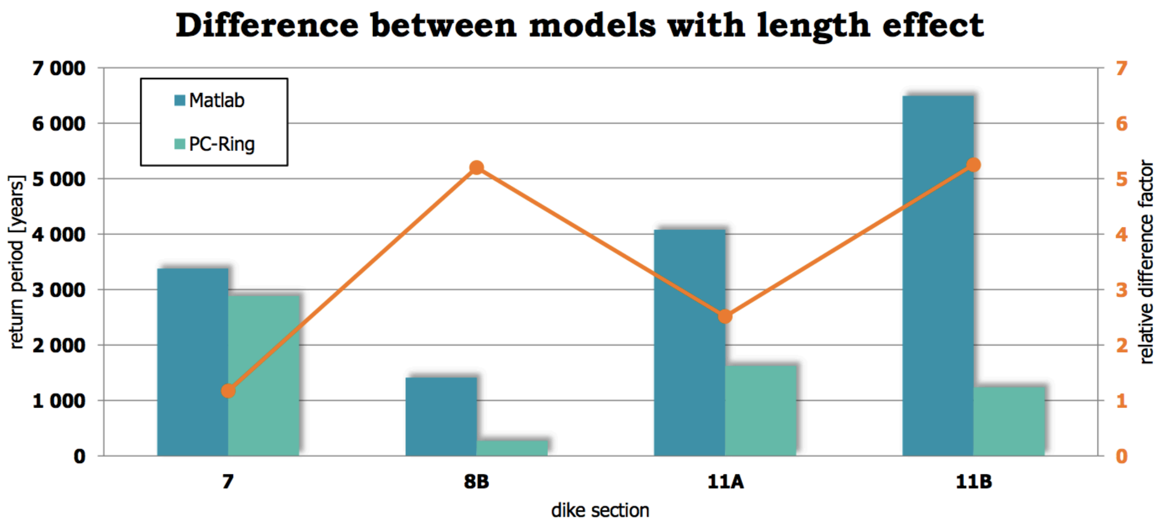

Figure 4.12: Difference between the results including the length effect in PC-Ring.

Dike Section	Length	Actual lengths		1 000 meter		100 meter	
		P_f	R	P_f	R	P_f	R
5	430	1.18E-05	84 898	1.25E-05	80 271	1.18E-05	84 898
6A	1 900	6.06E-04	1 650	4.17E-04	2 399	1.37E-04	7 305
6B	2 500	5.73E-04	1 745	3.44E-04	2 903	1.10E-04	9 071
7	700	3.47E-04	2 882	7.57E-04	1 321	2.41E-04	4 142
8A	3 400	1.68E-03	594	9.90E-04	1 010	3.33E-04	3 002
8B	4 050	3.68E-03	272	2.08E-03	481	7.21E-04	1 388
9	2 480	1.63E-03	613	9.63E-04	1 038	3.06E-04	3 273
10A	1 970	2.17E-03	461	1.43E-03	701	2.02E-04	4 949
10B	1 530	1.26E-03	792	9.52E-04	1 050	1.30E-04	7 694
11A	2 870	6.16E-04	1 622	3.80E-04	2 634	1.11E-04	8 990
11B	2 000	8.09E-04	1 237	4.33E-04	2 307	6.55E-05	15 266
17	4 020	3.92E-05	25 498	1.32E-05	75 567	8.31E-06	120 387
total		**4.36E-03**	**229**	**4.60E-03**	**218**	**1.41E-03**	**709**

Table 4.13: Influence of the length effect on the probability of failure due to overtopping.

Dike Section	PC-Ring		Matlab model	
	P_f	R	P_f	R
7	2.41E-04	4 142	2.96E-04	3 378
8B	7.21E-04	1 388	7.08E-04	1 412
11A	1.11E-04	8 990	2.45E-04	4 081
11B	6.55E-05	15 266	1.54E-04	6 493

Table 4.14: Difference between the models for 4 dike sections excluding the length effect.

With Equation D.5 and a safety standard for the Afsluitdijk the standard for an individual dike section per mechanism can be calculated. This equation can also be written as:

$$P_{eis,dsn} = \frac{P_{norm} \cdot \omega}{N}$$

in which N is the factor that takes the length effect into account and ω the factor that takes the influence of the failure mechanism into account. So for the failure mechanism of wave run-up and overtopping we assess as a reasonable estimate for the factor N a value of around 1.2 for the Afsluitdijk.

The factor N with a value of 1.2 has to be investigated further because if the design of the weakest

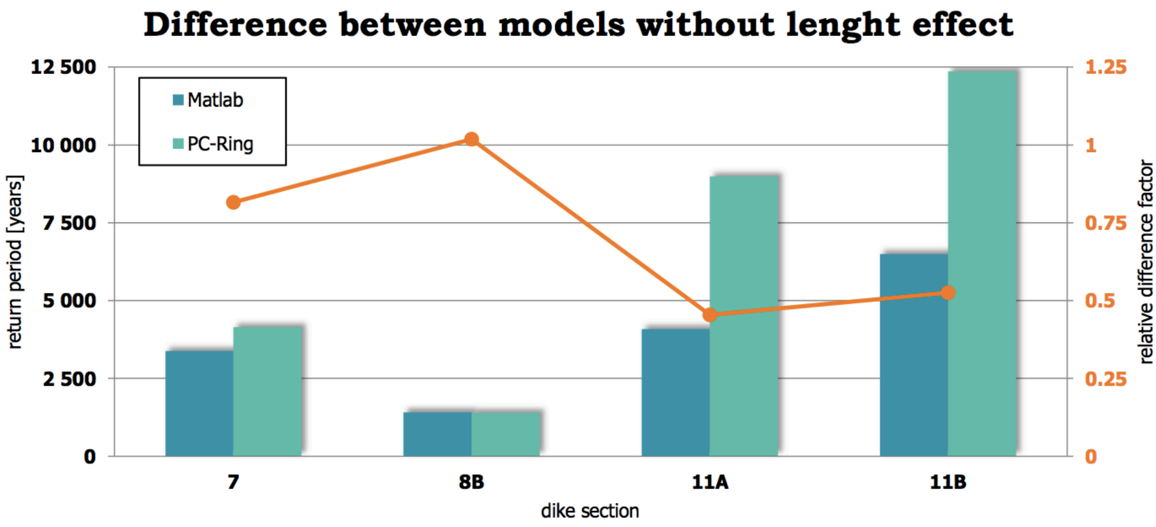

Figure 4.13: Difference between the results without the length effect in PC-Ring (length of dike sections = 100 meters).

section, in this case section 8B, is reinforced it is not the weakest link anymore. This results in a higher factor for N which has an influence on the design standard. For a higher factor of N the allowed probability of failure is smaller, resulting in a stronger and more robust design. Therefore it is recommended to investigate the factor N with the recommended designs to see the influence of the strengthening of the weaker sections.

The result of this combined probability of failure is also checked with the critical dike section 8B. It can be stated that this weakest cross section will always fail first and that the chance of a breach at another location during a single storm event is negligible. With this assumption the length of dike section 8B is changed to the total length of the Afsluitdijk (which is about 32 km). Because it is subjected to the same severe hydraulic boundary conditions, but now for a length of 32km this may result in a larger probability of failure. Therefore another check is done where all the dike sections are given the same geometry as dike section 8B, but they suffice less sever hydraulic loads than in dike section 8B. The results for these two methods and for the normal calculation are shown in Table 4.15.

Method in PC-Ring	P_f	R
normal	4.36E-03	229
dike section 8B with a length of 25.5 km	5.05E-03	198
cross section of 8B for all dike sections	5.19E-03	193

Table 4.15: Difference between the models for 4 dike sections including the length effect.

From Table 4.15 it is concluded that the results for the three different methods are close to each other. Therefore a first check in PC-Ring for the optimization of a alternative is done for dike section 8B with a length of 25.5 km.

With the correlation and length effect taken into account with PC-Ring the final results show that the current configuration of the Afsluitdijk has a probability of failure of 1/192 per year. As stated above this probability is dominated by the failure mechanism wave run-up and overtopping. However in this calculation failure of the sluice complexes is not taken into account. In reality this probability of failure is therefore higher. As stated before the length effect has a large influence on the total probability of failure. Therefore it is recommended to investigate the possibilities to reduce the uncertainty in the variables that are used to calculate the probability of failure. For example more detailed measurements of the dike height result in lower uncertainties which results in a smaller deviation and this will finally result in a lower probability of failure.

The consequences of a breach somewhere in the Afsluitdijk are not as severe as failure of a regular dike with valuables en inhabitants in the hinterland. In case of a breach in the Afsluitdijk the water level on

Lake IJssel will rise. The raise of the water level on Lake IJssel results in larger hydraulic loads on the flood defences along Lake IJssel which results in higher probabilities of failure of those flood defences. How much this probability of failure will increase depends on the amount of water flowing into Lake IJssel in case of failure of the Afsluitdijk and the growth of the breach itself. An investigation on this is done (Wijbenga and Meijer, 2006) which shows that the water level on Lake IJssel will rise with 5 to 25 cm (depending on the location). The influence of this higher water level on the flood defences has to be investigated before definite conclusions on the current probability of failure of the Afsluitdijk are drawn. Also the hydraulic structures have to be taken into account in this investigation.

The current safety standard for the Afsluitdijk is safety against hydraulic loads with an exceedance frequency of 1/10.000 per year. According to this standard the current safety of the Afsluitdijk is insufficient and improvement in the design is needed. In the current safety standard this exceedance frequency is chosen to comply with the adjacent flood defences, regardless of the possible consequences of failure of the Afsluitdijk. In the new safety standards these consequences will be taken into account which could result in lower probabilities of failure. An economic optimization for the standard of the Afsluitdijk is done (Zwaneveld and Verweij, 2014) which results in an optimal design of the Afsluitdijk with a probability of failure of 1:9.400. This investigation is based on uncertain factors and there still remains discussion about this proposed standard for the Afsluitdijk. However it can be concluded that the current configuration of the Afsluitdijk with a probability of failure of around 1/200 per year is insufficient. Therefore improvement of the Afsluitdijk is needed and the new designs by W+B are evaluated and one reference alternative is optimized in Chapter 5.

5

New design of the Afsluitdijk

In this chapter the alternatives that are designed according to a system design of the Afsluitdijk (Systeem Ontwerp Dijklichaam (SOD) is the Dutch acronym for system design dike) are described. A full report of the SOD is shown in (Witteveen + Bos, 2014). One of the alternatives is chosen for further detailed investigation in this MSc research.The alternative that is optimized is the reference alternative proposed by Witteveen+Bos. The probability of failure of this alternative is assessed and this reference alternative is used as a starting point for an optimization. The safety standard in this optimization is assumed to be 1/9 400 per year, which is a proposed safety standard after an investigation by the CPB (Zwaneveld and Verweij, 2014). The aim is to find a design that meets this safety standard with minimum costs.

Finally in this chapter some solutions for the uncertainty of the proposed safety standard are proposed. Because it is still unclear what the new safety standard will be for the Afsluitdijk in the future some possible adjustments to the optimized design are proposed. These adjustments aim at (relatively large) reductions of the probability of failure with minimal costs and without major impacts on the design and landscape value of the Afsluitdijk.

5.1. alternatives for the Afsluitdijk (SOD)

In this section the alternatives are described. These alternatives are described to show possible solution for the strengthening of the Afsluitdijk. Finally the reference alternative is chosen for further optimization.

5.1.1. General description SOD

In 2006 an assessment of the Afsluitdijk pointed out that it does not meet the safety standards against flooding from the Wadden Sea. This is the case for both the dike body and the sluice structures at Den Oever and Kornwerderzand. In 2011 the Dutch Government decided to invest in a renovation of the Afsluitdijk. In this year the National Structure Vision Afsluitdijk (Dutch acronym: Structuurvisie Toekomst Afsluitdijk (STA)) was made in which a preference alternative is described. This reference alternative consists of a phased construction of a overtopping resilient dike body with a green character and a bicycle path at the side of Lake IJssel. Besides this the hydraulic structures are upgraded so the Afsluitdijk is able to withstand a water level and wave height with a probability of occurrence of 1/10 000 per year until at least 2050. In the current plans the overtopping resilient dike body that can withstand large volumes of overtopping waves is no longer in sight.

The preference decisions about the water safety and the discharge volume of out of Lake IJssel are made separately. Because they strongly relate with each other it is decided to combine these two issues into one single project Afsluitdijk. In the SOD the solution space from the first phase is reduced, based on the (provisional) decision for the dike body of the system Afsluitdijk. Within this reduced solution space a reference alternative is made. The reference alternative is not the final design but it is to show the feasibility and affordability of the renovation. In this MSc research the focus will be on the design of the dike body of the Afsluitdijk.

The design of the dike body must meet the following functional requirements:

1. Retaining high water.
2. Retaining water (to maintain water levels in Lake IJssel).
3. Separation of fresh and salt water.
4. Withstand traffic load.
5. provide habitat for flora and fauna.
6. Provide landscape value.

In a design and assessment memorandum from phase 1 the solution space[1] is filled with concrete alternatives: A1, B1, B2, C1 and D1. These alternatives must meet the functional requirements and are set up to find the corners of the solution space. The solutions of the alternatives are described with three 'design rotaries':

1. Measures on the inner slope of the dike.
2. Heightening of the crown.
3. Adjusting the protection on the outer slope of the dike.

In Figure 5.1 the existing dike profile of the Afsluitdijk is shown by the number 1 and the solutions space by number 2. The design rotaries are also shown in this Figure.

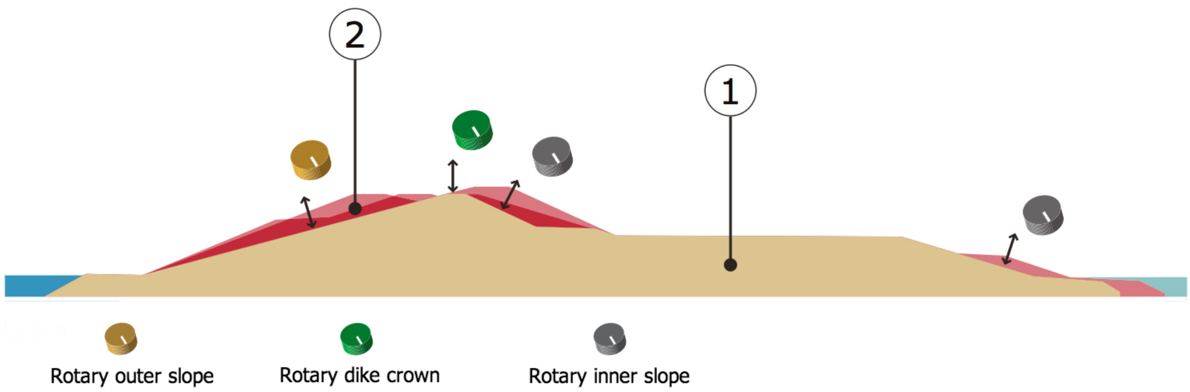

Figure 5.1: Design rotaries for the renovation of the Afsluitdijk.

From the assessment of the protection on the outer slope it follows that the current revetment does not meet the safety standard in the design period. The basalt blocks that are currently placed on the outer slope can not be reused. This means that the third 'design rotary' always has to be used in the alternatives. A description of the different alternatives in the SOD is given in the following subsection.

5.1.2. Description of the alternatives for the Afsluitdijk

A1: Retain the current geometry

This alternative is based on the goal to minimize the adjustments to the current dike profile and retain the current geometry. The outer slope of basalt will be covered with an asphalt layer, which results in a smooth outer slope leading to high overtopping discharges in the order of 140 l/s/m (with a 1/10 000 per year probability). This means that the the crown and the inner slope must be made overtopping resilient. This is done by an asphalt underlayer, covered with a ground layer with grass on top of it. The berms on the side of the road and the bicycle path will be constructed with open stone asphalt to guarantee the overtopping resistance. In Figure 5.2 an overview is shown with in the front the current profile of the dike body and the new design behind.

[1]The solution space is defined as a set of variable requirements that the solution has to meet.

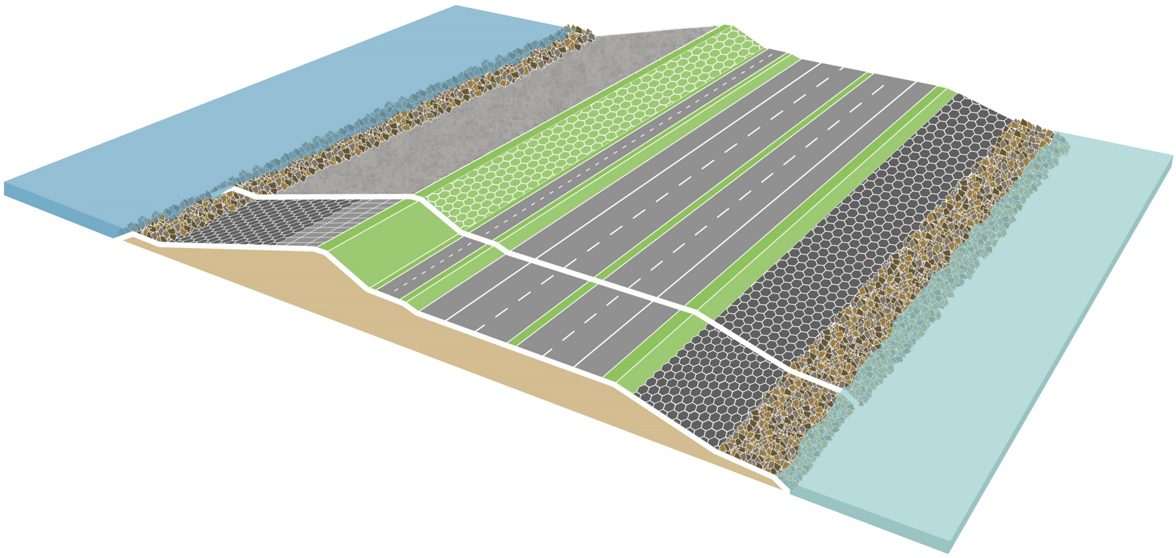

Figure 5.2: A schematic overview of alternative A1.

B1: Adjust the inner slope
This alternative is derived from the reference alternative for an overtopping resilient dike presented in the STA. This solution is based on a reduction of the angle of the inner slope to improve the stability. The space that is needed to do this will be created by moving the location of the bicycle path to Lake IJssel side.

The outer slope will be covered with rubble, which reduces the overtopping discharge to the order of 10 l/s/m. With this overtopping discharge the inner slope can be covered by grass. The crown of the dike will be wide and consists of partly rubble and partly grass. The angle of the inner slope will be reduced to 1:4 and will be grass covered.

The bicycle path will be replaced to a 5 meter wide berm at the side of Lake IJssel. The slope on this side will have a revetment of concrete columns and the bicycle path will be constructed with asphalt. An overview is given in Figure 5.3.

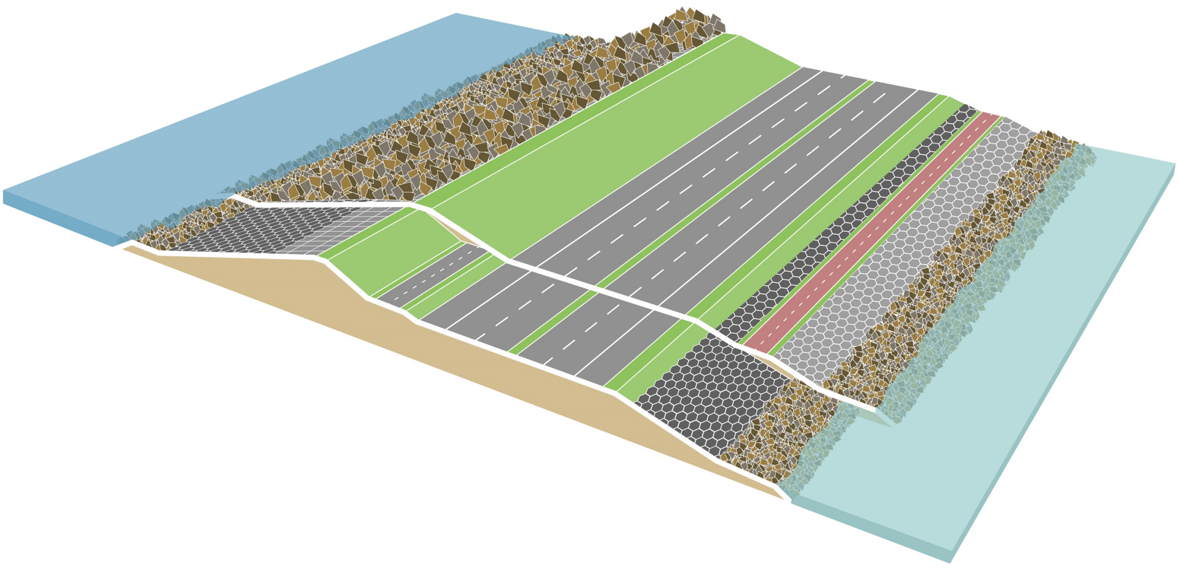

Figure 5.3: A schematic overview of alternative B1.

B2: Adjust the inner slope

This alternative is focussed on retaining the bicycle path at the the Northern side of the highway A7 to prevent space occupancy in Lake IJssel. This also solves the bottleneck in alternative B1, which is the crossing of the bicycle path with the hydraulic structure at the Lake IJssel side. The berm on the inner slope is primarily placed for stability but with the bicycle path on the berm the desire is met to provide a view to both sides of the Afsluitdijk for cyclists. The strengthening of the outer slope is similar to alternative B1, resulting in a overtopping discharge in the order of 10 l/s/m. The slope at Lake IJssel side will remain the same. An overview is given in Figure 5.4.

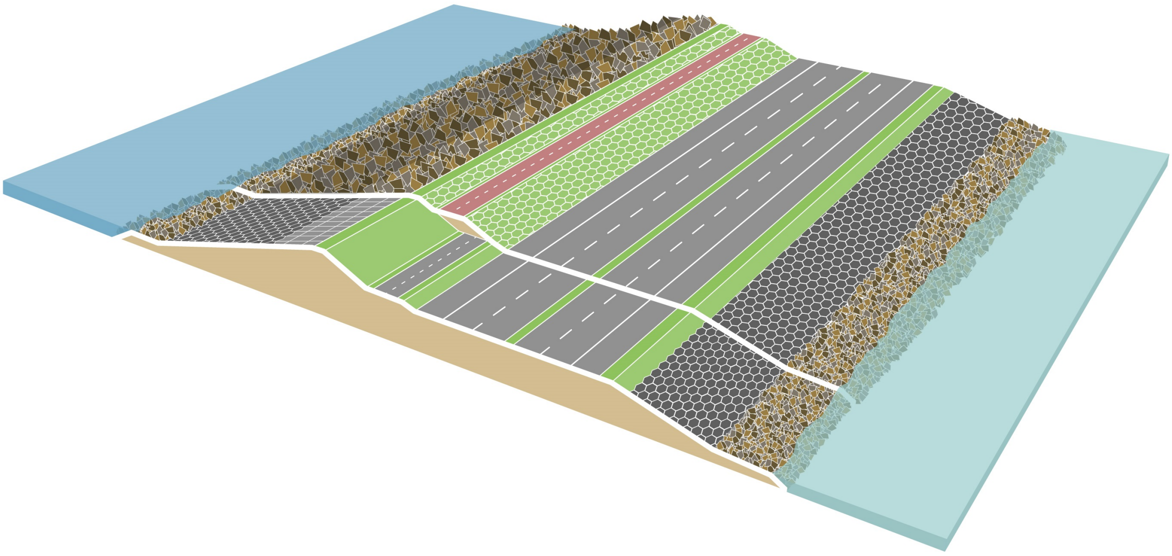

Figure 5.4: A schematic overview of alternative B2.

C1: Steepen and heighten the outer slope

This alternative is developed from the idea to provide a strong protection against the high wave impact of waves with a significant wave height (H_s = 3.9 m with a 1/10 000 per year probability). The outer slope revetment consists of concrete interlock elements which reduce the overtopping discharge to an order of 10 l/s/m. With this revetment type the outer slope can be steep enough (slope is 1:1.3) to prevent space occupancy of the Wadden Sea. Also the crown and the inner slope do not have to be adjusted. On the new crown a bicycle path is constructed with a small wall which is constructed as a separation between the interlocking elements and the bicycle path. The rest of the new crown will be covered with grass and the old bicycle path will remain the same. An overview is given in Figure 5.5.

D1: Adjusting the outer slope with a berm

A berm is an effective measure to reduce the wave run-up and overtopping. This berm will be constructed on top of the current outer slope which means that the new configuration will take up space in the Wadden Sea. In this alternative the protection of the other sloper will be completely new with a protection of concrete columns below the berm. Above the berm different lengths of the columns are used to increase the roughness against wave run-up and overtopping. On the berm a bicycle path is constructed to provide a view to the Wadden Sea. Because this bicycle may not be accessible all year round due to the fact that it is a possible breeding area for birds. Therefore the current bicycle path will remain intact. An overview of this alternative is given in Figure 5.6.

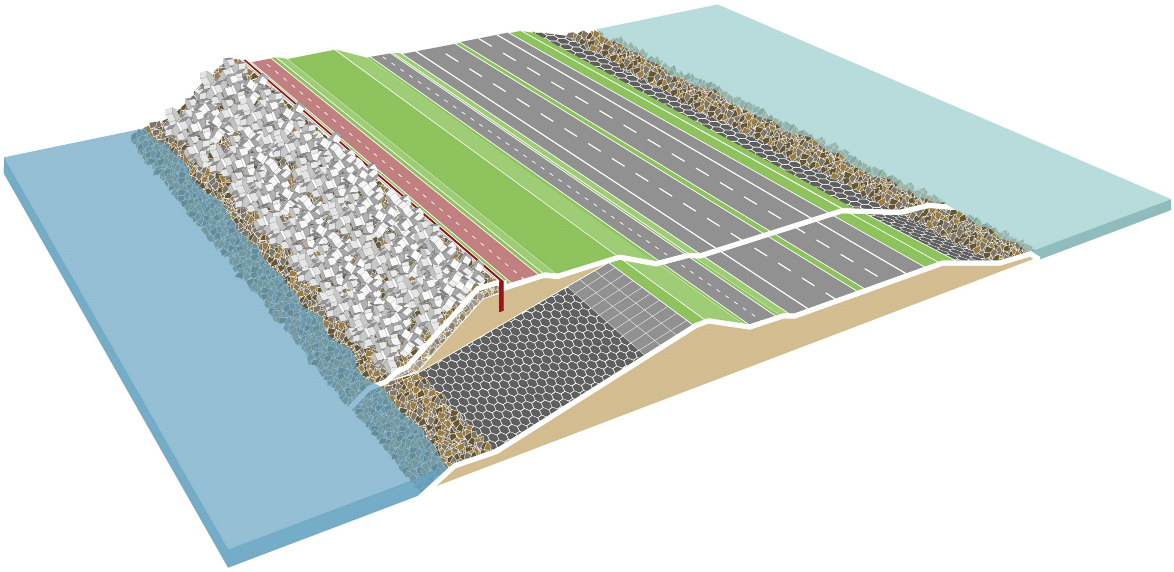

Figure 5.5: A schematic overview of alternative C1.

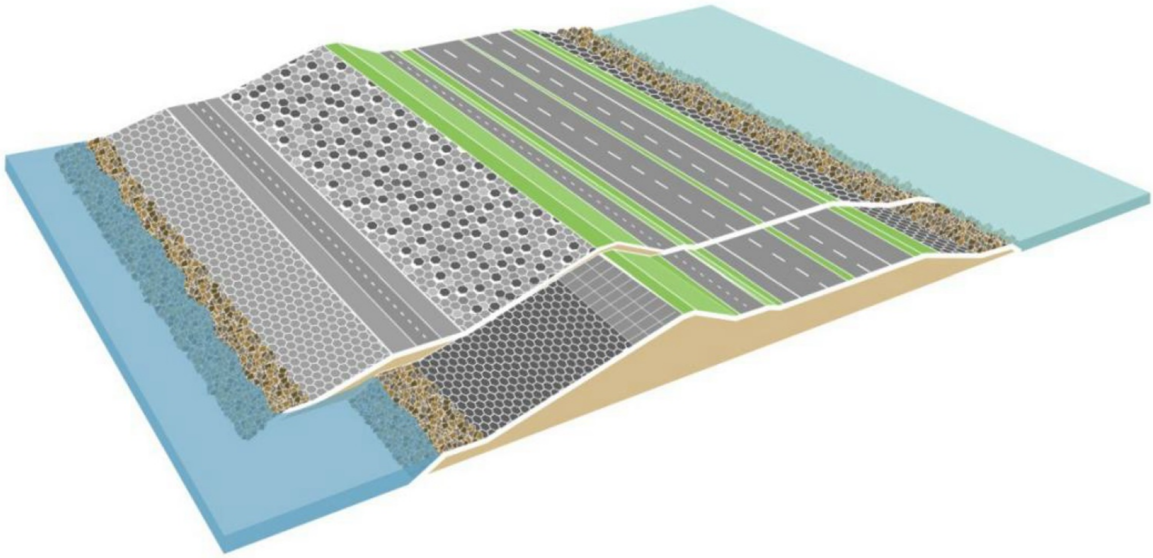

Figure 5.6: A schematic overview of alternative D1.

5.1.3. Evaluation alternatives

After an evaluation some decisions are made which reduce the solution space. One of these decisions is that the overtopping discharge may not exceed approximately 10 l/s/m. Further investigation is done at the moment to determine the the allowed overtopping discharge. Because alternative A1 has an asphalt layer on the outer slope this result in high overtopping discharges and a major improvement may be needed because this asphalt layer is not strength enough anymore in 2050. Because the large overtopping discharge results in improvements of the inner slope as well alternative A1 is not the cheapest solution. Based on these facts it is decided to reduce the allowed overtopping discharge to 10 l/s/m.

For the space occupancy in the Wadden Sea the solution space is reduced to an extra occupancy of the Wadden Sea of 7.5 meters into the Wadden Sea from the slope at a level of +0.5 m NAP. For the protection of the outer slope no explicit restrictions are introduced. The protection must provide stability until the year 2100 and the wave run-up and overtopping must be reduced to the above mentioned limit of 10 l/s/m.

Another decision is that the bicycle path must be on the North side of the A7. Also the solution for the increase of the safety of the dike must be North of the A7, so no major impacts on Lake IJssel side are introduced. With the reduction of the solution space some of the alternatives are dismissed. The reduction of the solution space results in the following conclusions:

- alternative A1 is excluded because the overtopping discharge is larger than 10 l/s/m and the asphalt layer is undesirable from an ecological point of view. Also the landscape value of this alternative is low compared to the other alternatives.
- alternative B1 is excluded because the bicycle path is located South of the A7 and space occupancy in Lake IJssel is needed. Also the high costs of the bicycle path are a reason to exclude this alternative.
- alternative B2 is a possibility keeping in mind the type of revetment with respect to the overtopping discharge and habitat.
- alternative C1 is a possibility keeping in mind the type of revetment with respect to the overtopping discharge and habitat.
- alternative D1 is a possibility but the expansion into the Wadden Sea may be a risk due to the Natura 2000 area[2].

To develop a new reference alternative one of the possible alternative is used as a starting point.The possible alternatives are alternative B2, C1 and D1. C1 is not favorable because it does not meet all the desires for the new design. alternative D1 has a high legal risk because of the Natura 2000 area. Therefore alternative B2 is chosen and further optimized in the reference alternative B3.

The optimization of alternative B2 is based on the reduction of overtopping and esthetic demands. Therefore the rubble mound cover is reduced until the design water level (MHW). The top of this rubble mound cover can be used as an inspection/maintenance road en it provides space for a possible bicycle path. Above MHW the current basalt cover must be replaced. To meet the desires a new place stone column revetment is chosen. To meet the maximum overtopping discharge possibly different lengths have to be used. A cross section of the reference alternative is given in Figure 5.7

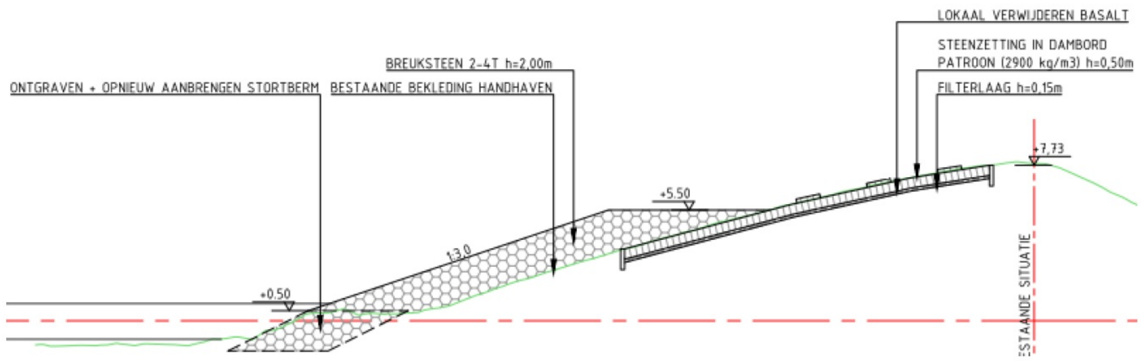

Figure 5.7: A cross section schematization of the reference alternative B3.

5.1.4. Further optimization reference alternative

At this moment the reference alternative is under development by W+B and the design is constantly changed. An overview of the current state of the design of alternative B3+ is shown in Figure 5.8.

[2]Natura 2000 areas are part of an European network of protected areas to improve nature, habitat and recreation in the Wadden Sea.

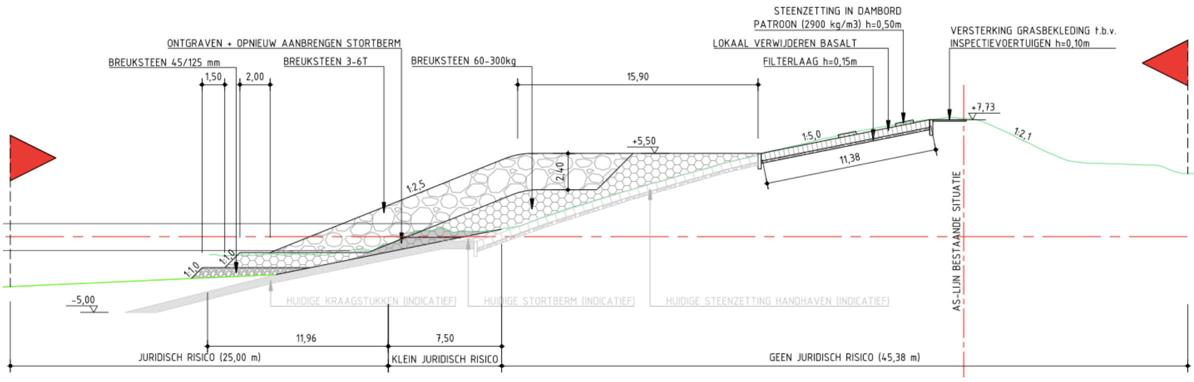

Figure 5.8: A cross section schematization of the reference alternative B3+.

In this reference alternative the basalt revetment at the upper part of the slope is replaced by placed stone blocks with different lengths to increase the friction. With the use of different lengths of the stones, also called a checkerboard pattern, the friction coefficient can drop to a value of 0.75 (instead of 1). At the lower part of the slope a berm is constructed of rubble on top of the existing dike protection. The height of the berm is located at 5.5m NAP and has a width of 16 meters. The friction of the rubble and the berm reduce the wave run-up and overtopping significantly.

The placed revetment consists of basalton blocks with a density of 2900 kg/m3 and with an average height of 0.5 m. The rubble placed directly on the existing slope consists of relatively small stones with weight class 60-300 kg and on top of that rubble with a class of 3-6 ton is used to withstand the wave impacts.

The solution space of the final alternative is bounded by the red arrows. This means that the redesign is focussed on improvement of the outer slope and minimum adjustments to the inner slope (up to the highway). The amount of space that may be occupied by the dike design in the Wadden Sea is limited due to legal bounds. In the optimization only the outer slope is adjusted so the limit for the solution is the left red arrow. This boundary is located at 32.5 meters from the current Afsluitdijk at 0 m NAP.

5.2. Probability of failure of reference alternative

The probability of failure of the reference alternative is assessed in the Matlab model developed in this MSc research and in PC-Ring. The results for the probability of failure for the failure mechanism of wave run-up and overtopping are shown in Table 5.1. The results are for the dike section 8B with a length of 4050 meters.

Model	P_f [1/year]	R [years]
Matlab	5.06E-05	19 763
PC-Ring	6.13E-05	16 301

Table 5.1: Probability of failure of the reference alternative for wave run-up and overtopping.

The standard that is used to assess the reference alternative is a probability of failure of 1/9 400 per year for the total Afsluitdijk. To translate this into a probability of failure for the cross section in this assessment and for the failure mechanism wave overtopping the following formula is used:

$$P_{eis,dsn} = \frac{P_{norm} \cdot \omega}{N}$$

in which $P_{eis,dsn}$ is the failure probability of failure for one failure mechanism and one cross section. The factor N is determined in paragraph 4.3.3 and is assumed to be 1.2 for the Afsluitdijk. The failure mechanism budget ω is according to the OI2014 24% (see paragraph D). For the Afsluitdijk these budgets are not applicable because the failure is mainly dominated by wave run-up and overtopping and piping does not occur. Therefore it is assumed that the failure mechanism budget for overtopping is equal to 50%. With these numbers the standard for the cross section is equal to:

$$P_{eis,dsn} = \frac{1/9\,400 \cdot 0.5}{1.2} = 1/22\,560 \approx 1/22\,500$$

The budgets for piping (see Table D.1 in paragraph 3.1) for the dike section and the hydraulic structures are added to the failure budget of overflow and overtopping resulting in the assumed value for ω of 50%. This assumption is an upper limit and further research is needed to see if this value is usable for the design of the Afsluitdijk. If the dike is reinforced and the probability of failure due to overtopping is reduced the value of this failure budget for overtopping is probably lower than the assumed upper limit of 50%. Therefore it is recommended to investigate the proposal for the failure budgets and determine new values for the Afsluitdijk for the final design of the renovation of the Afsluitdijk.

It is concluded that with the assumed values for N and ω the reference alternative is not safe enough according to the new standard. Therefore an optimization is done to find a alternative that is safe enough. Also the costs are minimized in this optimization to find a possible alternative that may be cheaper than the reference alternative. The standard for a single cross section with a probability of failure of 1/22 500 per year is also used in the optimization of the reference alternative.

5.3. Optimization for the design of the Afsluitdijk

The optimization of the alternative is only done for the mechanism wave run-up and overtopping. The class of the rubble and the dimensions of the basalton blocks are set at the given values in the reference alternative and are not investigated further. The stability of the revetment is however of importance for the final design, so the final optimal design must be checked for stability of the revetment as well.

In PC-Ring the cross section of this alternative is calculated to see which influence factors have the most impact on the total probability of failure. A schematization of the cross section is shown in Figure 5.9 and the values for the input parameters are shown in Table 5.2.

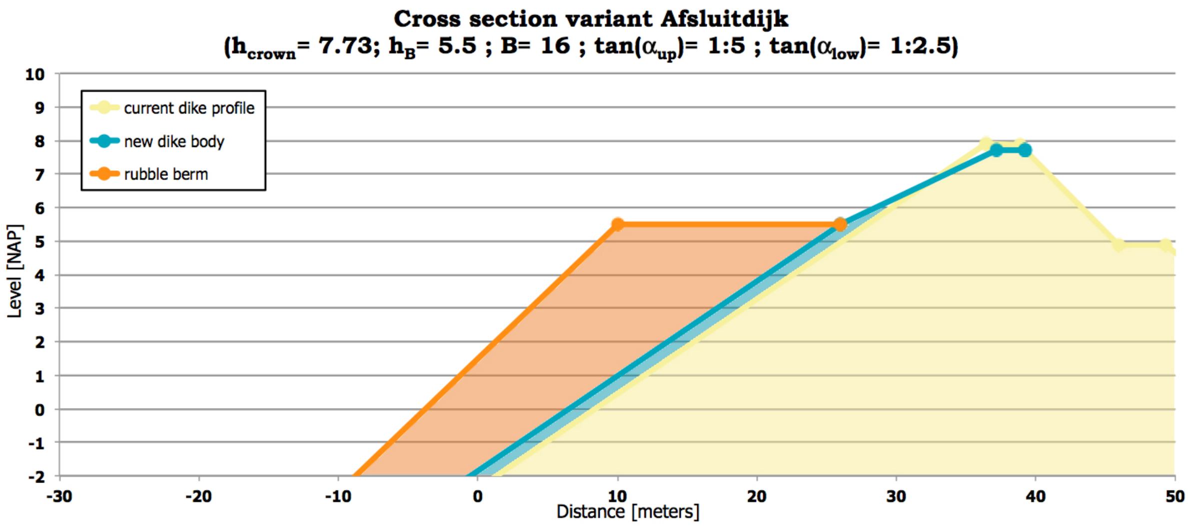

Figure 5.9: Schematization off the cross section of the reference alternative B3+. The varying variables for the optimization with the most influence on the probability of failure are based on this design.

Symbol	Unit	Name	Mean value	Spread
$tan(\alpha_{upper})$	–	upper slope	1:5	V = 0.05
$tan(\alpha_{lower})$	–	lower slope	1:2.5	V = 0.05
H_b	m	berm height	5.5	σ = 0.1
B	m	berm width	16	σ = 0.15
H_{crown}	m	crown height	7.73	σ = 0.1
$\gamma_{f,up}$	–	friction coefficient upper slope	0.75	–
$\gamma_{f,low}$	–	friction coefficient lower slope	0.55	–

Table 5.2: Parameters used to determine failure due to overtopping for the reference alternative.

With the results from a calculation in PC-Ring it is observed that the following variables have the most influence on the probability of failure for the failure mechanism wave run-up and overtopping:

- Height of the crown of the dike
- Height of the berm
- Width of the berm
- Angle of the slope above the berm
- Angle of the slope below the berm

To find an optimal design the values for these five variables are varied to find an optimal design. To see what the impact is of each variable on the probability of failure the reference alternative is used as a starting point. For each variable the influence on the probability of failure is shown in Figures 5.10, 5.11, 5.12 and 5.13. In these figures the value of the variables is shown on the x-axis and the return period (inverse of the probability of failure) is shown at the y-axis.

As we can see in Figure 5.10 the probability of failure is as suspected (see Formula B.19 in Appendix B) exponentially related to the height of the dike. The return period on the y-axis is plotted on a logarithmic scale to show this exponential relationship. In the optimization the height of the dike is varied from 7 to 8.5 meters with a step size of 0.25 m. These limits are chosen in such a way that the proposed safety standard of 1/9400 per year can be met (with the variation of other variables) and that the total costs do not reach extreme values (which is the case for large dike crown heights).

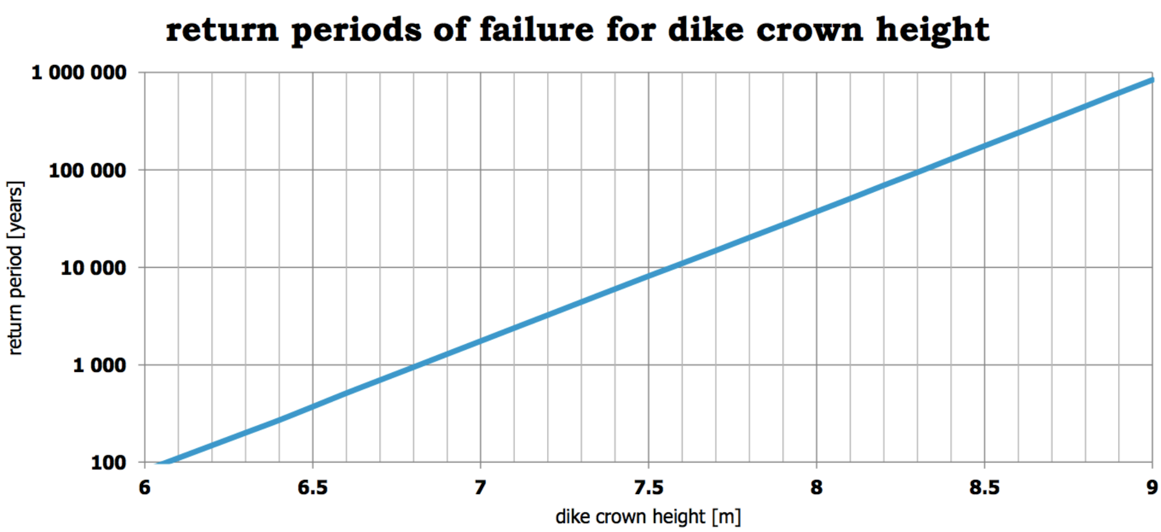

Figure 5.10: Return periods of failure for different values for the height of the crown of the dike.

In Figure 5.11 the influence of the berm height on the probability of failure is shown. A clear relationship is not found but it can be concluded that there is an optimal berm height. In the case of the reference alternative this optimal berm height is around 4 meters. This optimal berm height depends on the other variables as well therefore the height of the berm is varied from 2 to 6 meters with a step size of 1 meter.

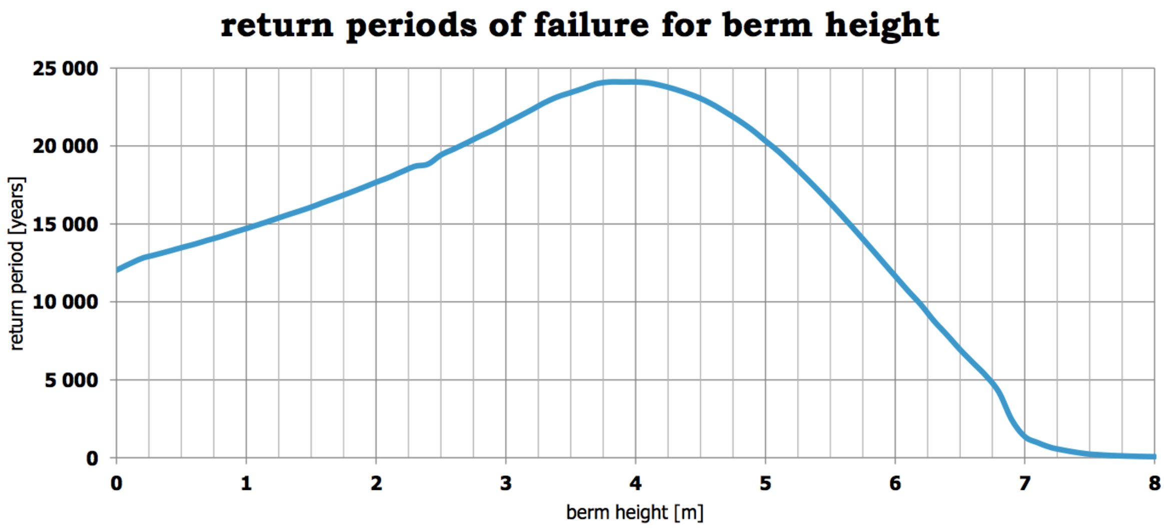

Figure 5.11: Return periods of failure for different values for the height of the berm.

In Figure 5.12 the influence of the width of the berm is shown. The probability of failure reduces with an increasing berm width until a certain point is reached where the width of the berm has no more influence on the probability of failure. This is around 20 meters in case of the reference alternative. The same as with the berm height this optimal width depends on the other variables as well. Because there is limited solution space for the design of the Afsluitdijk the maximum berm width is set at 25 meters. In the optimization it is varied from 0 meters to this maximum value of 25 meters with a step size of 5 meters.

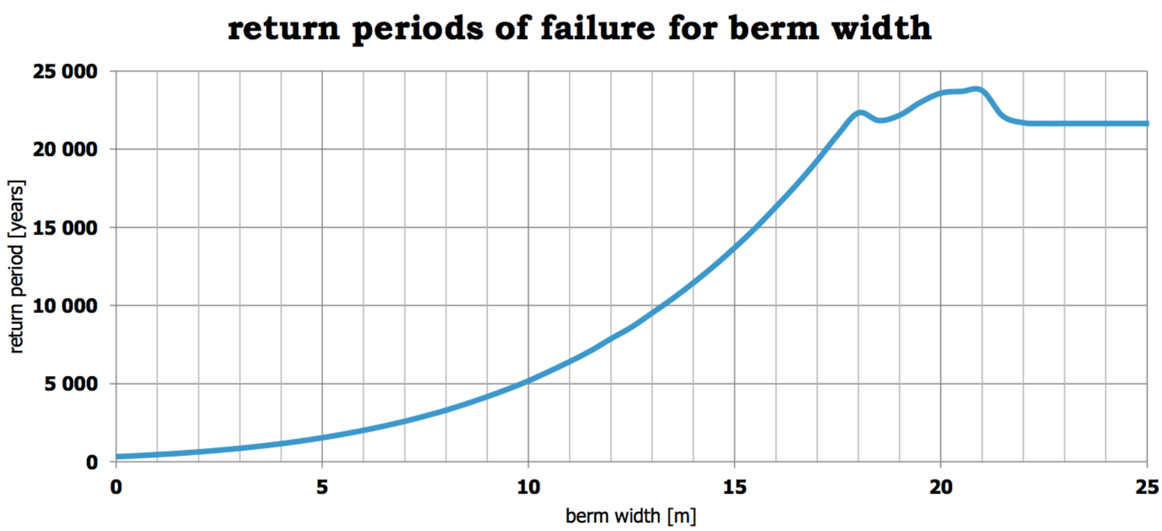

Figure 5.12: Return periods of failure for different values for the width of the berm.

A decrease of the slope results in lower probabilities of failure. This is both the case for the slopes below and above the berm, as shown in Figure 5.13. Both the x- and y-axis are plotted on a logarithmic scale. A very gentle slope results in a small probability of failure but the costs increase significantly with a decreasing slope. Therefore the upper limit of the slope is set at 1:6 in the optimization. The lower limit is set at 1:2 and in this dimension of 1:x for the slope the step size is set at x=1.

An overview of the range, step size and total number of values for each variable is shown in Table 5.3.

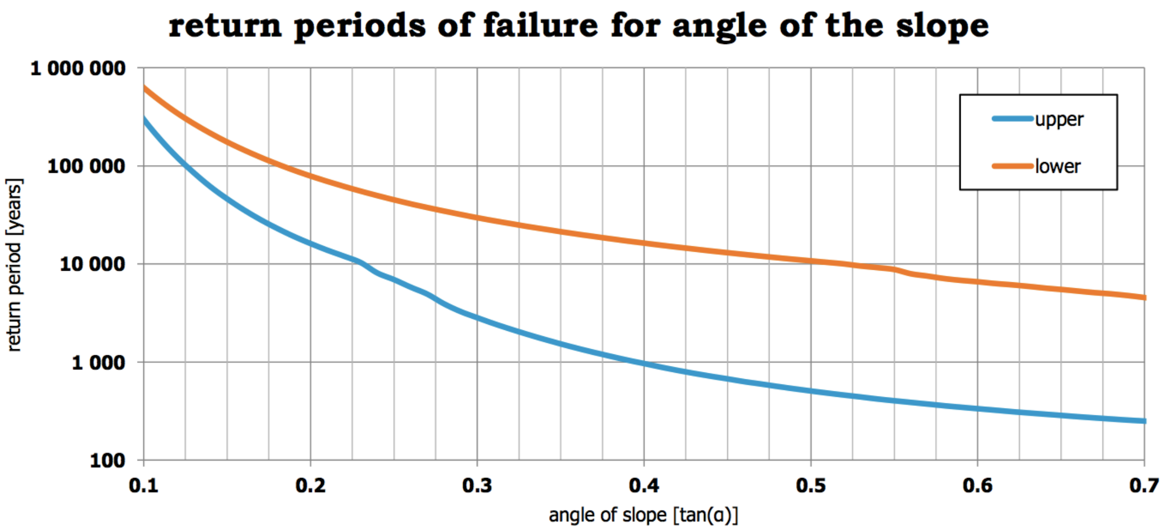

Figure 5.13: Return periods of failure for different values for the angle of the upper and lower slope.

Variable	Start value	End value	Δ step	Number of steps
dike height	7	8.5	0.25	7
berm height	2	6	1	5
berm width	0	25	5	6
upper slope	2	6	1	5
lower slope	2	6	1	5

Table 5.3: Steps for the variables used in the optimization for overtopping.

The total number of calculations that is done in this optimization equals the product of the total number of values for each variable:

$$N_{calculations} = 7 \cdot 5 \cdot 6 \cdot 5 \cdot 5 = 5\,250$$

In PC-Ring calculation with varying values is only possible for one variable. The results of such a calculation are given in a text file which has to be converted into a spreadsheet format, in which the total costs are calculated (see paragraph 5.3.1 for a description of the cost estimation). This means that a large amount of manual labour is needed to get results from PC-Ring. Also the calculation time in PC-Ring for a single calculation is around 4.5 minutes which is not an extra advantage compared to the calculation time with the Matlab model.

The model developed in this MSc research for the failure mechanism wave run-up and overtopping is used in the optimization. The formulas and the method of this model are described in Appendix B and the Matlab script for this calculation is given in Appendix E. In Matlab the five earlier mentioned variables are defined as changing values, resulting in an output spreadsheet consisting of 5 250 values for the probability of failure. Each individual calculation is done with the Monte Carlo method with 1 000 000 samples used in each calculation. For accurate results with the Monte Carlo method the minimum amount of samples is equal to:

$$N_{calculations} = \frac{1}{P_f \cdot 400}$$

If we take probabilities of failure into account of 1/20 000 this would lead to a minimum amount of samples of 8 000 000. To save calculation time the amount of samples is chosen at 1 000 000 which results with an average calculation time of 50 seconds in a total calculation time of 262 500 seconds.

5.3.1. Cost estimation alternative

The alternative is optimized with a cost minimization. For the alternatives that meet the standard the costs are estimated with the cost numbers given in Table 5.4. These numbers are rough estimates for the costs given by Witteveen + Bos. These numbers are the so-called direct costs that consist of the labour, machinery and materials. To calculate the building costs and the total investment costs the direct costs are multiplied with the factors given in Table 5.4.

Description costs	Value	Unit
remove basalt and re-use in work	5	€/ton
transport and apply dike clay	25	€/m^3
transport and apply granular filter	25	€/ton
transport and installation concrete columns (gamma = 2 900 kg/m^3 h=0.5 m)	160	€/m^2
transport and installation rubble (class 10-60 kg up to 1-5 ton)	25-30	€/ton
factor for building costs	1.52	[-]
factor for investment costs (including engineering and overall project costs)	2.3	[-]

Table 5.4: Cost numbers for the cost estimation of the alternatives.

With these numbers the direct costs are estimated. The following items are calculated per unit of length for each alternative to estimate the total costs:

- Area of removing old basalt blocks [m^2]
- Volume of dike clay that is placed [m^3]
- Area for placement of granular filter [m^2]
- Area for placement of concrete columns [m^2]
- Volume of rubble that is placed [m^3]

To estimate the costs some of the costs numbers are transformed into the units that are given above. The steps that are needed in these transformations are the density and in some cases the height of the different variables. The costs numbers and the transformations for all the values are given in Table 5.5.

Description costs	Value	Unit	Density [ton/m^3]	Height [m]	Value	Unit
basalt	5	€/ton	2.5	0.5	6.25	€/m^2
dike clay	25	€/m^3	-	-	25	€/m^3
granular filter	25	€/ton	2	0.15	7.5	€/m^2
concrete columns	160	€/m^2	-	-	160	€/m^2
rubble	25-30	€/ton	2	-	50-60	€/m^3

Table 5.5: Transformed cost numbers for the cost estimation of the alternatives.

In the optimization of the alternative only the investment costs are considered. Interest, maintenance costs and present value are all factors that are not taken into account in this optimization. The aim is to find a cost effective design with the lowest investments costs at this moment. Also the consideration between investment right now and investments in the future is not taken into account. In reality all these factors have an impact on the choice of the alternative, so it is recommended to take this into account before the final design is chosen.

The calculated values for areas and volumes that determine the costs are based on the adaption of the current profile of the Afsluitdijk and the defining parameters of the alternative, which are the dike crown height, the berm height, the berm width and the angle of the upper and lower slope. In the calculation of these areas and volumes the following things are taken as assumptions:

- The width of the crown of the dike is at least 2.5 meters.
- The inner slope is not changed.
- The dike core (clay) has an angle of 1:3.5 below the berm height (which is the current slope).
- The old basalt blocks are removed from the berm height up to the top.

- If extra clay has to be placed below the berm the old basalt below the berm is removed (in case of high dike crowns and/or a very gentle upper slope).

To show the impact of these assumptions two schematic cross sections are shown in Figure 5.14 and 5.15, in which the old dike profile is shown in yellow, the new dike profile in turquoise and the rubble berm in orange. These schematizations are not realistic alternatives but they show the impact of the assumptions that are used in the definition of the cross section.

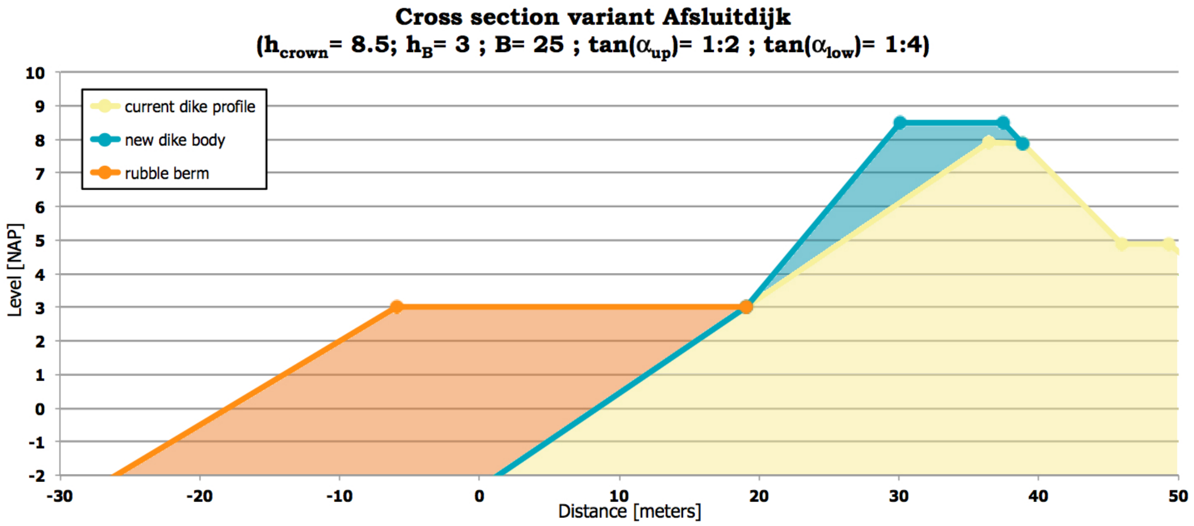

Figure 5.14: Schematization of the cross section of a possible alternative to show the impact of the different values for the variables.

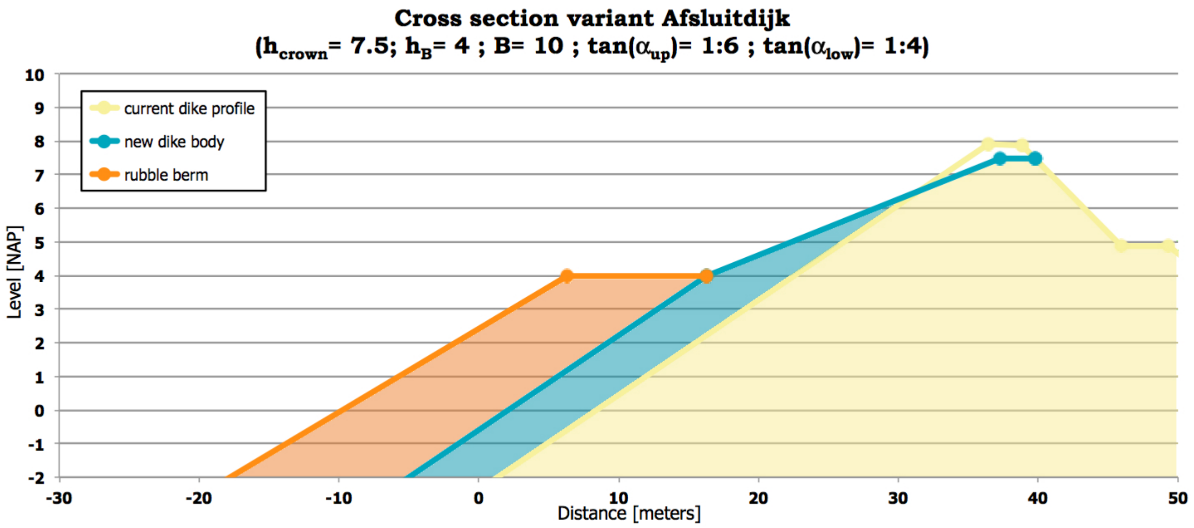

Figure 5.15: Schematization of the cross section of a possible alternative to show the impact of the different values for the variables.

5.3.2. Optimization alternative

To find an optimal design the aim is to minimize the costs for designs that meet the safety standard. The location of the cross section that is optimized is dike section 8B. The optimization is done for the failure mechanism of wave overtopping, because this has the largest influence on the total probability of failure. The designs that follow from the optimization can result in unstable designs or designs with insufficient strength of the revetment. Therefore only realistic designs are taken further into account.

In the optimization the same hydraulic loads from the assessment of the probability of failure of the Afsluitdijk are used. The design lifetime for the renovation of the Afsluitdijk is several decades so these hydraulic loads are probably larger than the current loads. This is due to the climate change which causes a rise of the sea level and increasing wave loads. In the development of the reference alternative by W+B the W+ scenario[3] is used, which predicts a sea level rise between 45 and 80 centimeter in 2085. In this thesis these adjusted hydraulic loads are not used, so the results obtained in this thesis are not completely accurate. Therefore it is recommended that in a further optimization in which other failure mechanisms are taken into account as well the increase of the hydraulic loads is taken into account as well.

All the alternatives with a probability of failure that is higher than 1/22 500[4] are deleted from the selection. For the remaining possible alternative the costs are estimated and the alternatives with the lowest costs are evaluated. As stated before some combinations of variables lead to unrealistic designs if we look at the failure mechanisms of macro instability and failure of the rubble and placed stones. The ten realistic alternatives with the lowest costs are shown in Table 5.6, with the variables, the return periods and the investment cost estimation. On the bottom of this Table also the results for the reference alternative are shown to compare it with the optimization.

h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
8.5	4	5	3	4	22 727	12 747
8.5	5	5	3	4	25 000	12 851
8.5	6	5	3	4	30 303	13 052
8.5	6	10	3	3	30 303	13 932
8.25	6	5	4	4	34 483	13 951
8	4	5	3	5	24 390	14 143
8.5	5	10	3	3	25 641	14 145
8.5	4	10	3	3	32 258	14 317
7.75	6	5	6	4	41 667	14 371
8.25	4	5	3	5	26 316	14 483
7.73	5.5	16	5	2.5	19 763	18 890

Table 5.6: Cost estimation for the ten alternatives with minimal costs.

The investment costs per unit of length given in Table 5.6 are based on the adaption of dike section 8B. To calculate the total investment costs for the Afsluitdijk it is not possible to multiply these costs with the total length of the Afsluitdijk. This is because for each dike section an optimization of the cross section results in other values for the different variables. However this optimization gives an idea of the amount of money that can be saved.

With the results from the optimization some conclusions are drawn with respect to the reference alternative. The crown height of the dike is for the top ten alternatives in the optimization higher than the reference alternative. A higher dike crown results in lower probabilities of failure but more investment costs are needed. These investments costs can be saved by reducing the other variables, like the width of the berm (which saves investment costs on rubble). Therefore a consideration is needed between influence on the probability of failure of the different variables and the corresponding investment costs.

The height of the berm in the top ten of the optimization varies between 4 and 6 meters. Compared to the reference alternative with a berm height of 5.5 meters this is in the same range. For each design an optimal berm height can be estimated in terms of the lowest probability of failure. However the investment costs are also needed in the determination of the optimal design. The investments costs strongly depend on the costs for the hard dike protections (rubble and basalton blocks). The berm height influences both these variables because the height of the berm determines the amount of rubble and basalton blocks that are needed. A higher berm leads to more investment costs for the rubble but

[3]The W+ scenario is described by the Royal Netherlands Meteorological Institute and it is one of four scenarios that predict future weather conditions.
[4]This standard is determined in paragraph 5.2

for lower investment costs for the basalton blocks. A clear conclusion on the berm height can not be drawn from these results.

The width of the berm is for all the optimal alternatives lower than the reference alternative. A conclusion is that lower probabilities of failure are reached with a smaller berm, which is compensated by the other variables (for instance a higher dike crown). The investment costs for rubble have a large influence on the total investment costs so smaller berms result logically in lower investment costs. However investigation on the stability of the rubble is needed for smaller berms.

The investment costs for the hard dike protection depend besides the berm height on the angle of the slopes as well. Compared to the reference alternative two differences are noticed. In the optimization the upper slopes are (most of the time) steeper and the lower slopes are more gentle. The steeper upper slopes can result in instability of the basalton revetment, so it is recommended to investigate this before a definite design is chosen. The lower slopes are in the optimal designs gentler which results in a more stable situation. However a more gentler slope results also in higher investment costs because more material is needed.

Another option is to investigate the possible alternatives with relatively low probabilities of failure and lower investment costs than the reference alternative. From all the possible alternatives with lower investments costs than the reference alternative and with return periods that are twice as high as the standard (1/45 000) the top ten is shown in Table 5.7.

h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
8.5	6	5	4	4	50 000	15 379
8.25	5	5	3	5	45 455	15 553
8.5	2	5	3	6	62 500	15 628
8.5	4	5	3	5	62 500	15 663
8	3	5	3	6	100 000	15 712
8.25	3	5	3	6	83 333	15 855
8.5	5	5	4	4	55 556	16 008
8.25	6	5	5	4	111 111	16 042
8	6	5	6	4	62 500	16 241
7.5	3	5	4	6	45 455	16 453
7.73	5.5	16	5	2.5	19 763	18 890

Table 5.7: Cost estimation for the ten alternatives with return periods higher then 45 000 years and lowest costs.

Compared to the results from the optimization with minimal costs no clear differences are depicted, except for the width of the berm. In these results the berm widths are all 5 meters but this can result in instability of the rubble, so extra investigation is needed. However we see that still for smaller amounts of money a lot higher safety levels are reached, up to a factor of 5 times as safe as the reference alternative.

5.4. Optimal design recommendation

Taking the results of the optimization into account an optimal design is recommended. First of all from the results it is concluded that a higher dike crown has a major influence on the lowering of the probability of failure. Therefore a higher dike crown is recommended. One of the desires for the design of the Afsluitdijk is that it has a uniform design all along the complete dike body. Some dimensions can vary slightly along the dike body for different dike sections but major differences are not desirable. For the dike crown height a variation along the dike does not have a major impact on the character of the Afsluitdijk so a higher dike crown height, at least for dike section 8B, is recommended.

With a higher dike crown heigh the width of the berm can also be reduced compared to the reference alternative. The reduction of the berm width results in lower investment costs, however the stability of the rubble must be guaranteed. Also a wider berm results in lower probabilities of failure. As an average of the first optimization the berm width is set at a minimum value of 7.5 meters. Also a more gentle lower slope results in lower probability of failure. At the same time a more gentle slope results in more stability of the rubble.

Keeping this in mind an extra optimization is done with smaller step sizes and smaller ranges for the different variables. The values and step sizes are shown in Table 5.8.

Variable	**Start value**	**End value**	**Δ step**	**Number of steps**
dike height	8	8.25	0.25	2
berm height	4.5	5.5	0.5	3
berm width	7.5	12.5	2.5	3
upper slope	4	5	0.5	3
lower slope	3	5	0.5	5

Table 5.8: Steps for the variables used in the detailed optimization for overtopping.

In Table 5.9 the five alternatives with realistic designs and the lowest costs are shown. The results of these further optimization are higher than the optimized alternative in the previous paragraph. This is partly because the minimum berm width is set at 7.5 meters in stead of 5 meters which results in extra costs.

h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
8	5	7.5	4.5	3.5	31 250	15 160
8.25	5	7.5	4	3.5	37 037	15 227
8	5	10	4	3.5	34 483	16 409
8	5	7.5	5	3.5	34 483	16 502
8.25	5	7.5	4.5	3.5	38 462	16 689
7.73	5.5	16	5	2.5	19 763	18 890

Table 5.9: Cost estimation for five realistic alternatives with the lowest costs.

With all the results taken into account it is concluded that there is still room for improvement on the reference alternative if only the failure mechanism of wave run-up and overtopping is taken into account. To make sure this optimal design is safe enough further investigation on the stability of the rubble and the basalton revetment is needed. Also a final check on the macro stability is needed before this optimized alternative is definitely safe enough for the standard of 1/9 400 per year.

This safety standard, proposed by a study by the CPB, is as stated before unsure and it remains unclear what the safety standard is going to be in the future. The reference alternative and the new design are based on the old standard, which prescribes that the Afsluitdijk has to able to withstand a storm with an exceedance frequency of 1/10 000 per year. If this standard is used as a starting point the design is most likely safe enough for the proposed standard with a probability of failure of 1/9 400 per year. However this standard may be lower or higher in the future.

Besides the uncertainty in the new safety standard there is a lot of uncertainty in the determination of the critical overtopping discharge. The strength of the inner slope, the pulsating character of the overtopping waves and non water retaining objects all have influence on the value for the critical overtopping discharge. Standards for this critical overtopping discharge are only for dike with significant wave heights up to 3 meter. The Afsluitdijk is attacked by even higher waves so these standards do not apply to the Afsluitdijk. In Figure 5.16 the sensitivity of the critical overtopping discharge is shown for the reference alternative.

From this Figure it is clear that if a higher value is 'chosen' as a standard this results in lower probability of failure. Therefore the uncertainty in the determination of the value of the critical overtopping discharge in the future also implies uncertainties. As the failure mechanism wave run-up and overtopping has the largest influence on the probability of failure of the Afsluitdijk this uncertainty is an issue that can not be neglected.

In the optimization a value of 10 l/s/m is used, but in an earlier MSc thesis on the Afsluitdijk (Landa, 2014) it is stated that the Afsluitdijk is able to withstand overtopping discharges up to 30 l/s/m. From

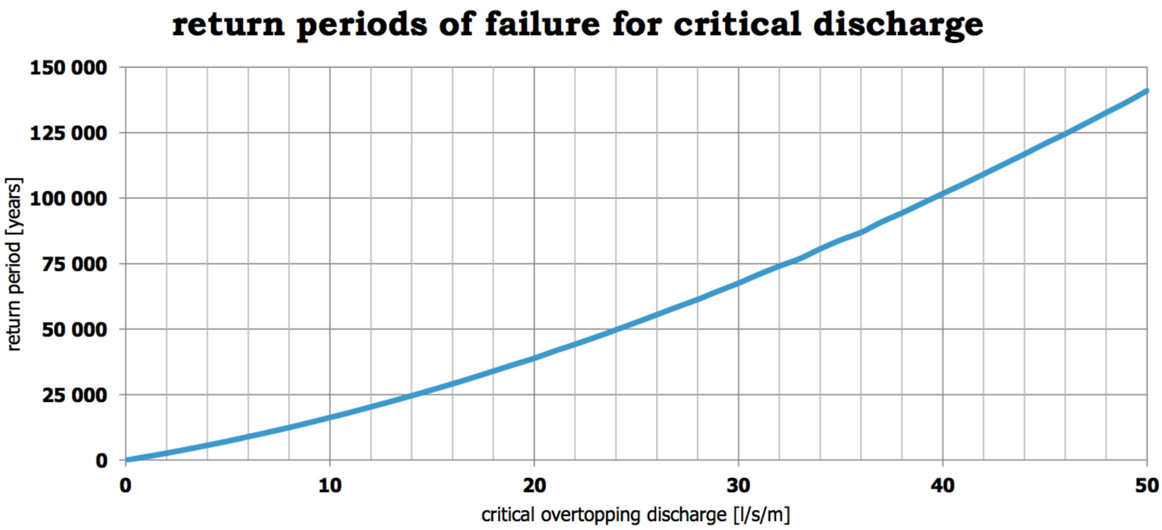

Figure 5.16: Return periods of failure for different values for the height of the berm.

a sensitivity analysis in this MSc research it is clear that this can result in a probability of failure which is four times lower. The uncertainty in the determination of the critical overtopping can have a major influence on the design of the Afsluitdijk which must be kept in mind during the development of a design for the Afsluitdijk. Also the value of the critical overtopping discharge where initial damage occurs is lower than the value where a breach starts to develop. The choice for the allowable amount of damage therefore also has an influence on the critical overtopping discharge that is used in design and assessment standards. Therefore it is recommended to investigate the critical overtopping discharge further.

With these uncertainties in mind it is recommended to make an easily adaptable design that is not safer than required at this moment. This is recommended to save money on the strengthening of the Afsluitdijk at this moment. It is possible that the new safety standard will be lower than the recommended probability of failure of 1/9 400 per year or that the critical overtopping discharge, which is now set at 10 l/s/m, may be higher after new insights and/or research. The design however must be easily adaptable and improvements with low investment costs but a high influence on the lowering of the probability of failure are needed to be ready for future changes.

The design of the reference alternative is a good example of an adaptable alternative, in which the height and the width of the berm are easily adaptable by adding more rubble. Also the lower slope is easily adaptable. Heightening of the dike crown or adjustments to the upper slope are a lot harder with this design. Changing the upper slope or the height of the dike crown results into changes to the lower part as well. If this is done the rubble has to be removed before changes can be made which will cost extra money.

In the design of the renovation of the Afsluitdijk the sustainability and the landscape value also play an important role. In this thesis these factors are not taken into account because the concept of the alternative is the same as the reference alternative B3+ developed by W+B. For the final design it is recommended to investigate these factors as well to guarantee a sustainable design that 'fits' in the landscape as well.

For the design of the Afsluitdijk it is recommended to make a design with a higher dike crown height of at least 8 meter. This is because it is harder to change after the rubble is placed in front of the dike. Because the dike is higher a steeper upper slope compared to the reference alternative can be used (1:4.5). Furthermore it is recommended to place rubble with a berm at + 5 m NAP with a width of 7.5 meters and a lower slope of 1:3.5. A cost estimation of the alternative is shown in Table 5.10 and a schematization of this alternative is shown in Figure 5.17.

h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
8	5	7.5	4.5	3.5	31 250	15 160
7.73	5.5	16	5	2.5	19 763	18 890

Table 5.10: Cost estimation for the recommended alternative and the reference alternative.

The safety in the future is increased by changing one or more of the following variables:

- berm height
- berm width
- lower slope

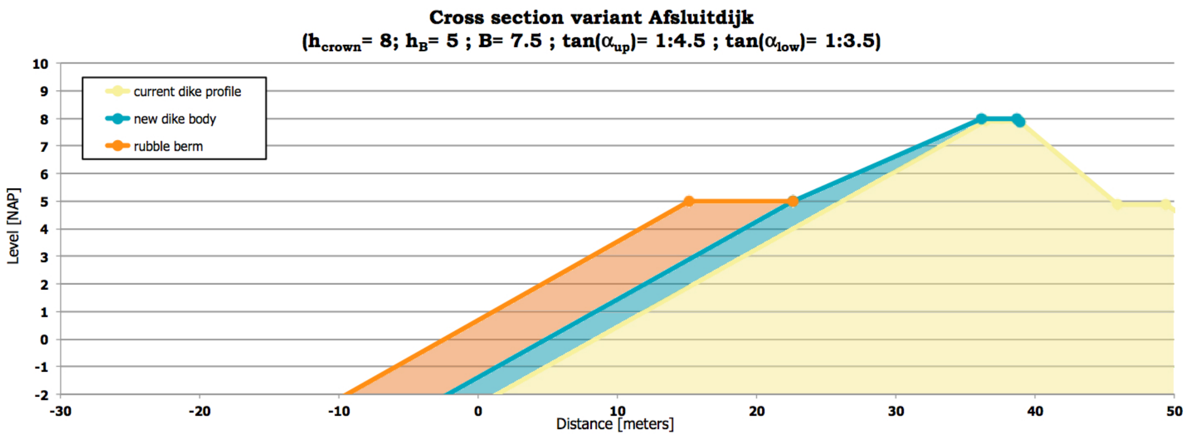

Figure 5.17: Schematization of the cross section of the recommended adaptable alternative.

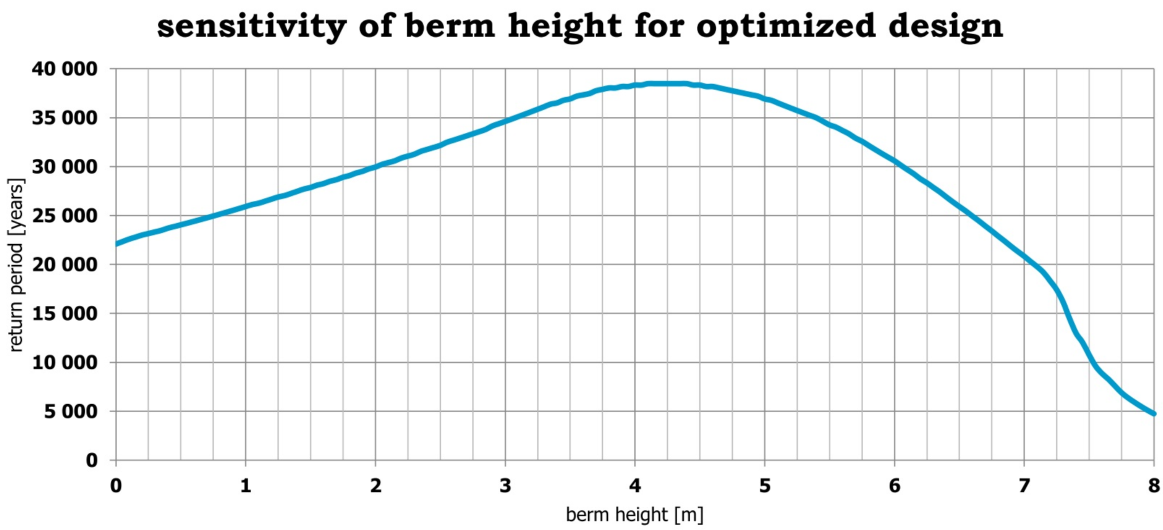

Figure 5.18: Return periods of failure for different values for the height of the berm for the recommended alternative.

In Figure 5.18 the influence of the berm height on the total probability of failure for the recommended design is shown. It can be concluded that increasing the berm height causes a decrease of the safety, so this is not favorable. Although the optimal berm height in terms of safety is around + 4 to + 4.5 m NAP a height for the recommended alternative is chosen at 5 m. This is done because this is cheaper (basalton revetment above the berm is more expensive than rubble) and it is effective against sea level rise. The efficiency of a berm depends on the still water level during a storm event, so taking the future sea level rise into account a higher berm is more effective.

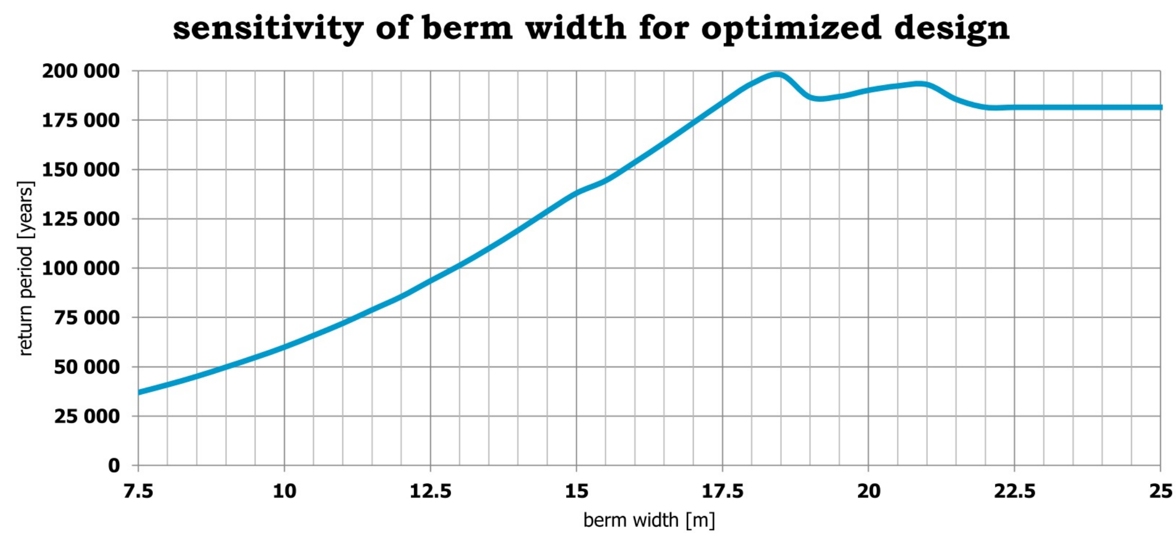

Figure 5.19: Return periods of failure for different values for the width of the berm for the recommended alternative.

In Figure 5.19 the influence of the berm width on the total probability of failure is shown. Widening of the berm further than 18.5 meters is not effective anymore, as shown in this Figure. Investing in extra berm width up to this limit results in significant lower probabilities of failure.

In Figure 5.20 the influence of the angle of the lower slope on the total probability of failure is shown. A more gentle slope, up to a slope of 1:10, results in significant decreases of the probability of failure.

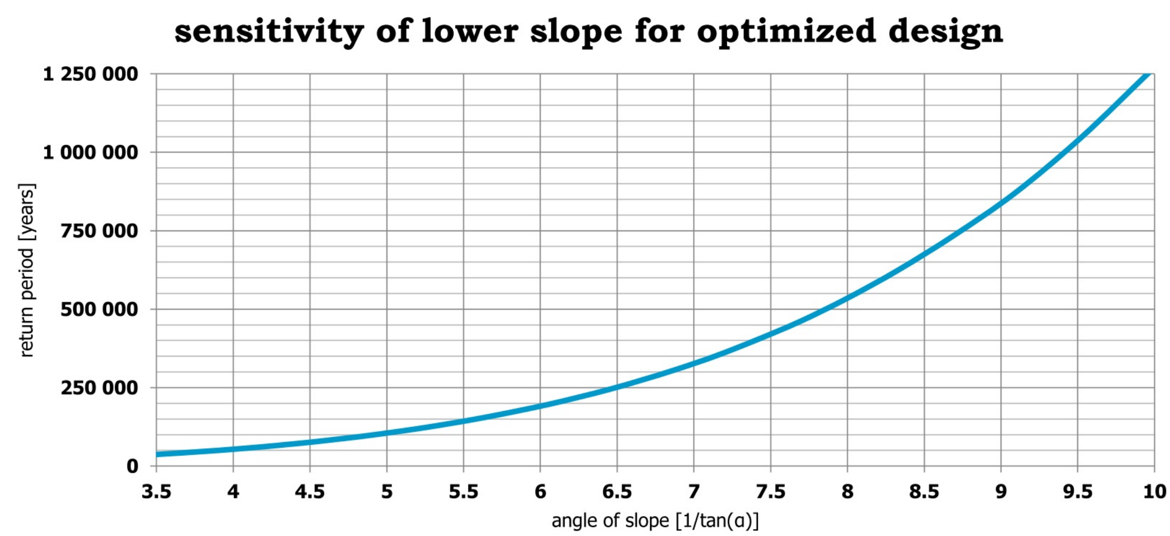

Figure 5.20: Return periods of failure for different values for the lower slope for the recommended alternative.

It can be concluded that only two measures are effective in reducing the probability of failure of the recommended design. Widening of the berm and decreasing the angle of the lower slope are two solutions that are effective. In Figure 5.21 the investment costs for the extra rubble are shown for both the solutions.

To compare how effective these measures are in terms of investment costs the costs of extra rubble for both the solutions are plotted against the return period of failure, shown in Figure 5.22. From this Figure it is concluded that both the solutions of widening of the berm and decreasing the lower slope have the same efficiency until the effect of widening of the berm is gone.

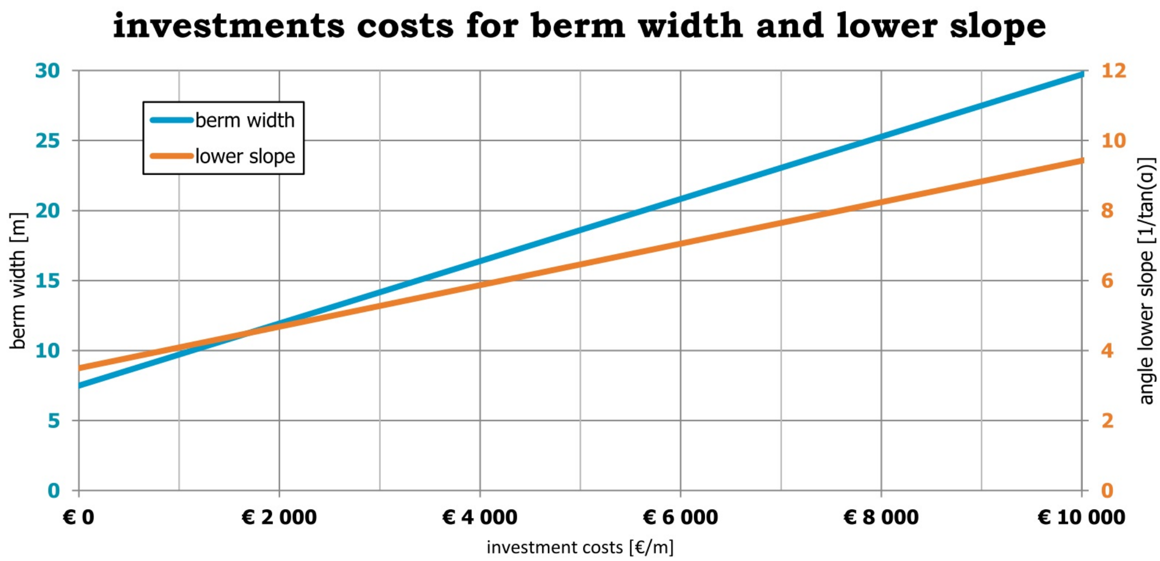

Figure 5.21: Investment costs for increasing berm width and making the lower slope more gentle.

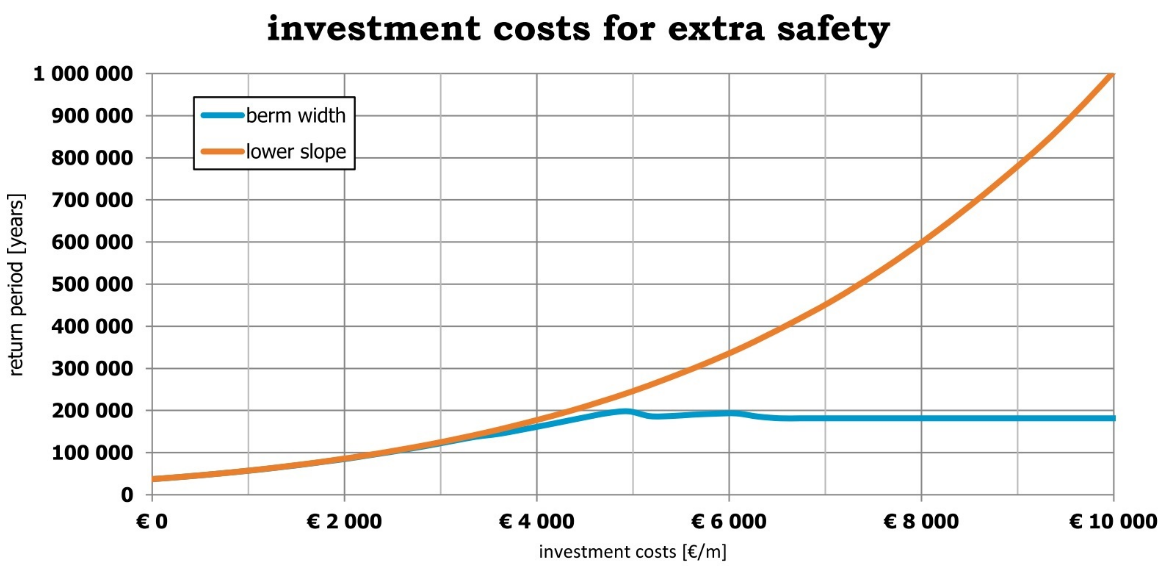

Figure 5.22: Effect of the investment costs on the return period of failure.

To see if both measures are evenly effective we zoom in on the lower left corner of the previous Figure. In this enlarged plot, shown in Figure 5.23 both the graphs for the widening of the berm and the decrease of the lower slope follow more or less the same line. Therefore it is concluded that both measures have the same efficiency up to the point of an extra investment of €3 000. If the probability of failure has to be reduced to a value lower than 1/125 000 per year it is more effective to invest in a decrease of the lower slope.

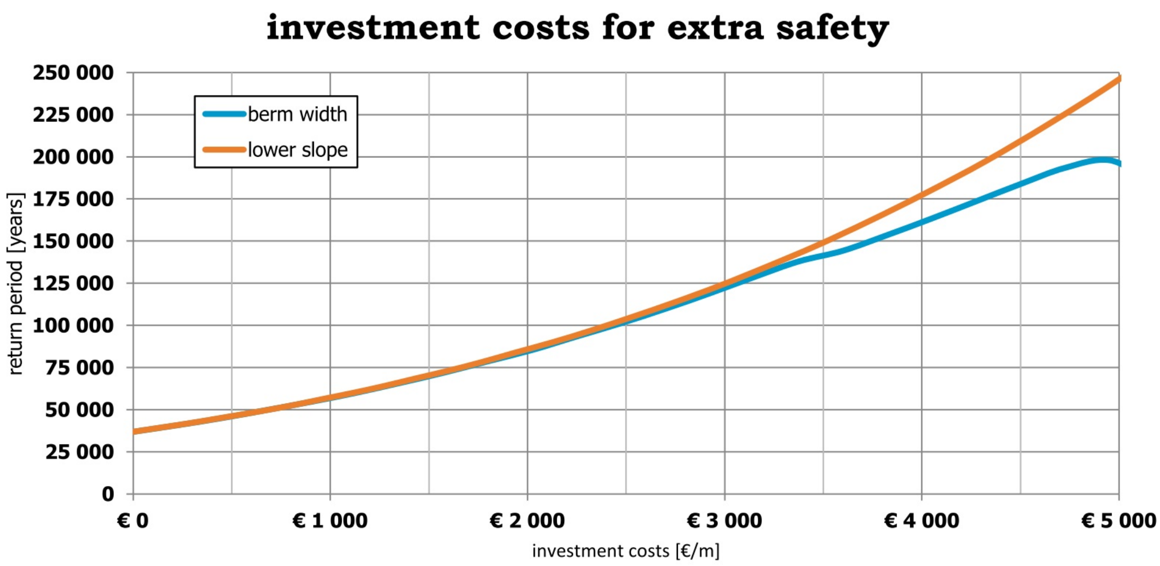

Figure 5.23: Zoomed effect of the investment costs on the return period of failure.

6

Conclusions, recommendations and discussion

The research question for this MSc thesis is formulated as follows:

"*What is a probabilistic, cost-effective and adaptable design for the renovation of the Afsluitdijk?*"

In this chapter conclusions are drawn from the obtained results to answer this research question. Following from these conclusions some recommendations are done for the future design of the Afsluitdijk taking the new safety standard into account. After this paragraph the results obtained in this MSc research are discussed to point out uncertainties and the reliability of the results.

6.1. Conclusions

Before the final conclusion is made which answers the main research questions the sub questions are answered first. These sub questions divide the problem in smaller parts which are easier to answer and this forms a complete background for the final conclusion of this MSc research.

6.1.1. Research sub questions

1. Which failure mechanism are of importance for the Afsluitdijk?
In this Msc research the focus is on the dike body of the Afsluitdijk. The sluice complexes are not taken into account and therefore failure mechanisms for these hydraulic structures are not investigated. For the dike body itself the most likely failure mechanisms that have an influence on the probability of failure are investigated. These failure mechanisms are the following:

- wave run-up and overtopping
- instability of the revetment on the outer slope
- macro instability of the inner and outer slope
- piping and heave

Results from calculations with the Matlab model show that the failure mechanism with the largest influence on the probability of failure is wave run-up and overtopping. The second most important failure mechanism is the instability of the revetment on the outer slope. The probabilities of failure of the failure mechanisms of macro instability and piping and heave are very small compared to the other two failure mechanisms and they have an influence on the total probability of failure that can be neglected.

In this thesis only the failure mechanisms of wave run-up and overtopping and instability of the revetment on the outer slope are considered. In the assessment of the probability of failure of the Afsluitdijk, taking the correlation and length effect into account in PC-Ring, both these failure mechanisms are calculated. From the results in the Matlab model and from the results in PC-Ring it is concluded that the failure mechanisms of wave run-up and overtopping has a significant larger influence on the total probability of failure. Therefore only this failure mechanism is taken into account in the optimization of the alternative for the restrengthening of the Afsluitdijk.

2. What is the probability of failure of the existing Afsluitdijk?

In earlier assessments on the safety the Afsluitdijk is disapproved. Therefore a restrengthening design is developed to guarantee the safety against failure again. These assessments and the new design are based on the current standard and the alternatives that are developed at this moment do not take the new safety standard into account. It is possible that the existing Afsluitdijk is safe enough for the new safety standard, so the current probability of failure is assessed.

To assess the probability of failure of the existing design of the Afsluitdijk only the dike body is investigated. The sluice complexes are outside the scope of this MSc research. Because the dike sections around the sluice complexes are protected by breakwaters the hydraulic loads are less severe. The hydraulic loads are not known for these locations so these dike sections are not taken into account.

The rest of the dike sections are all assessed on the probability of failure due to overtopping. As expected before the dike section with the most severe hydraulic loads (dike section 8B) has the largest probability of failure, which is 1/275 per year. For all the dike sections combined the total probability of failure due to overtopping is 1/225 per year.

For the failure mechanism of instability of the stone revetment around 30% of the dike sections is calculated. This selection is based on a screening method proposed by the VNK project. The selection of the dike sections is done by selection the dike sections with the most severe hydraulic boundary conditions and/or the weakest revetment sections. For the Afsluitdijk the dike sections with the most severe hydraulic boundary conditions are selected (dike section 8A up to 10B). The weakest revetment section is in dike section 10A, where basalt blocks are placed directly on clay. The probability of failure at this location due to instability of the basalt blocks is 1/625 per year. For the dike sections combined the total probability of failure for this failure mechanism is 1/600 per year.

The total probability of failure of the existing Afsluitdijk is found after combining the results for overtopping and instability of the revetment. This results in a total probability of failure of around 1/200 per year. The current safety standard for the Afsluitdijk is safety against hydraulic loads with an exceedance frequency of 1/10 000 per year. In the current safety standard this exceedance frequency is chosen to comply with the adjacent flood defences, regardless of the possible consequences of failure of the Afsluitdijk.

In the new safety standards these consequences will be taken into account which could result in lower probabilities of failure.The consequences of a breach somewhere in the Afsluitdijk are not as severe as failure of a regular dike as only the water level of Lake IJssel will rise (up to 25 cm for a single storm event (Wijbenga and Meijer, 2006)). An economic optimization for the standard of the Afsluitdijk is done (Zwaneveld and Verweij, 2014) which results in an optimal design of the Afsluitdijk with a probability of failure of 1/9 400. The results of this investigation are the best estimation at this moment and it can be concluded that the current configuration of the Afsluitdijk with a probability of failure of around 1/200 per year is insufficient. Therefore improvement of the Afsluitdijk is needed.

3. What flood safety standard (probability of failure) for the Afsluitdijk is most likely to be introduced with the new standard?

As concluded above the Afsluitdijk has to be improved because the probability of flooding is too high at this moment. To develop a design it is useful to know what the new standard is going to be. In the answer on the previous question an economic optimization for the flood safety standard of the Afsluitdijk is mentioned (Zwaneveld and Verweij, 2014). In this investigation an economic optimal design for the Afsluitdijk is found with a probability of failure of 1/9 400 per year. This investigation is based on uncertainties and there still remains discussion about this proposed standard for the Afsluitdijk. At the moment no other studies have investigated the safety standard for the Afsluitdijk so in this MSc research the proposed probability of failure of 1/9 400 per year is used.

4. What is the probability of failure of the reference alternative that is developed at this moment?

In the assessment and the optimization of the alternative only the failure mechanism of wave run-up and overtopping is considered. With the design instrument of OI2014 and earlier results the standard for a single dike section for a single failure mechanism is determined. The safety standard (P_{norm}) for the total failure of the Afsluitdijk is multiplied by a failure mechanism budget (ω) and divided by a factor (N) to take the correlation and length effect into account:

$$P_{eis,dsn} = \frac{P_{norm} \cdot \omega}{N}$$

From the results a value of 0.5 is assumed for ω, 1.2 for N and with a standard of 1/9 400 per year the standard for a single dike section for the failure mechanism of wave overtopping is a probability of failure of around 1/22 500 per year.

The reference alternative B3+ that is developed by W+B is both calculated in the Matlab model and PC-Ring. The results are given in Table 6.1.

Model	**P_f [1/year]**	**R [years]**
Matlab	5.06E-05	19 750
PC-Ring	6.15E-05	16 250

Table 6.1: Probability of failure of the reference alternative for wave run-up and overtopping.

It is concluded that the probability of failure of this design is lower than the standard. With the assumed safety standard this design is not sufficient enough and the design must be improved. In the optimization of the reference alternative the probability of failure of 1/22 500 is set as a lower limit.

5. How can the design of the reference alternative be optimized?

The aim of the optimization of the reference alternative is to minimize the investment costs for a design that meets the safety standard of 1/22 500 per year for the failure mechanism of overtopping. In the optimization five variables that have the most influence on the probability of failure are varied resulting in possible alternatives. The variables with the largest influence coefficients are:

- dike height
- berm height
- berm width
- upper slope
- lower slope

With varying values for these variables a total of 5 250 alternatives are assessed on the probability of failure. From all these possible alternatives the ones with a probability of failure higher than the required standard are deleted. For all the possible alternatives left the investment costs are estimated, in which the rubble berm and the basalton blocks have the largest influence on the total costs. Looking at the designs with the lowest costs some alternatives are not realistic because of very steep slopes or a rubble berm that is not wide enough to provide safety against the instability of the revetment. These not realistic alternatives are also deleted from the selection.

The possible alternatives with the lowest costs are found to be cheaper than the reference alternative, while having a larger safety against flooding. The result of the cheapest possible alternative and the reference alternative is shown in Table 6.2. It is concluded that optimization of the design results in lower investment costs while improving the safety against failure due to overtopping. However a check on the stability of the revetment and the slope of the optimized alternative is needed to guarantee that the standard for the total probability of failure is met.

alternative	h_{crown} [m]	h_B [m]	B [m]	$tan(\alpha_{up})^{-1}$ [-]	$tan(\alpha_{low})^{-1}$ [-]	R [years]	investment costs [€/m]
Optimized	8	5	7.5	4.5	3.5	31 250	15 000
B3+	7.73	5.5	16	5	2.5	19 750	19 000

Table 6.2: Cost estimation for the alternative with minimal costs and the reference alternative B3+.

6. How can the uncertainty of the new risk approach be taken into account in the design of the Afsluitdijk?

To cope with the uncertainty of the new standard for the Afsluitdijk a design is needed that is easily adaptable for changes in the future. By not investing too much money at this point, keeping possible adaptions in the future in mind, possible unnecessary expenditures can be saved. It is possible that the new safety standard will be lower (or higher) than the recommended probability of failure of 1/9 400 per year. Therefore it is recommended to implement a design that is easily adaptable and improvements with low investment costs but a high influence on the lowering of the probability of failure are recommended to be ready for future changes.

The design of the reference alternative is a good example of an adaptable alternative, in which the height and the width of the berm are easily adaptable by adding more rubble. Also the lower slope is easily adaptable. Heightening of the dike crown or adjustments to the upper slope are a lot harder with this design. Changing the upper slope or the height of the dike crown results into changes to the lower part as well. If this is done the rubble has to be removed before changes can be made which will cost extra money.

6.1.2. Final conclusion

The final conclusion that has to be made is the answer to the main research question. By taking the preliminary conclusion from the sub questions into account a final conclusion is drawn.

What is a probabilistic, cost-effective and adaptable design of the Afsluitdijk?

The new safety standard that is planned to be implemented in 2017 is being developed at this moment. For all the dike rings in the Netherlands the project VNK gained insights in the probabilities of flooding and the possible consequences of a flood. This is done with probabilistic methods, which is in line with the new flood safety standard. The results from this VNK project are very useful in the development of the new safety standard for flood defences. All the dike rings along Lake IJssel are assessed with the assumption that the Afsluitdijk will not fail. Therefore the Afsluitdijk itself is not investigated in the VNK project. Therefore there are no results available that are useful to come to a conclusion about the safety of the Afsluitdijk at this moment, or the possible future safety standard.

As the Afsluitdijk is a very unique dike it demands a unique approach to determine the new design. Because the Afsluitdijk does not protect direct hinterland but Lake IJssel, a choice between investing in the Afsluitdijk itself and investing in dike rings along Lake IJssel has to be made. In a study of the Lake IJssel region economic optimal flooding probabilities for all the dike rings around Lake IJssel are proposed, including the Afsluitdijk. This study is based on uncertainties and there is discussion about the reliability of the results. The results of this study are useful in the development of the new safety standard for the Afsluitdijk but future research may result in different proposals for the standard.

To cope with these uncertainties it is recommended to make an easily adaptable design for the renovation of the Afsluitdijk. A conclusion from the results is that the reference alternative that is developed by Witteveen+Bos is an easily adaptable design because in the design a berm is constructed off rubble, which can easily be added to widen the berm and/or decrease the lower slope. Therefore the main characteristics of the design of the reference alternative are good taking the new safety standard into account. Optimization of the dimensions of the reference alternative could reduce construction costs of the renovation of the Afsluitdijk.

To answer the main research question the optimized alternative shown in Figure 6.1 is a cost-effective and adaptable design for the renovation of the Afsluitdijk. In the next years the flood safety standard for the Afsluitdijk is chosen and this design can easily be adapted to meet the future uncertain standard. The probabilistic design method used in this MSc thesis is partly derived from the temporary flood defense design instrument OI2014 and it is shown that further investigation is needed. In this investigation the failure mechanism budgets ω and the correlation and length-effect factor N need to be derived to improve the accuracy of the results. Besides the uncertainty in these factors only the failure mechanism of wave overtopping is taken into account and the increasing increasing hydraulic loads (due to climate change) are not taken into account.

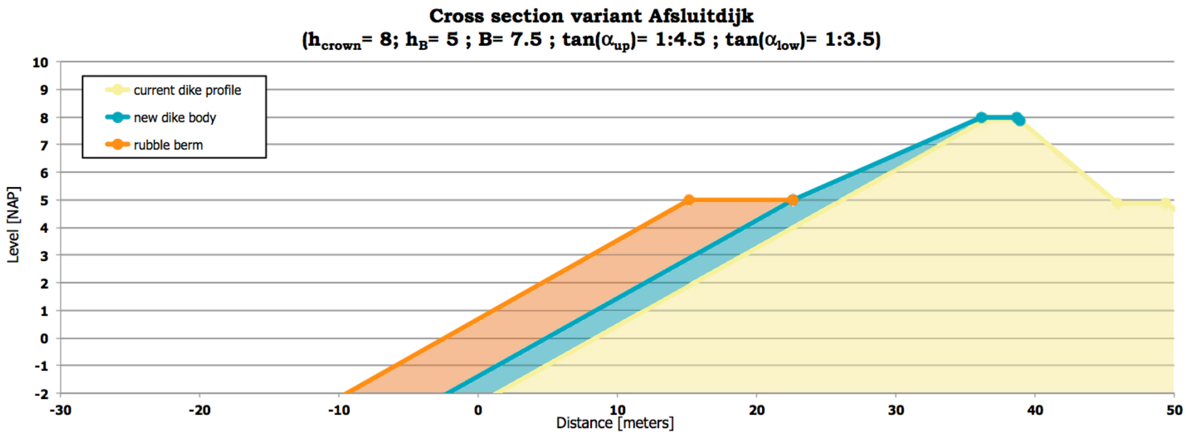

Figure 6.1: Schematization of the cross section of the recommended adaptable alternative.

The final conclusion is that the reference alternative B3+ developed at this moment is a good concept for the renovation design of the Afsluitdijk. The design is easily adaptable but improvement can result in a more cost-effective solution. To find a final solution further investigation is needed which is recommended in the next paragraph.

6.2. Recommendations

In this paragraph recommendations are given for further research and investigation. The results obtained in this MSc research are obtained under assumptions and in most of the cases further research is recommended that can increase the results. Besides this some results in this MSc research bring up questions that are outside the scope of this MSc research. Therefore further investigation on those topics is something that is recommended as well in this paragraph. Furthermore a part of this MSc research focusses on the reference alternative that is under development at the moment and some recommendations for this reference alternative are given as well.

6.2.1. Recommendations for further research

- Investigate the economic optimal design of the Afsluitdijk by analyzing the complete Lake IJssel region. The study by the CPB (Zwaneveld and Verweij, 2014) is based on numerous assumptions and discussion remains about the reliability of the results.
- Assess the probability of failure for the sluice complexes. The hydraulic structures are not considered in this MSc research but they have an influence on the total probably of failure.
- Investigate the residual strength of the inner slope and the dike crown for the failure mechanism

of overtopping. In the VNK method the CIRIA model is used, which is a model that is not up-to-date and more detailed models are available nowadays.

- Investigate the failure mechanism of macro stability for more cross sections and calculate the total probability of failure in PC-Ring. In PC-Ring the probabilities of failure at different water levels are taken into account in a more accurate way resulting in more reliable results.
- Investigate the correlation and length effect coefficients for all the stochastics used in the VNK method. As shown in this MSc research the correlation and length effect has a large influence on the results. Research may result in decrease of the variation and correlation and length effect coefficients, resulting in lower probabilities of failure.
- Investigate the factor N that takes the length effect into account further. With a new design the probabilities of failure for the different dike sections are closer to each other, resulting in a factor of N which is probably higher than the estimated value of 1.2 for the weakest dike section 8B.
- Investigate the failure budget mechanisms ω proposed in the OI2014 for the Afsluitdijk. In this MSc research it is assumed that because piping does not play a role and overtopping has the largest influence on the total probability of failure the budget for piping can be added to the budget for overtopping.
- Investigate the influence of sea level rise and increasing wave loads. For the future design a rising sea level results in more severe hydraulic boundary conditions which could result in higher probabilities of failure.
- Investigate the hydraulic loads and the strengths of the dike sections that are protected by breakwaters in front of the Afsluitdijk. These sections are dike sections 1 to 4 and and 12 to 16.
- Investigate the critical overtopping discharge that is chosen in the design and assessment standards for the failure mechanism of wave run-up and overtopping.
- Investigate the influence of the cumulation effect of multiple failure mechanisms. A starting failure mechanism may have an influence on other failure mechanisms which possibly increases the total probability of failure.
- Investigate the possibilities to reduce the uncertainty of the stochastic variables that are used as an input. With increased measurement the uncertainty of some stochastics can be reduced, resulting in a lower variation which results in a lower probability of failure.
- Assess the current strength of all the dike sections for the failure mechanism of instability of the revetment on the outer slope. In this MSc research a selection is made in which the dike sections with the most severe boundary conditions are selected to calculated the probability of failure for this mechanism. However weaker revetment sections can be present on other dike sections although the hydraulic boundary conditions are less severe.
- Investigate the development of a breach in the Afsluitdijk. Some research is done ((Visser, 2002) and (Wijbenga and Meijer, 2006)) but more detailed studies can result in better knowledge on the growth of a breach in the Afsluitdijk.
- Investigate the effects of the entering of salt water in Lake IJssel on the flora and fauna and fresh water supply. Although a breach in the Afsluitdijk may not have a large effect on the safety of the flood defences along Lake IJssel the entering of salt water may have severe negative effects on the environment.
- Investigate the correlation between the different variables of the hydraulic boundary conditions. In this MSc research the wave height depends on the water levels for the failure mechanism of overtopping. For the failure mechanisms of instability of the revetment these two variables are not linked to each other but in both cases the wave period depends on the wave height. Further research must show if these dependencies between the variables are correct.

6.2.2. Recommendations for the design of the Afsluitdijk

- Investigate the probability of failure of the optimized reference alternative for other failure mechanisms as well. In this MSc research the alternative is optimized for the failure mechanism of wave run-up and overtopping to get an idea of the possible optimization of the alternative, however further research is recommended to guarantee a safe design.
- Increase the height of the dike crown in the restrengthening design of the Afsluitdijk. The dike

crown height has a large influence on the safety against overtopping and taking into account that heightening of a dike is harder after the installation of rubble it is recommended to invest in higher dike crown right now.

- Make use of an easily adaptable design like the reference alternative. By adding more rubble to this design the berm can be widened or heightened resulting in lower probabilities of failure without radical measures. It is recommended to make a design that is not on the safe side of the current standard. If the future standard turns out to be stricter than the current standard extra rubble can easily be added. With a less conservative design money is saved. A recommended design following from the optimization in this MSc research has the following variables:
  - dike crown height = + 8 m NAP
  - berm height = + 5 m NAP
  - berm width = 7.5 m
  - upper slope = 1:4.5
  - lower slope = 1:3.5

Bibliography

Bakker, H.L. Omrekenen schuifsterkteparameters voor M-Prostab en Uplift Van Probabilistic. Technical report, VNK, 2004.

Commissie Lorentz, . Verslag van de Staatscommissie - Gevolg van de afsluiting van de Zuiderzee. Technical Report September, 1926.

de Valk, C.F. Load models for reliability analysis of six dike sections along the Westerschelde. Technical report, ARGOSS, 1998.

den Heijer, C. Reliability methods in OpenEarthTools October 2012. Technical Report October, Delft University of Technology, 2012.

Forster, U.; van den Ham, G.; Calle, E., and Kruse, G. Onderzoeksrapport Zandmeevoerende Wellen. Technical report, Rijkswaterstaat, 2013.

Hai Trung, L. *Overtopping on grass covered dikes*. PhD thesis, Delft University of Technology, 2014.

Kind, J. Maatschappelijke kosten-batenanalyse Waterveiligheid 21e eeuw. Technical report, Deltares, 2011.

Klein Breteler, M. Handleiding Steentoets2008. Technical report, Deltares, 2012.

Landa, P.M. Wave Overtopping Resilient Afsluitdijk, 2014.

Ministerie van IM, . Structuurvisie Toekomst Afsluitdijk - Ontwerp. Technical report, 2011.

Ministerie van IM, and Ministerie van EZ, . Deltaprogramma 2014 - Werk aan de delta. 2013.

Ministerie van VW, . Legger afsluitdijk. Technical Report november, Ministerie van VW, 2009.

Projectbureau VNK2, . Van ruwe data tot overstromingsrisico. Technical Report September, Rijkswaterstaat, 2012.

Steenbergen, H.M.G.M. and Vrouwenvelder, A.C.W.M. Theoriehandleiding PC-RING - Deel C: Rekentechnieken. Technical report, TNO, 2003a.

Steenbergen, H.M.G.M. and Vrouwenvelder, A.C.W.M. Theoriehandleiding PC-RING - Deel B: Statistische modellen. Technical report, TNO, 2003b.

Steenbergen, H.M.G.M.; Vrouwenvelder, A.C.W.M., and Koster, T. Theoriehandleiding PC-Ring - Deel A: Mechanismenbeschrijvingen. Technical report, TNO, 2007.

Stichting CUR, . *Probabilities in civil engineering, Part 1: Probabilistic design in theory*. 1997.

Stijnen, J.W.; Thonus, B.I.; Diermanse, F.L.M.; Geerse, C.P.M.; van Haaren, D., and Nicolai, R.P. Functionele documentatie Hydra-K, 2012.

Trompille, V.; Eerninck, N.L.M.; Morsch, F.M.; Best, H., and van der Meij, R. D-Geo Stability manual. Technical report, Deltares, 2011.

van der Meer, J.W. Technisch Rapport Golfoploop en Golfoverslag bij Dijken. Technical report, Technische Adviescommissie voor de Waterkeringen, 2002.

van der Wouden, F. and Grashoff, P.S. Gebruikershandleiding PC-Ring. Technical Report Sp 3, Demis, 2009.

van Ierland, E.; Koopmans, C.; Rietveld, P., and van der Veen, A. Advies van de commissie van economische deskundigen over de CPB studie "Economisch optimale waterveiligheid in het IJsselmeergebied". Technical report, 2014.

Visser, P.J. Groei bres in Afsluitdijk na dijkdoorbraak. Technical Report September, TU Delft, 2002.

Weijers, J. and Tonneijck, M. *Lecture notes CT5314 - Flood Defences*. Number February. 2009.

Wijbenga, J H A and Meijer, K L. Hydra-M Achterland - Consequenties falen Afsluitdijk en Houtribdijk. Technical report, HKV, 2006.

Witteveen + Bos, . Nota van uitgangspunten en randvoorwaarden dijklichaam en kunstwerken, Afsluitdijk. Technical report, Rijkswaterstaat, 2013.

Witteveen + Bos, . Systeemontwerp Dijk (concept 1.0). Technical report, Rijkswaterstaat, 2014.

Zwaneveld, P. and Verweij, G. Economisch optimale waterveiligheid in het IJsselmeergebied, 2014.

A

Hydraulic boundary conditions

In this chapter the method of deriving the hydraulic boundary conditions is described. For the previous assessment of the Afsluitdijk in 2011 the hydraulic boundary conditions were derived with Hydra-K. Hydra-K is described in the first paragraph. In the second paragraph the method to derive the hydraulic boundary conditions for the probabilistic calculation of failure of the Afsluitdijk is described. In the third paragraph the hydraulic boundary conditions for the different dike sections of the Afsluitdijk are given.

Description of Hydra-K

Hydra-K is a software module which can be used to assess the primary flood defences along the Dutch coast. For the failure mechanisms wave run-up, wave overtopping and instability of the revetments the current flood defences can be assessed. Hydra-K can also be used to generate water levels and wave conditions for the hydraulic boundary conditions. Hydra-K is able to make four types of calculations:

- Probability of failure for a specific failure mechanism.
- Determination of design parameters for a specific failure mechanism for a given failure frequency.
- Calculation of an illustration point (for the hydraulic boundary conditions).
- Deterministic validation calculation.

Hydra-K is developed for the WTI2011. Also Hydra-K is used to derive the hydraulic boundary conditions (HR2011) for the legal assessment of the flood defences in the Western Scheldt and the Wadden Sea. Hydra-K is used by Rijkswaterstaat to assess the flood defences along the Dutch coast. The software is developed by HKV commissioned by Rijkswaterstaat. For the complete background of Hydra-K see (Stijnen et al., 2012).

The used method for the determination of the hydraulic boundary conditions on the sea defences can be divided in four steps:

- Determination of the statistics of each of the four variables on deep water. These variables are the wind speed, water level, significant wave height and the wave peak period. All these variables are determined for different wind directions. To these statistics a series of simultaneous observations of storm surge conditions are added. With these extreme conditions the correlation of the variables is determined.
- Translation of the physical parameters on deep water (offshore) to values that occur 50 meters from the toe of the dikes (near shore).
- Probabilistic calculation of the exceedance probability of the failure criterium. In this calculation the complete statistics of the parameters are used. With a deterministic strength model it is determined if failure occurs.
- Calculation of the illustration points for a given dike profile.

Hydra-K uses a parameter free method to estimate the failure frequencies of flood defences, caused by multiple loads. Hydra-K is based on multi dimensional extreme value statistics. To calculate the failure frequencies method De Haan is used. Method De Haan shifts the observations diagonally until a specified amount of observations is in the failure plane. The failure frequency can be determined by the number of observations in the failure plane, the total time of the observations and the size of the shift of observations.

Method De Haan

The determination of the failure frequency with the method De Haan is done with a special type of the Monte Carlo simulation; importance sampling. In importance sampling draws for the Monte Carlo simulation are done in the area where the limit state function is around zero. This is done with the use of a transformation (like the shift of the observations described in the previous paragraph). An advantage of the method De Haan is the relatively small number of realizations and an efficient calculation. The method can only be used if the different variables are asymptotically dependent.

Asymptotically dependency is a special type of dependency in which the variables are (almost) completely dependent in the extreme value area. This total dependency means that with one observation of an extreme value of a stochastic the other values of the variables can be determined within a relatively small uncertainty boundary. The assumption of asymptotically dependency must always be validated. Previous research (de Valk, 1998) has shown that for the wind directions between 225° and 345° the water level and wind speed are indeed asymptotically dependent. As in extreme value events the direction of the wind is in between those boundaries the method De Haan can be used.

Method De Haan consists of three steps:

- (Omni)directional extreme value statistics for the individual stochastic variables.
- Extrapolation of the correlation structure of the simultaneous observations of the stochastic variables.
- Validation on asymptotical independency.

In Hydra-K four stochastic variables are taken into account; wind speed, still water level, significant wave height and wave period. These variables are conditionally dependent on the wind direction. In Hydra-K twelve different wind directions are separated. For a description of how the wind direction is taken into account see (Stijnen et al., 2012).

The first step in method De Haan is to determine extreme value statistics. Because there are no observations of extreme storm conditions the observations of normal storm conditions are extrapolated. After transformation of the observations the values are shifted in the exponential space. This results in good extreme value statistics with asymptotically dependent variables.

The exceedance frequency is given by the total independent storms per year. The water levels, wave heights and wave periods are selected in such a way that four tidal periods before and after the observation no higher values are observed. For the wind speed hourly observations are used instead of independent storms. For every variable the marginal distribution functions are determined before they are combined into the multi variable extreme value statistics.

If these stochastic variables are described by the same distribution function the second step is done in the most efficient way. In Hydra-K the conditional Weibull distribution is used. This distribution is defined as:

$$P[U > a|U > \omega] = exp\left\{-\left(\frac{a}{\sigma}\right)^{\alpha} + \left(\frac{\omega}{\sigma}\right)^{\alpha}\right\} \tag{A.1}$$

$$F[U > a] = \rho P[U > a|U > \omega] \tag{A.2}$$

in which:

$P[U > a \mid U > \omega]$	probability that stochastic variable U is larger than a, given that U is larger than ω
$F[U > a]$	$(= \mu(a))$ frequency of stochastic variable U exceeding value a
$F[U > \omega]$	$(=\rho)$ storm frequency (of threshold value ω)
U	stochastic variable
a	realization of stochastic variable U
α	shape parameter
σ	scale parameter
ω	threshold value

The parameter values for the conditional Weibull distributions are used as input for Hydra-K. The inverse of the conditional Weibull distribution is used for the transformation from the exponential space to the Weibull space and is given by:

$$a = \sigma\left\{\left(\frac{\omega}{\sigma}\right)^{\alpha} + log(\rho) - log(\mu(a))\right\}^{\frac{1}{\alpha}} \qquad \text{(A.3)}$$

In the second step (extrapolation of the correlation structure) the most unfavorable load combination of variables is used. This combination depends on the failure mechanism. In case of wave run-up and overtopping the angle of wave impact also plays a role. The extrapolation is done by shifting of the simultaneous observations to extreme values around the failure area of the flood defense. The amount of shifting is denoted by a shift vector λ^* and is the same for all the simultaneous observations. By iteration $\lambda*$ is changed until a pre specified amount of points are in the failure area. In the method De Haan it is assumed that the correlation structure remains the same for extrapolation in the exponential linear space of the exceedance frequencies.

The failure frequency can be obtained by the simultaneous observations, without the determination of the multi dimensional statistics by:

$$\nu = \frac{\kappa \cdot exp(-\lambda^*)}{\Delta} \qquad \text{(A.4)}$$

in which:

ν	failure frequency per year
κ	number of points in failure area
$\lambda*$	shift vector in exponential linear domain
Δ	length of period in which observations are gathered

The failure frequency can be transformed into a probability of failure:

$$P_f = 1 - exp(-\nu) \qquad \text{(A.5)}$$

The final step is the check on asymptotical independency. This is the case when the extreme values of stochastic variables remain between a uncertainty boundary.The dependency can be assessed in the linear exponential space by the conditional probability:

$$P[U_2 > a \mid U_1 > a] = \frac{P[U_2 > a \cap U_1 > a]}{P[U_1 > a]} \text{ with: } P[U_1 > a] = P[U_2 > a] \qquad \text{(A.6)}$$

in which U_1 and U_2 are exponentially distributed stochastic variables. Asymptotical dependency is the case if the limit of the conditional probability is a positive value when a approaches infinity. If this limit is equal to zero the variables are asymptotically independent.

For total indecency the following for the stochastic variables applies:

$$P[U_2 > a \mid U_1 > a] = \frac{P[U_2 > a \cap U_1 > a]}{P[U_1 > a]} = \frac{P[U_2 > a] \cdot P[U_1 > a]}{P[U_1 > a]} = P[U_2 > a] \qquad \text{(A.7)}$$

When a approaches infinity this probability is equal to zero.

For total dependency the following for the stochastic variables applies:

$$P[U_2 > a \mid U_1 > a] = 1 \qquad \text{(A.8)}$$

For partial dependency of the stochastic variables a method is used to determent the asymptotical dependency. A 'slope' ξ and a dependency function $\phi_{0.5}$ are defined for a approaching infinity:

$$\xi = \frac{log(P[U_2 > a|U_1 > a])}{log(P[U_1 > a])} \tag{A.9}$$

$$\phi_{0.5} \in [0.5, 1] \tag{A.10}$$

It holds that:

$$\xi = 2\phi_{0.5} - 1 \tag{A.11}$$

Only if $\phi_{0.5} = 0.5$ and according to that $\xi = 0$ asymptotical dependency is the case. In all the other cases the variables are asymptotically independent. In Hydra-K is is assumed the variables are asymptotically dependent so this validation is not part of the Hydra-K module.

Upscaling of the storm build up

For the failure mechanisms wave run-up and wave overtopping the maximum load is normally more or less when the water level reaches a maximum (in a single storm event). For the failure mechanisms instability of the revetment this is not the case, especially not for revetments on the lower slopes of flood defences. These parts may be under the SWL during the maximum overtopping load of a storm. The maximum load for those revetments may occur during the build up of a storm, so the complete storm event must be taken into account. This means that the upscaling must be done for a complete storm event.

The upscaling is done with a upscaling triangle. The upscaling value for the maximum water level is defined with method De Haan. This means that the maximum water level is determined according to the exceedance frequency. The upscaling value is the difference between the maximum water level before and after extrapolation with method De Haan. The upscaling values at the begin and at the end of a storm event are equal to zero. This upscaling triangle is added to the water levels to determine the extreme values for a complete storm event. An example of an upscaling triangle for a storm at Harlingen in February 1990 is shown in Figure A.1. The same upscaling triangle is used to determine the wind speed, in spite of the fact that the maximum wind speed may be at a different time than the maximum water level.

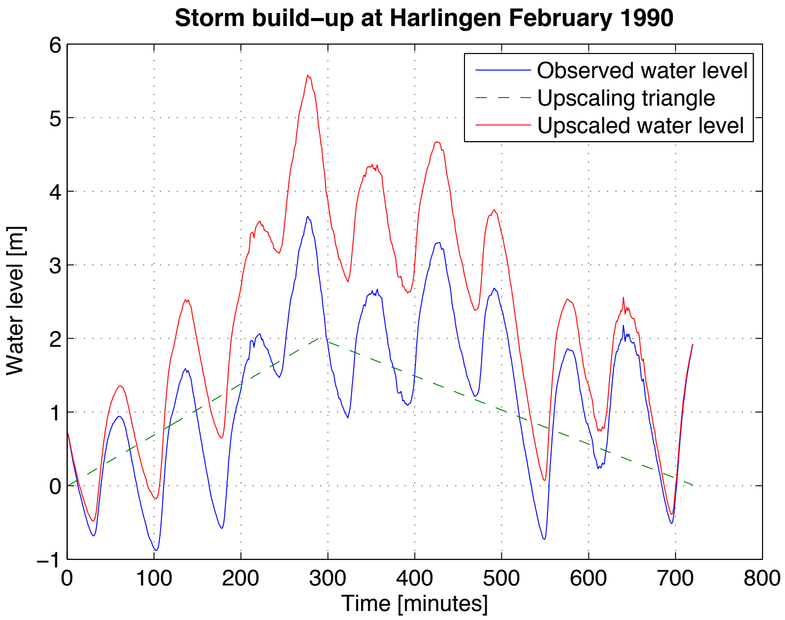

Figure A.1: Storm build up with an upscaling triangle for a storm at Harlingen, February 1990.

Translation offshore-nearshore

To determine the hydraulic boundary conditions for a flood defense the calculated extreme values for the different variables offshore (with method De Haan) are translated into nearshore values. To do this the shifted points are translated with a numerical wave model (e.g. SWAN) to the locations of the flood defences. To prevent unnecessary calculation work a matrix of load combinations is calculated with a translation from offshore to nearshore water in preparation of a Hydra-K calculation. For every location a separate matrix is determined and with these matrices the dependencies between the variables for offshore and nearshore water are derived with multilinear interpolation. De transformation of the load variables from offshore to nearshore water consists of the following steps:

1. Relate the offshore wave height and period with the offshore wind speed. Set the direction of the waves to the same direction of the wind. Assume that the offshore and nearshore wind direction is equal.
2. Interpolate and extrapolate the nearshore high water level for each wind direction linear based on the available nearshore and offshore high water levels.
3. Transform the nearshore wave direction by taking the sine and the cosine. To prevent dividing by zero interpolation is done with regard to the arctangent.
4. A relative significant wave height is used which is the quotient of the significant wave height and the local water depth. This increases the accuracy during inter- and extrapolation.
5. Extrapolate the nearshore relative significant wave heights and nearshore wave periods boor low nearshore high water levels and wind speeds, by assuming that the nearshore wave parameters are linear dependent to the nearshore high water level en to the logarithm of the nearshore wind speed.
6. For high nearshore high water levels and wind speeds the extrapolation is done with a linear dependency for both the significant wave height and the wave period.
7. Apply multi linear interpolation for the intermediate nearshore relative significant wave heights, wave periods an the transformed wave directions.

Method for determination of the hydraulic boundary conditions for the Afsluitdijk

To determine the hydraulic boundary conditions for the different dike sections of the Afsluitdijk Hydra-K is used. With Hydra-K for different design exceedance frequencies the corresponding variables are calculated. This is done at the different locations that represent the dike sections of the Afsluitdijk. The representative dike section (6a) for the Afsluitdijk is used as an example in this appendix. In Table A.1 the values of the variables are given for different return periods.

Return period [year]	SWL [m]	Hs [m]	Tm-1,0 [s]
1	2.51	0.76	2.27
3	2.92	1.16	3.11
10	3.29	1.50	3.91
30	3.62	1.79	4.24
100	3.94	2.06	4.53
300	4.21	2.27	4.70
1 000	4.52	2.31	4.87
3 000	4.77	2.47	5.02
10 000	5.02	2.64	5.17
30 000	5.24	2.77	5.27
100 000	5.48	2.92	5.39

Table A.1: Return periods for the high water level, significant wave height and mean energy wave period.

As described above the variables are dependent in the extreme value area. Therefore we estimate a distribution for the water level and find relations between the water level and the other variables. If for all the variables different distributions are derived this will result in a much lower probability of failure when the Monte Carlo method is used. Because in a Monte Carlo simulation values are randomly drawn from the stochastic variables the chance that for both the water level and the wave height extreme values are drawn is very small. However as described above the wave height and the water level are dependent and for extreme values of the water level also extreme values for the wave height occur. So therefore only a stochastic variable for the water level is estimated and the other variables are described with a relationship to the water level.

The distribution type of the still water level is a conditional Weibull which can be estimated from the exceedance frequency values with Equation A.3. This estimation is done in Matlab with the lsqcurvefit function which solves non-linear least square problems. For the values given in Table A.1 the following parameters for the conditional Weibull distribution are found:

Parameter	Value
ω	2.36
ρ	1.91
α	1.76
σ	1.14

In Figure A.2 the data points obtained from Hydra-K and the conditional Weibull fit are shown.

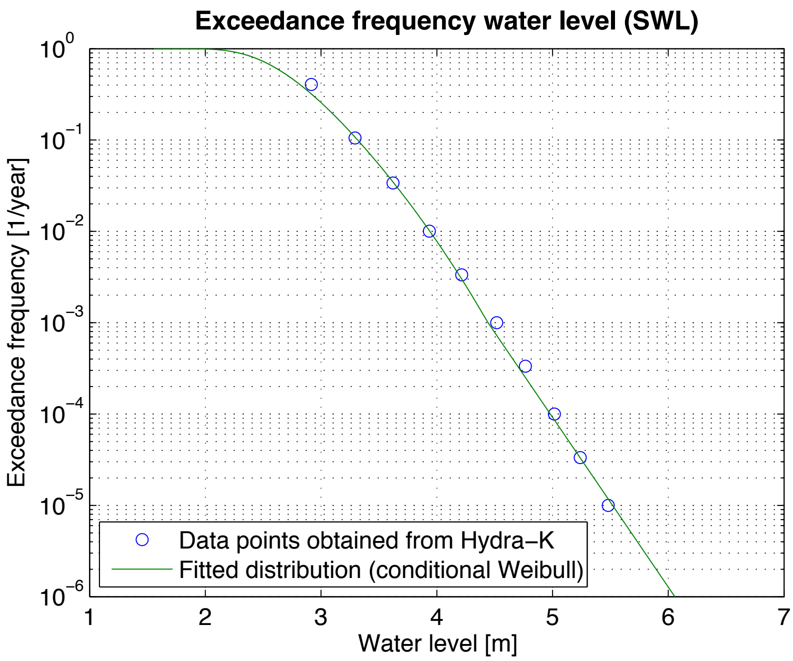

Figure A.2: Fitted conditional Weibull distribution according to the exceedance frequency values given by Hydra-K.

To obtain the relationship between the water level and the wave height the following formula is used:

$$H_s = C_1 log(SWL) + C_2 \tag{A.12}$$

With the data obtained from the Hydra-K the constants C_1 and C_2 are estimated in Matlab with the lsqcurvefit function. The estimates of the constants are:

Constant	Value
C_1	2.70
C_2	-1.70

The data points from Hydra-K for the relationship between the water level and the wave height and the estimated fit are shown in Figure A.3.

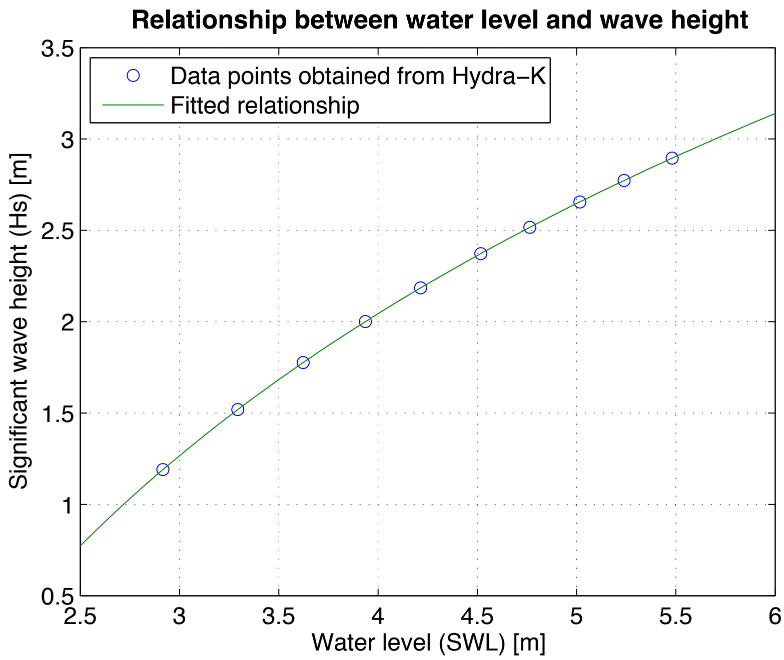

Figure A.3: Fitted conditional Weibull distribution according to the exceedance frequency values given by Hydra-K.

The final relationship that is estimated is the relationship between the significant wave height and the mean energy wave period. For this relationship the following formula is used:

$$T_{m-1,0} = \sqrt{\frac{H_s}{C_3}} \tag{A.13}$$

The estimation of C_3 is done with the lsqcurvefit function in Matlab and the estimate is:

Constant	Value
C_3	0.10

The data points from Hydra-K for the relationship between the water level and the wave period and the estimated fit are shown in Figure A.4. It is observed that the lowest data point is not close to the fitted line. This can be explained by the fact that the dependency is for extreme values due to asymptotically independence. This means that for the values with smaller return periods (so for this lowest data point) the dependency is not certain.

The distribution of the water level, wave height and wave period can be shown from the realizations by the Monte Carlo simulation. The generated values in this simulation are plotted as a histogram, see Figure A.5, A.6 and A.7. With these distributions values for the return periods are derived and compared with the values obtained with Hydra-K. The error between the fitted distributions and the obtained values with Hydra-K is expressed as the squared difference. The results are shown in Table A.2 and as we can see the errors are relatively small.

The Matlab function file that is used to determine the constants for the hydraulic boundary conditions is given in Appendix E.

Hydraulic boundary conditions for the Afsluitdijk

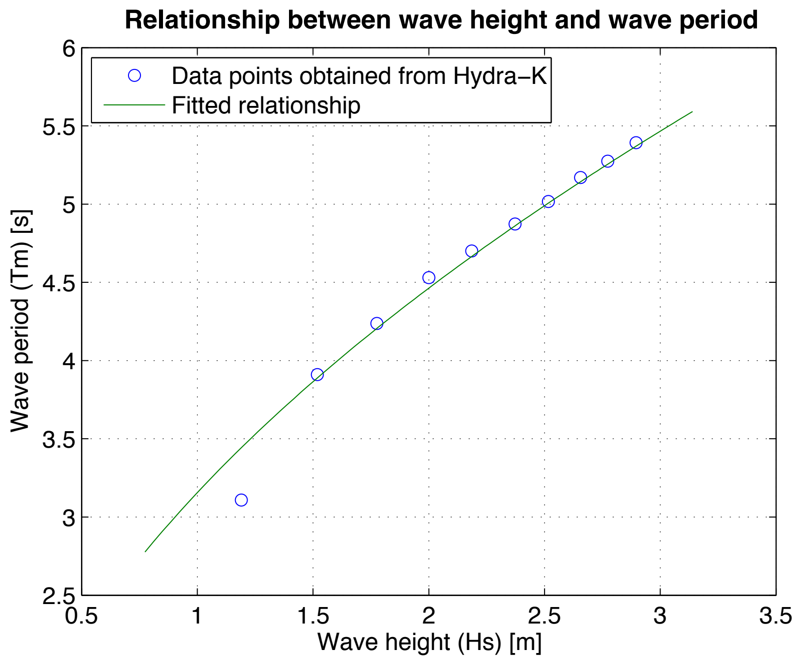

Figure A.4: Fitted conditional Weibull distribution according to the exceedance frequency values given by Hydra-K.

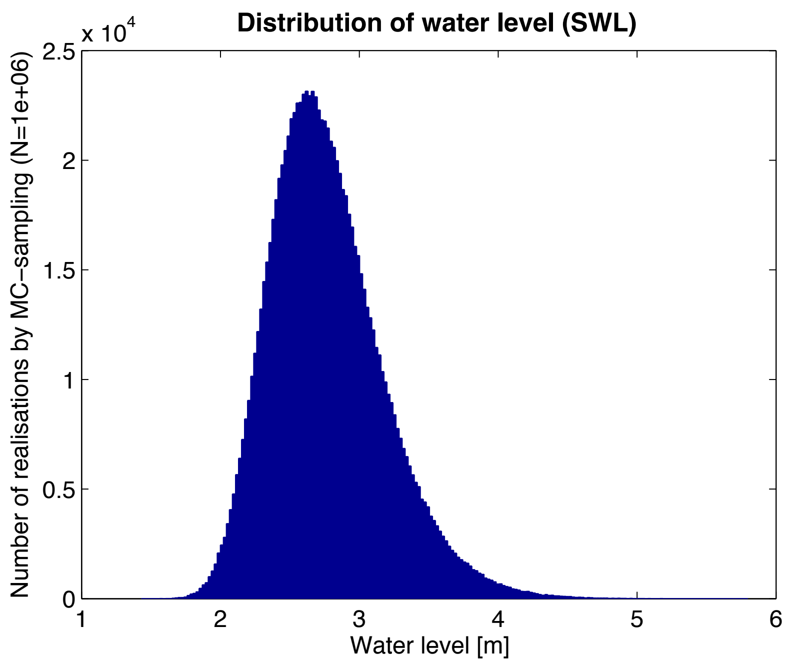

Figure A.5: Fitted conditional Weibull distribution according to the exceedance frequency values given by Hydra-K.

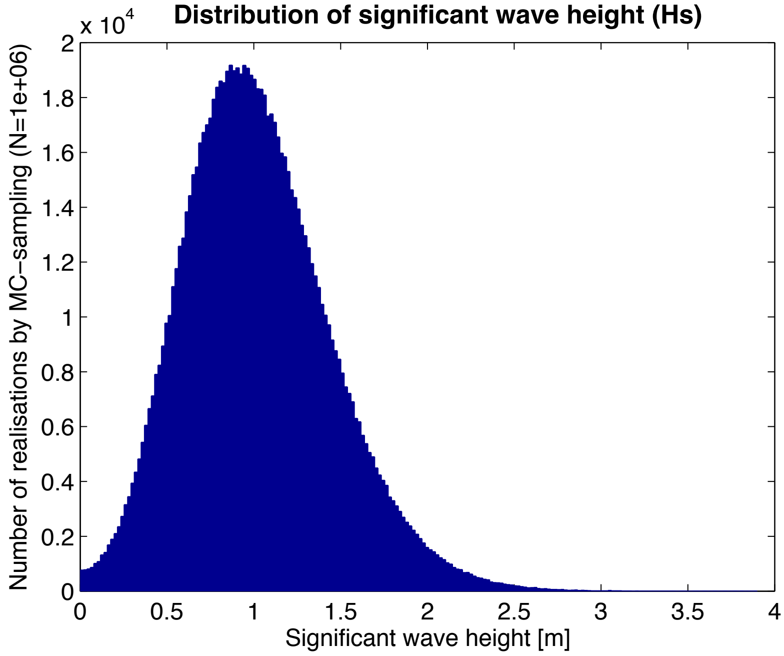

Figure A.6: Distribution of the significant wave height from the relationship with the water level.

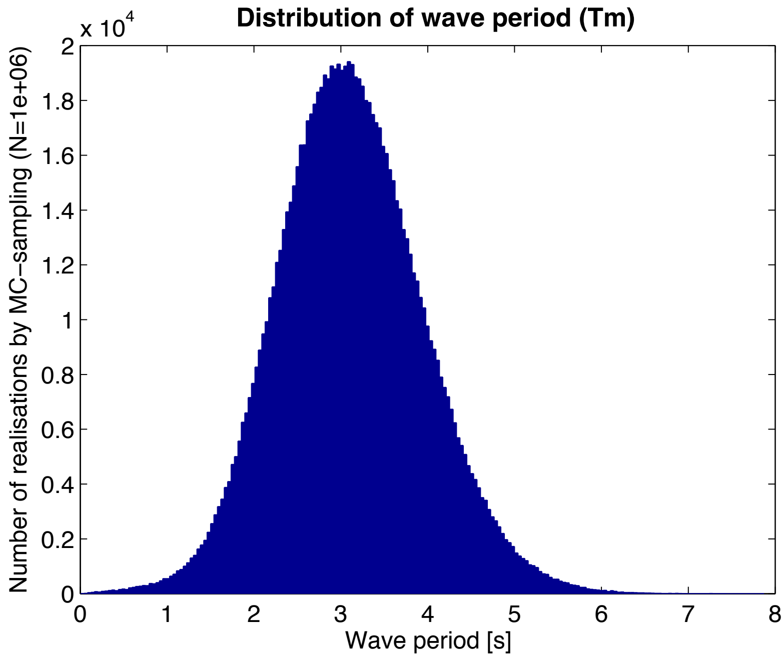

Figure A.7: Distribution of the wave period from the relationship with the significant wave height.

Return period [year]	**SWL [m]**	**fit**	**R2**	**Hs [m]**	**fit**	**R2**	**Tm-1,0 [s]**	**fit**	**R2**
1	2.51	2.59	7.07E-03	0.76	0.87	1.29E-02	2.72	2.95	5.27E-02
3	2.92	2.90	3.84E-04	1.19	1.17	3.32E-04	3.11	3.42	9.44E-02
10	3.29	3.31	2.22E-04	1.52	1.53	1.49E-04	3.91	3.90	3.01E-05
30	3.62	3.63	2.11E-05	1.78	1.78	1.17E-05	4.24	4.21	7.77E-04
100	3.94	3.95	1.01E-04	2.00	2.01	4.76E-05	4.53	4.47	3.41E-03
300	4.21	4.22	2.93E-05	2.19	2.19	1.20E-05	4.70	4.67	1.03E-03
1 000	4.52	4.50	1.71E-04	2.37	2.36	6.12E-05	4.87	4.85	3.84E-04
3 000	4.77	4.75	1.56E-04	2.52	2.51	5.03E-05	5.02	5.00	3.14E-04
10 000	5.02	5.01	8.36E-06	2.66	2.65	2.43E-06	5.17	5.14	8.35E-04
30 000	5.24	5.24	1.35E-05	2.77	2.77	3.59E-06	5.27	5.26	2.93E-04
100 000	5.48	5.49	1.76E-05	2.90	2.90	4.27E-06	5.39	5.37	4.22E-04

Table A.2: Return periods for the high water level, significant wave height and mean energy wave period with the corresponding fitted distributions and the squared error.

B

Probabilistic approach failure mechanisms

To determine the probability of flooding for the Afsluitdijk the possible failure mechanisms for every dike section are taken into account. In this appendix the method of assessment of the probability of failure for these failure mechanisms are described. These methods are used to calculate the total probability of failure of the Afsluitdijk. In the first subsection the general probabilistic model is described that calculates the probability of failure for each failure mechanism. In the successive sections the theory of the failure mechanisms is described and the stochastic variables and limit state functions are given. These variables and the limit state function are the input for the probabilistic calculation. For the failure mechanism macro stability this probabilistic model is not used but a probabilistic module of the software program Mstab[1] is used.

Probabilistic method for determination of probability of failure

The method used to determine the probability of failure is the crude Monte Carlo method. This method and the formulas used in this paragraph are described in more detail in (Stichting CUR, 1997). In this method the possibility of drawing random numbers form a uniform probably density function between zero and one is used. The non-exceedance probability of an arbitrary random variable is uniformly distributed between zero and one, regardless of the distribution of the variable:

$$F_X(X) = X_u \tag{B.1}$$

in which X_U is the uniformly distributed variable between zero and one and $F_X(X)$ is the non-exceedance probability $P(x < X)$. So for the variable X:

$$X = F_X^{-1}(X_u) \tag{B.2}$$

In which $F_X^{-1}(X_u)$ is the inverse of the probability distribution function of X. A random number X can be generated from an arbitrary distribution $F_X(X)$ by drawing a number X_u from the uniform distribution between zero and one.

With this method for every variable a random value can be drawn from their corresponding distribution functions, leading to a vector which is used as input the limit state function. The resulting value of the vector is calculated and if this limit state function is smaller than zero failure occurs. By repeating this procedure a large number of times the probability of failure can be estimated as follows:

$$P_f \approx \frac{n_f}{n} \tag{B.3}$$

[1] Mstab is a previous version of D-Geo, which is software to calculate macro instability. It is developed by Deltares, for a more detailed descriptions see Section B.

in which:

n_f is the number of simulation for which the limit state function is smaller than zero
n is the total number of simulations

To obtain a result with a reliability of 95% the total number of simulations is:

$$n > 400(\frac{1}{P_f} - 1) \tag{B.4}$$

To determine the probability of failure for one cross section and one failure mechanism probabilistic tools (den Heijer, 2012) from OpenEarth are used. OpenEarth [2] is a free and open source initiative to deal with data, models and tools in earth science and engineering projects. Currently OpenEarth is mainly focussed on marine and coastal engineering. OpenEarth provides a platform to archive and host data, model systems and tools for practical analysis. OpenEarth is, amongst others, supported by the effort from professionals from Deltares, Delft University of Technology (Hydraulic Engineering and Environmental Fluid Mechanics sections), Van Oord Dredging and Marine Contractors, Arcadis-Alkylon and UNESCO-IHE.

The probabilistic tools in OpenEarth are developed for Matlab in which a probabilistic calculation can be done with a FORM or Monte Carlo analysis. Two scripts must be generated in Matlab:

- A function file to determine the stochastic variables.
- A function file which describes the limit state function.

In the first function file a structure with fields is generated:

- Name; a unique name for each stochastic variable
- Distr; function handle of the corresponding distribution function (e.g. @norm_inv)
- Params; parameters as input for the corresponding distribution function
- propertyName; indicate how to analyse variable in Z-function

The distribution functions that can be used are shown in Table B.1.

Distribution	Function handle	Parameters
Normal	@norm_inv	μ, σ
Exponential	@exp_inv	λ, ϵ
Triangular	@trian_inv	a, b, c
Lognormal	@logn_inv	μ, σ
Uniform	@unif_inv	a, b
Conditional Weibull	@conditionalWeibull	ω, ρ, α, σ
Deterministic	@deterministic	x

Table B.1: Available distribution function in the probabilistic tools in OpenEarth.

The hydraulic boundary conditions are random variables with a conditional Weibull distribution. According to VNK2 (Projectbureau VNK2, 2012) other variables have a normal or a lognormal distribution. Therefore these three distributions are used in the probabilistic tools. More information about the hydraulic boundary conditions and the conditional Weibull distribution can be found in Appendix A.

A normal distribution is defined by mean value μ and a standard deviation σ and the density function has a symmetrical shape. A lognormal distribution has an asymmetrical shape and has a minimum, contrary to a normal distribution which has no boundaries. Therefore the lognormal distribution is used for variables that can not be negative, like dimensions (length, width, height), model factors, coefficients and discharges. The different shapes of the the normal and lognormal distribution for the same parameters μ and σ are given in Figure B.1.

The input of the mean value μ and the standard deviation σ is in these probabilistic tools (and also in PC-Ring[3]) different for the lognormal distribution. The mean values and standard deviations for

[2]Source: http://publicwiki.deltares.nl/display/OET/OpenEarth
[3]PC-Ring is a software module used for VNK2 to calculate risks of flood defences in the Netherlands

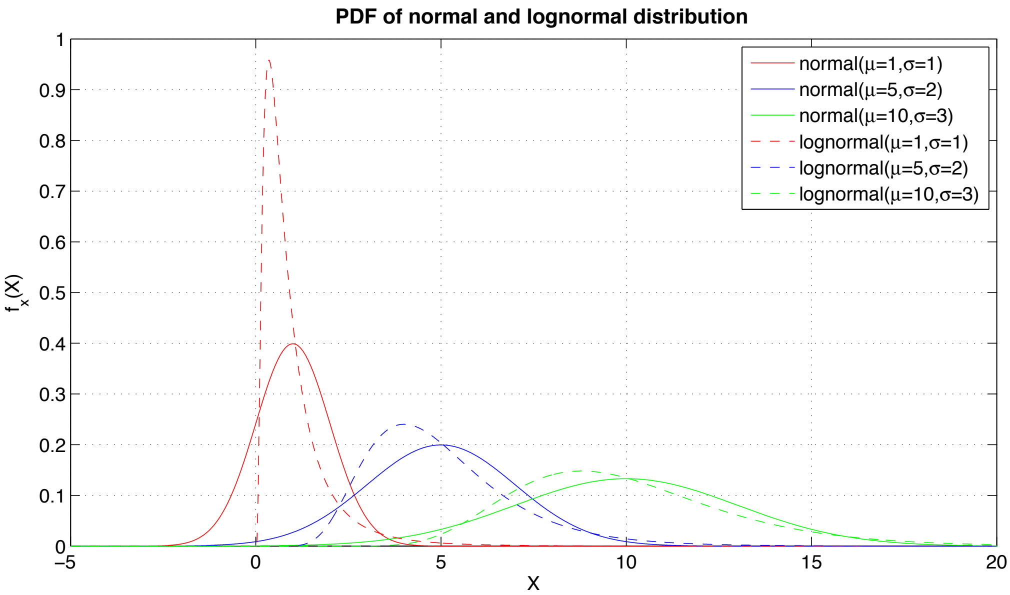

Figure B.1: Probability density functions for the normal and lognormal distribution functions with several parameters.

lognormal distributed variables are defined as values for the parent normal distribution in the VNK2 manual (Projectbureau VNK2, 2012). However, in the probabilistic tools in OpenEarth μ and σ must be defined according to the lognormal distribution. So the values from must be translated before they can be used. This translation is given by:

$$\mu = ln(\frac{m^2}{\sqrt{s^2 + m^2}}) \tag{B.5}$$

$$\sigma = \sqrt{log(\frac{s^2}{m^2} + 1)} \tag{B.6}$$

with:

- μ the mean value of the lognormal distribution used as input for the calculations
- σ the standard deviation of the lognormal distribution used as input for the calculations
- m the mean value
- s the standard deviation

If these values are not translated into the right values for the lognormal distribution this results in extreme values that are incorrect. During the MSc research extreme results were obtained for the probability of failure, especially for the failure mechanism instability of the revetment. Because a lot of variables are defined by a lognormal distribution in this failure mechanism the wrong input parameters resulted in extreme values for the strength parameters, which led to a very small probability of failure. After modifying the function file for the lognormal distribution (with the translation according to Equation B.5 and B.6 build into the function file) the results were realistic values. The extreme spread of the wrong input can be seen in Figure B.2 (especially in comparison to Figure B.1 were the same parameters are used).

In the second function file the limit state function is described which uses the samples from the variables that are defined in the previous function file. This limit state function file is defined as Z=samples.R - samples.S. The samples are loaded from the structure as input in this function file and the output is a single value for the Z-function. The number of samples can be defined to get an estimate of the probability of failure.

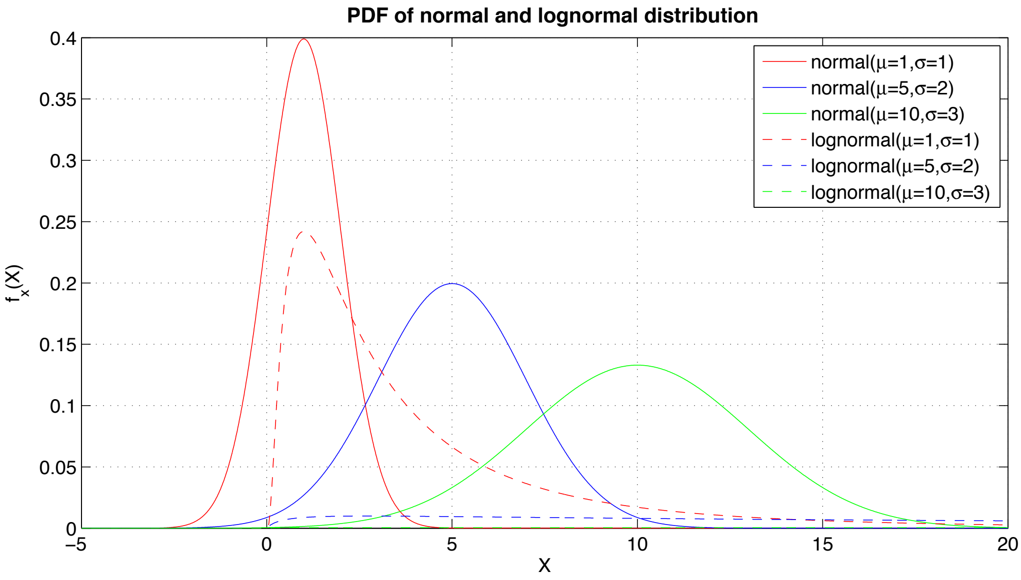

Figure B.2: Probability density functions for the normal and lognormal distribution without translated parameters.

Overtopping

The method for determining the amount of overtopping is described in (van der Meer, 2002). This method is also used in the PC-OVERSLAG software module, which is used in the VNK-project. The calculation method for wave run-up and wave overtopping are described as follows:

1. Determination of the wave boundary conditions at the toe of the dike (H_s and $Tm_{-1,0}$).
2. Calculation of the influence factor for the angle of wave impact (γ_β).
3. Adjust the wave boundary conditions if $\gamma_\beta > 80°$.
4. Calculation of the average slope ($tan\alpha$).
5. Calculation of the wave run-up without berm and roughness coefficients ($z_{2\%,smooth}$).
6. Calculation of the roughness coefficient (γ_f).
7. Calculation of the wave run-up with roughness coefficient ($z_{2\%,rough}$).
8. Calculation of the influence coefficient of the berm reduction (γ_b).
9. Calculation of the total wave run-up ($z_{2\%}$).
10. Calculation of the γ_β for wave overtopping.
11. Calculation of the wave overtopping with γ_f and γ_b.
12. Calculation of the overtopping volume per wave.

The relative wave run-up is defined as a function of the surf similarity paramater (or braking parameter), which is defined as:

$$\xi_0 = \frac{tan\alpha}{\sqrt{s_0}} \tag{B.7}$$

with:

$$s_0 = \frac{2\pi H_s}{gT_{m-1,0}^2} \tag{B.8}$$

in which:

ξ_0	[-]	surf similarity parameter
α	[∘]	angle of the slope
s_0	[-]	wave steepness
H_s	[m]	significant wave height
$T_{m-1,0}$	[s]	spectral wave period
g	[m/s^2]	gravitational acceleration

The average wave run-up is given by:

$$z_{2\%} = C_1 \gamma_b \gamma_f \gamma_\beta \xi_0 H_s \tag{B.9}$$

with a maximum of:

$$z_{2\%} = \gamma_f \gamma_\beta (\frac{C_2 - C_3}{\sqrt{\xi_0}}) H_s \tag{B.10}$$

in which:

$z_{2\%}$	[m]	level of 2% wave run-up above still water level
H_s	[m]	significant wave height
γ_b	[-]	influence factor for the berm reduction
γ_f	[-]	influence factor for the friction
γ_β	[-]	influence factor for the angle of wave attack
C_1	[-]	constant (=1.65 in a probabilistic calculation)
C_2	[-]	constant (=4.0 in a probabilistic calculation)
C_3	[-]	constant (=1.5 in a probabilistic calculation)

The total reduction is bounded to a minimum:

$$\gamma_b \cdot \gamma_f \cdot \gamma_\beta \geq 0.4 \tag{B.11}$$

The average angle of the slope is determined as follows:

$$tan(\alpha) = \frac{1.5H_s + z_{2\%}}{L_{slope}B} \tag{B.12}$$

in which:

L_{slope}	[m]	the horizontal length of the slope from $SWL - 1.5H_s$ to $SWL + z_{2\%}$
B	[m]	width of the berm

Because $z_{2\%}$ is not known in the beginning a first value is estimated at $z_{2\%} = 1.5H_s$. With an iteration the final value can be determined.

For the wave run-up and wave overtopping different reduction factors for the angle of wave attack apply. The reduction factor for the wave run-up is determined as:

$$\gamma_\beta = \begin{cases} 1 - 0.0022|\beta| & (0° \leq |\beta| \leq 80°) \\ 1 - 0.0022 \cdot 80 & (|\beta| \geq 80°) \end{cases} \tag{B.13}$$

The angle of wave attack reduction factor for wave overtopping is defined as:

$$\gamma_\beta = \begin{cases} 1 - 0.0033|\beta| & (0° \leq |\beta| \leq 80°) \\ 1 - 0.0033 \cdot 80 & (|\beta| \geq 80°) \end{cases} \tag{B.14}$$

The reduction factor of the berm depends on the total width of the berm and the level of the berm with respect to the still water level. The reduction factor of the berm is defined as:

$$\gamma_b = 1 - r_B(1 - r_{dh}) \tag{B.15}$$

with:

$$r_B = \frac{B}{L_berm} \tag{B.16}$$

and:

$$r_{dh} = 0.5 - 0.5cos(\pi \frac{d_h}{x}) \tag{B.17}$$

in which:

$d_h = SWL - H_B$		
$x = z_{2\%}$	if $z_{2\%} > -d_h > 0$	(berm above SWL)
$x = 2H_s$	if $2H_s > -d_h \geq 0$	(berm below SWL)
$r_{dh} = 1$	if $-d_h \geq z_{2\%}$ or $d_h \geq 2H_s$	(outside influence zone)

The reduction factor for the berm is bounded by $0.6 \leq \gamma_b \leq 1.0$.

The friction factor takes the friction of different materials into account that reduce the run-up and overtopping. Friction factors are determined empirical for different materials. If a slope consists of different type of materials (leading to different friction factors) the friction factor is determined as follows:

$$\gamma_f = \frac{\sum L_i \gamma_{f,i}}{\sum L_i} \tag{B.18}$$

in which the L_i is the length of section i with corresponding friction factor $\gamma_{f,i}$.

To determine the wave overtopping first a dimensionless wave overtopping is calculated. There are different dimensionless wave overtopping formulas for breaking and non-breaking waves. The formulas for the dimensionless wave overtopping are:

$$Q_b = \frac{0.067}{\sqrt{tan\alpha}} \gamma_b \xi_0 exp(-C_4 \frac{h_k}{H_s} \frac{1}{\xi_0 \gamma_b \gamma_f \gamma_\beta}) \tag{B.19}$$

$$Q_n = 0.2 exp(-C_5 \frac{h_k}{H_s} \frac{1}{\gamma_f \gamma_\beta}) \tag{B.20}$$

in which:

Q_b	[-]	dimensionless wave overtopping discharge for breaking waves
Q_n	[-]	dimensionless wave overtopping discharge for non-breaking waves
C_4	[-]	normally distributed constant (μ=5.2 and σ is 0.55)
C_5	[-]	normally distributed constant (μ=2.6 and σ is 0.35)
h_k	[m]	height of the crown of the dike

The overtopping discharge q_0(with a dimension of $m^3/m/s$) is given by:

$$q_0 = min(Q_b, Q_n)\sqrt{gH_s^3} \tag{B.21}$$

A first calculation is made for a typical cross section of the Afsluitdijk. The used variables and their distributions and parameters are shown in Table B.2. These variables are defined in the structure function file in matlab. The matlab script which describes the limit state function is given in Appendix E.

The critical overtopping discharge can be calculated with a grass strength model (Steenbergen et al., 2007), which uses a relationship according to a CIRIA-research. The formula for the critical overtopping discharge is given by:

$$q_c = \frac{v_c^3}{tan(\alpha_i)C^2} \tag{B.22}$$

in which v_c is the critical flow velocity, α_i the angle of the inner slope and C the Chezy roughness factor. The roughness factor on the inner slope is determined by:

$$C = 25\left(\frac{q_c}{kv_c}\right)^{1/6} \tag{B.23}$$

in which k is the Strickler roughness factor. An alternative for the roughness factor is the Manning factor n. The relationship between the Manning factor and the Strickler factor is given by:

$$n = \frac{k^{1/6}}{25} \tag{B.24}$$

Symbol	Unit	Name	Distribution	Parameters
β	$\circ$	angle of wave attack	norm_inv	$\mu = 2$ $\sigma = 0.5$
mHm_0	$-$	modelfactor wave height	norm_inv	$\mu = 1$ $\sigma = 0.15$
mTm_0	$-$	modelfactor wave period	norm_inv	$\mu = 1$ $\sigma = 0.15$
SWL	m	still water level	conditionalWeibull	$\omega = 2.36$ $\rho = 1.9074$ $\alpha = 1.7548$ $\sigma = 1.1447$
α_{upper}	$1/tan(\circ)$	upper slope	norm_inv	$\mu = 3.5$ $V = 0.05$
α_{lower}	$1/tan(\circ)$	lower slope	norm_inv	$\mu = 3.5$ $V = 0.05$
g	m/s^2	gravitational acceleration	deterministic	$x = 9.81$
H_b	m	berm height	norm_inv	$\mu = -0.05$ $\sigma = 0.1$
B	m	berm width	norm_inv	$\mu = 7$ $\sigma = 0.15$
H_{crown}	m	crown height	norm_inv	$\mu = 7.9$ $\sigma = 0.1$
γ_f	$-$	friction coefficient	deterministic	$x = 1$
C_1	$-$	coefficient	deterministic	$x = 1.65$
C_2	$-$	coefficient	deterministic	$x = 4$
C_3	$-$	coefficient	deterministic	$x = 1.5$
C_4	$-$	coefficient	norm_inv	$\mu = 5.2$ $\sigma = 0.55$
C_5	$-$	coefficient	norm_inv	$\mu = 2.6$ $\sigma = 0.35$
mq_c	$-$	modelfactor critical overtopping discharge	logn_inv	$\mu = 1$ $\sigma = 0.5$
mq_0	$-$	modelfactor overtopping discharge	logn_inv	$\mu = 1$ $\sigma = 0.5$

Table B.2: Parameters used to determine failure due to overtopping.

Equation B.22 and B.23 combined lead to the following formula:

$$q_c = \frac{v_c^{5/2} k^{1/4}}{125tan(\alpha_i)^{3/4}} \tag{B.25}$$

The critical flow velocity v_c that results into failure of the grass cover after a certain amount of time t_e is given by:

$$v_c = f_g \frac{3.8}{(1 + 0.8^{10} logt_e)} \tag{B.26}$$

in which f_g is a factor for the quality of the grass, varying from 0.7 for bad grass up to 1.4 for good quality grass. The quality of the grass and the erosion resistance are strongly correlated, therefore a relationship between c_g and f_g is used. This relationship is given by:

$$c_g = 6 \cdot 10^5 f_g^{1.5} \Leftrightarrow f_g = \left(\frac{c_g}{6 \cdot 10^5}\right)^{2/3} \tag{B.27}$$

The needed time for failure t_e is given by:

$$t_e = P_t t_{RT,inner} \tag{B.28}$$

in which P_t is the percentage of time where overflow/overtopping occurs and $t_{RT,inner}$ stands for erosion resistance of the grass cover, which is defined by:

$$t_{RT,inner} = \frac{c_g d_w}{c_g d_w + (0.4c_{RK} L_{K,inner})} t_s \tag{B.29}$$

in which d_w is the root length of the grass, c_{RK} is a factor for the erosion resistance of the clay cover, $L_{K,inner}$ is the width of the clay cover and t_s is the storm duration.

A first calculation is made for a typical cross section of the Afsluitdijk (section 8b). The used variables and their distributions and parameters are shown in Table B.3. These variables are defined in the structure function file in matlab. The matlab script which describes the critical overtopping discharge is given in Appendix E.

Symbol	Unit	Name	Distribution	Parameters
f_g	$-$	erosion resistance grass	norm_inv	$\mu = 0.7\ \sigma = 0.05$
d_w	m	height grass roots	norm_inv	$\mu = 0.3\ \sigma = 0.01$
c_{RK}	ms	erosion resistance clay cover	norm_inv	$\mu = 23000\ \sigma = 100$
$L_{K,inner}$	m	length of clay cover	norm_inv	$\mu = 5\ \sigma = .2$
k	m	Strickler roughness factor	logn_inv	$\mu = 0.015\ V = 0.25$
α_i	$1/tan(\circ)$	inner slope	norm_inv	$\mu = 3.5\ V = 0.05$
t_s	s	storm duration	logn_inv	$\mu = 7.5 * 3600\ V = 0.25$

Table B.3: Parameters used to determine critical overtopping discharge.

Stone revetment failure and erosion outer slope

Wave attack can cause erosion of the outer slope. This erosion can result in collapse of a dike. If the wave attack is not too big a grass cover on a clay layer is often used. In case of larger waves the dike is protected against wave attack by a stone or asphalt revetment. The total failure depends therefore on failure of the revetment/grass cover and the erosion of the dike body, which results in two limit state functions. One for the erosion of the dike body and, if applicable, one for the failure of the revetment. To calculate the probability of failure for the Afsluitdijk the mechanisms and corresponding limit state functions that are used are described in the VNK2 manual (Steenbergen et al., 2007).

There are three types of covers that can be calculated according to this model:

1. grass cover
2. placed stone block revetment
3. asphalt cover

For the Afsluitdijk the normative cross section with the highest probability of failure is constructed with a placed stone (basalt) block revetment. For this type of cover a distinction is made between the placement of the blocks directly on clay or with an granular filter in between. In case of the Afsluitdijk a granular filter is present so only the method for this type of revetment is described.

The transition between different types of revetments is a weak spot in the cover. In this MSc research the transitions between the different types of revetments is not taken into account. The results of this model are compared to a calculation with PC-Ring, a software module that is used in the VNK-method. Because these transitions are not taken into account in this model they are neglected in this MSc research as well to be able to compare the results.

In this Msc research the weakest revetment area for each cross section determines the probability of failure. The total probability of failure will be higher in reality because of the transitions in the different types of revetments.

Failure of the revetment

As mentioned earlier failure occurs if both the cover fail and the dike body will erode. First we look at the failure of the stone block revetment. To prevent failure the stones must meet two conditions. The first condition is described by the following limit state function:

$$Z_f = \frac{c_f \Delta D^{1.67} \Gamma^{1.67}}{(\Lambda tan(\alpha_u))^{0.67}} - \frac{rH_s r_h}{(rS_{op})^{0.33}} \tag{B.30}$$

in which:

c_f	[-]	coefficient for the strength of the placed stone revetment
Δ	[-]	relative density
D	[m]	thickness of the stone revetment
Γ	[-]	influence factor for the friction between the stones, flow and inertia
Λ	[m]	leakage length
α_u	[°]	angle of outer slope
S_{op}	[-]	wave steepness

The wave steepness S_{op} is defined by:

$$S_{op} = \frac{2\pi H_s}{gT_p^2} \tag{B.31}$$

In which T_p is the peak wave period and g is the gravitational acceleration. The relative density is given by:

$$\Delta = \frac{\gamma_s - \gamma_w}{\gamma_w} \tag{B.32}$$

In which γ_s is the density of the stone blocks and γ_w the density of water. The leakage length is given by the following equation:

$$\Lambda = \sqrt{kd_f D/k'} \tag{B.33}$$

with:

$$k = \frac{-a_f + \sqrt{a_f^2 + 4b_f}}{0.6b_f}$$

$$k' = c_f \frac{-a' + \sqrt{a'^2 + 4b'}}{2b'}$$

$$a_f = c_a \frac{160\nu(1-n)^2}{gn^3 D_{f15}^2}$$

$$b_f = c_b \frac{2.2}{gn^2 D_{f15}}$$

$$a' = \frac{12\nu l}{gs^2} + \frac{lsa_f}{\pi D} ln\left(\frac{ls}{\pi e r_{min}}\right)$$

$$b' = \frac{l^2}{2gD}\left(\left(\frac{1}{n} - 1\right)^2 + 1\right) + \frac{lsb_f}{\pi D}\left(\frac{ls}{\pi r_{min}} - 2\right)$$

$$l = \frac{BL}{Bs + Ls}$$

$$r_{min} = max(0.5D_{f15}; 0.4s)$$

in which:

c_a	[-]	coefficient for the determination of the leakage length
c_b	[-]	coefficient for the determination of the leakage length
c_t	[-]	coefficient for the determination of the leakage length
B	[m]	width of the stone blocks
L	[m]	length of the stone blocks
s	[m]	width of the splits between the stones
n	[-]	porosity of the filter
d_f	[m]	thickness of the filter layer
D_{f15}	[m]	grain size of the 15 % weight fraction of the filter material
ν	[m^2/s]	kinematic viscosity of water

To meet the second condition for the stone blocks the following limit state function is defined:

$$Z_b = c_{gf}\left(\frac{tan(\alpha_u)}{\sqrt{S_{op}}}\right)^{-2/3} - \frac{H_s r_h}{\Delta D} \tag{B.34}$$

in which $c_g f$ is a coefficient for the strength of the placed stone block revetment. The other variables in this formula have been described for the limit state function Z_f.

A first calculation is made for a typical cross section of the Afsluitdijk. The used variables and their distributions and parameters are shown in Table B.4. These variables are defined in the structure function file in matlab. The matlab script which describes the limit state function is given in Appendix E.

Symbol	Unit	Name	Distribution	Parameters
c_{gf}	$-$	coefficient	logn_inv	$\mu = 5.2$ $V = 0.124$
γ_s	kg/m^3	density stone blocks	norm_inv	$\mu = 2900$ $V = 0.02$
γ_w	kg/m^3	density water	norm_inv	$\mu = 1025$ $V = 0.02$
Γ	$-$	coefficient	deterministic	$x = 1$
α_u	$\circ$	upper slope	norm_inv	$\mu = 14.322$ $V = 0.05$
H_s	m	significant wave height	conditionalWeibull	$\omega = 2.36$ $\rho = 1.9074$ $\alpha = 1.7548$ $\sigma = 1.1447$
g	m/s^2	gravitational acceleration	deterministic	$x = 9.81$
c_a	$-$	coefficient	logn_inv	$\mu = 1$ $V = 0.35$
c_b	$-$	coefficient	logn_inv	$\mu = 1$ $V = 0.35$
c_t	$-$	coefficient	logn_inv	$\mu = 1$ $V = 0.35$
D	m	thickness stone blocks	logn_inv	$\mu = 0.4$ $V = 0.03$
l	$-$	inverse open area in stone revetment	deterministic	$x = 8.33$
s	m	width of the splits	logn_inv	$\mu = 0.015$ $V = 0.16$
n	$-$	porosity of filter	deterministic	$x = 0.35$
d_f	m	thicknes filter layer	logn_inv	$\mu = 0.15$ $V = 0.15$
D_{f15}	m	15% weight fraction grain size of filter	logn_inv	$\mu = 0.02$ $V = 0.1$
ν	m^2/s	kinematic viscosity of water	deterministic	$x = 1.2 \cdot 10^{-6}$

Table B.4: Parameters used to determine failure of the stone placed blocks on a filter.

Erosion of the dike body

Before failure occurs the dike body itself has to be eroded as well. This strongly depends on the storm duration and the properties of the dike body. The limit state function of the erosion of the dike body is given by:

$$Z = t_{RS} + t_{RK} + t_{RB} - t_s \tag{B.35}$$

in which t_{RS} is the residual strength of the stone blocks on a granular filter, t_{RK} the residual strength of the clay layer, t_{RB} the residual strength of the dike body and t_s the storm duration. The residual strength for the stone blocks on the granular filter is given by:

$$t_{RS} = 57 \cdot 10^3 T_p e^{-\sqrt{H_s L_{op}}/c} \tag{B.36}$$

in which T_p is the wave peak period, H_s is the significant wave height, L_{op} is the wave length and a coefficient c. The wave length L is given by:

$$L_{op} = \frac{g}{2\pi} T_p^2$$

The residual strength of the clay cover is given by:

$$t_{RK} = \frac{0.4 L_K c_{RK}}{r^2 H_s^2} \tag{B.37}$$

in which L_K is the width of clay cover, c_{RK} is an erosion resistance coefficient and r is a reduction factor.

The residual strength of the dike body itself is defined by:

$$t_{RB} = \frac{0.4 L_B c_{RB}}{r^2 H_s^2} \tag{B.38}$$

in which L_B is the width of the dike body at the occurring water level, c_{RB} is an erosion resistance coefficient and r is a reduction factor. The width of the dike core L_B is defined by the horizontal distance between the point where the erosion breaks through the clay cover and the inner crown point. It is given by:

$$L_B = \frac{h_k - h + 0.25H_s}{tan(\alpha)} - L_K + B \geq B \tag{B.39}$$

In which h_k is the height of the brown and h the still water level, both with respect to NAP. In the rudimentary erosion model an adaption to the the factor c_{RB} is proposed (Steenbergen et al., 2007). This coefficient is defined by the the erosion of the covering clay layer:

$$c_{RB} = \frac{c_{RK}}{v_{ZB}} \tag{B.40}$$

in which v_{ZB} is an acceleration factor for the erosion speed of the core with respect to the clay cover, because the core consists of sand. This factor can be modeled as:

$$v_{ZB} = (1 + \alpha_Z r_Z)h_Z \tag{B.41}$$

with α_Z a coefficient for the acceleration of the erosion, r_Z the average amount of sand in the erosion profile of the dike and h_Z a retardation factor. The factor α_Z is determined by the type of material of the inner core by:

$\alpha_Z = 0$ for the same quality of clay in the core
$\alpha_Z = 2$ for a lower quality of clay in the core
$\alpha_Z = 6$ for a sand core

The amount of sand in the erosion profile r_Z is formulated by the following formula:

$$r_Z = \frac{0.5tan(\alpha)(L_B^2 - B^2)}{0.5tan(\alpha)(L_B^2 - B^2) + \{(L_B - B)/cos(\alpha) + B\}\, dk} \geq 0 \tag{B.42}$$

in which B is the width of the dike crown and d_k is the thickness of the clay cover. The retardation factor h_Z takes into account that the erosion process will take longer if the total volume that has to be eroded is larger. This partly depends on the hight of the dike with respect to the SWL. The retardation factor is formulated by:

$$h_Z = \alpha_h \frac{L_B d_k/cos(\alpha)}{0.5tan(\alpha)(L_B^2 - B^2) + \{(L_B - B)/cos(\alpha) + B\}\, dk} \geq 1 \tag{B.43}$$

in which α_h is a factor in which the erosion speed decreases with the hight.

A first calculation is made for a typical cross section of the Afsluitdijk. The used variables and their distributions and parameters are shown in Table B.5. These variables are defined in the structure function file in Matlab. The Matlab script which describes the limit state function is given in Appendix E.

Symbol	**Unit**	**Name**	**Distribution**	**Parameters**
c	$-$	coefficient	logn_inv	$\mu = 2.4\ \sigma = 0.43$
α_u	$\circ$	upper slope	norm_inv	$\mu = 15.642\ V = 0.05$
H_s	m	significant wave height	conditionalWeibull	$\omega = 2.36\ \rho = 1.9074\ \alpha = 1.7548\ \sigma = 1.1447$
g	m/s^2	gravitational acceleration	deterministic	$x = 9.81$
c_{RK}	$-$	coefficient	logn_inv	$\mu = 10000\ V = 0.3$
L_K	[m]	length clay cover	logn_inv	$\mu = 5\ \sigma = 0.2$
t_s	[s]	storm duration	logn_inv	$\mu = 7 \cdot 3600\ V = 0.25$
L_{KB}	[m]	width dike crown	logn_inv	$\mu = 3\ \sigma = 0.2$
α_h	[-]	retardation factor	logn_inv	$\mu = 0.5\ \sigma = 0.3$
h_k	[m]	height of the crown	norm_inv	$\mu = 7.84\ \sigma = 0.1$

Table B.5: Parameters used to determine failure due to erosion of dike body.

Macro instability (inner and outer slope)

In this failure mechanism a dike fails because a part of the dike becomes unstable and it slides away. This phenomenon can occur both on the inner and outer slope of a dike. Normally only failure of the inner slope is taken into account because instability usually occurs on the side with low water levels (and high water levels inside the dike itself). Failure of the outer slope may occur if after periods of high water the water level drops fast and the remaining water inside the dike body will cause overpressure which can result in instability.

To calculate the probability of failure due to macro instability M-Prostab is used. This is a software module developed by Deltares that is used to calculate the probability of failure with the Bishop method. The slip circle method of Bishop is based on a moment equilibrium of the whole slip circle and vertical force equilibrium of the individual slices. The Bishop method considers the driving moments of the slices by soil weight, water pressures and loads around the center of a slip circle (see Figure B.3).

Stability requires that the sum of these driving moment is equal to (or less than) a certain resisting moment. This resistance moment is determined by the shear strength of the soil along the slip circle (and a possible additional contribution from geotextiles).

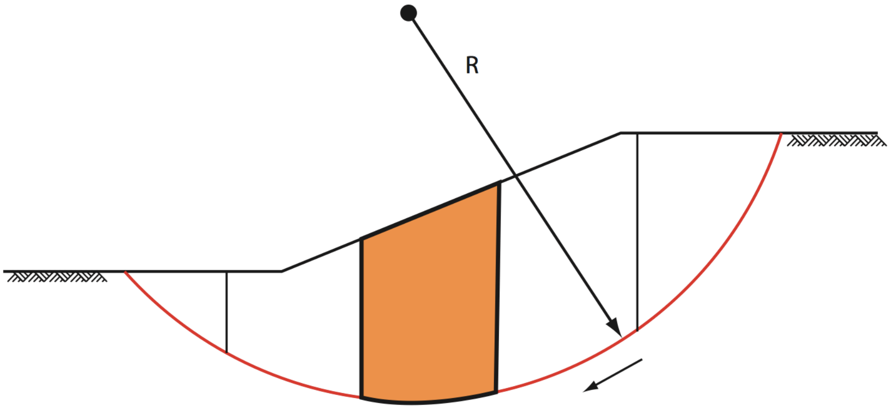

Figure B.3: Slip plane with the vertical slices according to the Bishop method.

The center of the slip circle and the radius of the slip circle determine the safety factor. To find the normative slip circle radius and center calculation must be made until a lowest value is found. With the use of M-Prostab a starting grid of the center of the slip circle and different horizontal slip lines (which determine the radius of the slip circle) are defined to start a calculation. If a lowest safety factor is found at the edges of the center grid or at the boundary of horizontal lines M-Prostab automatically extends the ranges. This will guarantee the slip circle with the lowest safety factor will be found.

The shear strength is based on drained parameters for the cohesion c' and internal angle of friction ϕ' according to the formula:

$$\tau = c' + (\sigma - u)tan(\phi') \tag{B.44}$$

in which τ is the shear strength along the bottom of a slice, σ the total normal pressure on the bottom of a slice and u the water pressure. The shear strength parameters c' (cohesion) and ϕ' (internal friction) can be determined by triaxial tests, cel tests and direct or simple shear tests.

In a probabilistic calculation the hydraulic boundary conditions are stochastics. In M-Prostab however a calculation is made for one water level with a corresponding probability of failure. The other variables such as soil parameters are defined as stochastic values, but for each water level a new calculation has to be made.

According to the VNK method (Projectbureau VNK2, 2012) the shear strength parameters must be converted before they are used in M-Prostab. For a correct determination of the distribution of the strength parameters original unmodified lab results are needed. In practice these results are mostly not available and therefore the parameters that are available must be adapted.

The shear strength calculated from averages, standard deviations and a correlation coefficient is in general non-linear, while the characteristic shear strength, which is normally represented by characteristic values of the cohesion and angle of internal friction, is linear. This results in an error between the starting characteristic value and the characteristic $\tau - \sigma$ relation based on averages and standard deviations.

Two spreadsheets are developed which modify the shear strength parameters so they can be used in M-Prostab. These spreadsheets are focussed on determining the average values and standard deviations of c and ϕ with a minimum error. A detailed description of this conversion is given in (Bakker, 2004).

For the normative cross section calculations are made for water levels from NAP up to the water level that occurs with a probability of 1/10 000 per year, with an interval of one meter. For the maximum water level the slip circles for the instability of the inner and outer slope are given in Figure B.4 and B.5.

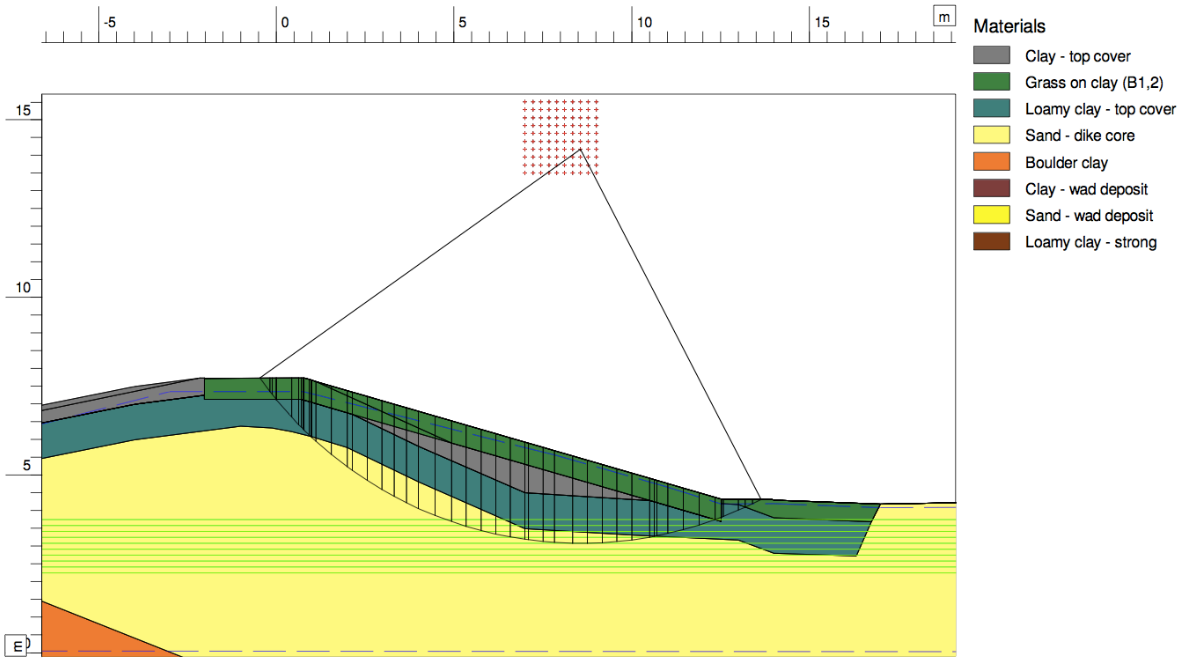

Figure B.4: Slip plane for the inner slope with hydraulic boundary conditions with a probability of 1/10.000 per year.

Piping and heave

Piping is a micro-instability process as small particles are eroded from underneath a cohesive layer in the subsoil. The seepage water erodes particles from underneath the dike if the hydraulic head is large enough. The erosion forms a pipe which grows from the inner to the outer side and this can lead to a settlement and failure of the dike. This mechanism can only occur underneath a cohesive layer (clay) on top of an erodible and permeable soil (sand).

The assumption is that the probability of failure of the Afsluitdijk due to piping is so small it can be neglected. In this section this assumption is verified. The assessment method for piping has changed and older rules that have been used in the past are no longer valid. Therefore it is desirable to assess the Afsluitdijk on piping and heave (without detailed calculations) with the new assessment standard.The physics of the failure mechanisms of piping and heave and the assessment and design methods are described in (Forster et al., 2013).

To assess a soil structure or hydraulic structure on piping several formulas are used. Three different formulas are developed by Bligh, Lane and Sellmeijer. All these formulas are based on three main variables:

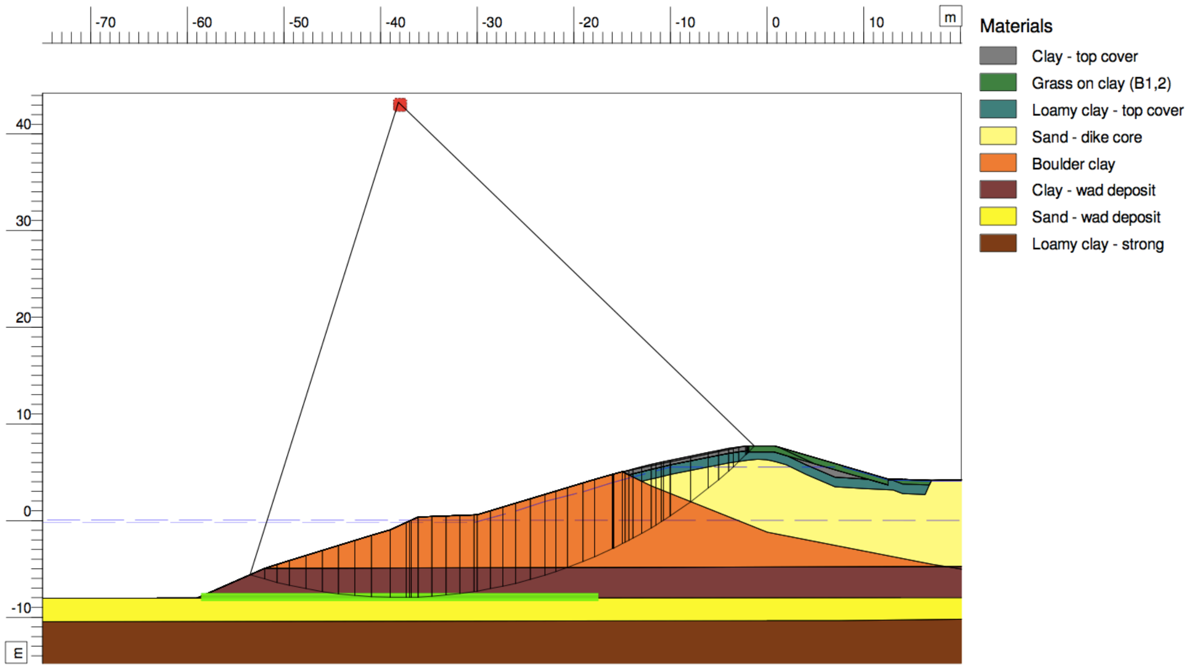

Figure B.5: Slip plane for the outer slope with hydraulic boundary conditions with a probability of 1/10.000 per year.

- The head (difference in water level) over the structure.
- The length of the aquifer from the high water entrance point to outflow point (mostly were the clay cover is the smallest).
- The flow resistance of the aquifer.

A condition for the occurrence of piping is that the critical uplifting force has to be reached. This uplifting is defined by the ground pressure at the bottom of the covering layer divided by the water force at the bottom of covering layer caused by the hydraulic head. If the pressure of the hydraulic head is larger than the downward force of the covering layer uplifting and piping can occur.

The assessment of piping according to the empirical formula of Bligh is as follows:

$$\Delta H \leq \Delta H_c = \frac{L}{C_{creep}} \tag{B.45}$$

in which:

ΔH	[m]	occurring hydraulic head over structure
ΔH_c	[m]	critical hydraulic head over structure
L	[m]	total seepage length
C	[-]	creep factor

The creep factor depends on the soil layer characteristics for which piping is assessed. This creep factor deviates from ± 4 (for coarse material) to ± 18 (for fine sand and silt). According to Bligh the total seepage length is taken into account, both vertical and horizontal length. This means that in case of seepage screens the total length of these screens is taken into account. According to Lane Bligh's method was not accurate. Lane stated that the vertical seepage length has a relatively larger influence than the horizontal length. The formula of Lane is as follows:

$$\Delta H \leq \Delta H_c = \frac{(\frac{1}{3}L_h + L_v)}{C_{creep}} \tag{B.46}$$

In which the horizontal seepage length is given by L_h and the vertical seepage length by L_v.

In the Dutch design philosophy the formula of Bligh was recommended because in the dikes no vertical seepage screens were used. The method of Bligh was assumed to be a safe simple estimation for the possible piping. However this method and especially the creep factor do not have any probabilistic support. Also if there were no data available of the soil the creep factor was assumed to be 18 to guarantee a conservative (and safe) design.

However research[4] showed that the rule of Bligh is not conservative but even less safe than the method of Sellmeijer, which was used in detailed assessment. Therefore the method of Bligh is no longer in use. The assessment of piping is based on the model of Sellmeijer for which an adapted calculation rule is derived. The method of Sellmeijer is based on representative parameters which means that sufficient data of soil parameters must be available. The criterium for the assessment on piping is as follows:

$$\frac{\Delta H_c}{\gamma_n \cdot \gamma_b} > (\Delta H - 0.3d) \tag{B.47}$$

with:

$$\Delta H_c = L \cdot F_{resistance} \cdot F_{scale} \cdot F_{geometry} \tag{B.48}$$

$$F_{resistance} = \frac{\gamma_p'}{\gamma_w}\{\eta tan(\theta)\} \tag{B.49}$$

$$F_{scale} = \frac{d_{70m}}{\sqrt[3]{\kappa L}}\left(\frac{d_{70}}{d_{70m}}\right)^{0.4} \tag{B.50}$$

$$F_{geometry} = 0.91 \cdot \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8}-1}+0.04} \tag{B.51}$$

in which:

ΔH_c	[m]	critical hydraulic head over structure
ΔH	[m]	occurring hydraulic head over structure
d	[m]	thickness of cover layer
γ_n	[-]	safety factor
γ_b	[-]	schematization factor
γ_p'	[kN/m^3]	($\gamma_p' = \gamma_p - \gamma_w$) apparent density sand in water
γ_p	[kN/m^3]	density of sand
γ_w	[kN/m^3]	density of water
θ	[°]	rolling resistance angle of sand
η	[-]	White coefficient
κ	[m^2]	intrinsic permeability sand
k	[m/s]	specific permeability of the upper sand layer
ν	[m^2/s]	kinematic viscosity of water
g	[m/s^2]	gravitational accelaration
d_{70}	[m]	70 percent value of grain size distribution of sand
d_{70m}	[m]	70 percent value of grain size distribution of sand in model scale tests
D	[m]	thickness of sand layer
L	[m]	horizontal seepage length

The formulation of $F_{geometry}$ is only applicable in case of standard dike geometries. If non-standard geometries have to be assessed advanced software (Mseep) must be used to calculate $F_{geometry}$.

The rule of Bligh that has been used for the design and assessment of many flood defences in the Netherlands is now longer applicable and the rule of Sellmeijer is also updated so results obtained with the old Sellmeijer rule are no longer valid. A first simplified assessment is done to analyze the overall safety of the structure. In this simplified analysis at least the following parameters are needed:

- The maximum head over the structure.
- The overall layout of the subsoil, defined in different layers of aquifers and aquitards.
- Geometry of the (soil) structure.
- Minimal thickness and density of the aquiclude (low water permeability layer) on top of the aquifer.

[4] SBW Piping research program to investigate piping and heave mechanisms and improve standards.

For the Afsluitdijk the assumption is that piping is not an issue because the probability of failure due to piping is very low compared to the other failure mechanisms. To assess if this assumption is valid a calculation for a normative cross section is made. The geometry, soil parameters and hydraulic boundary conditions that are used are described in the boundary conditions report for the design of the alternatives of the Afsluitdijk by Witteveen + Bos (Witteveen + Bos, 2013). An overview of a the normative cross section is shown in Figure B.6.

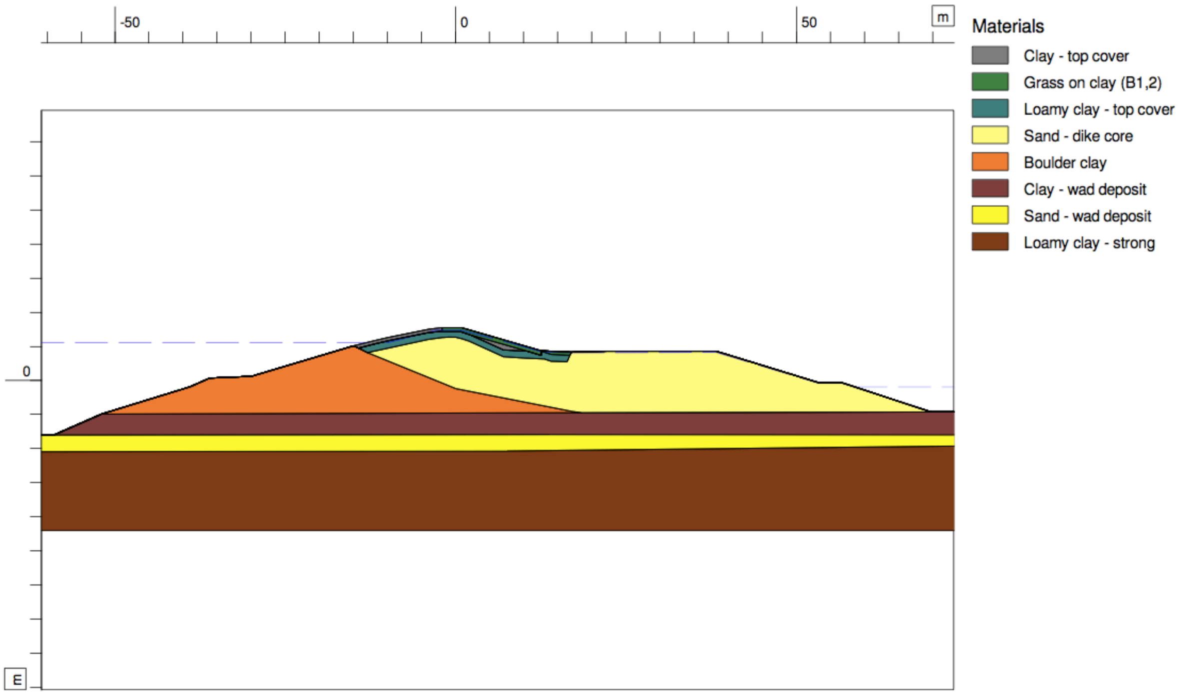

Figure B.6: Normative cross section of the Afsluitdijk including subsoil structure.

A condition for the occurrence for piping is that heave can occur, which is defined by the weight of the covering layer and the hydraulic head. Heave (and piping) can occur if the following condition is met:

$$\Delta H \geq \frac{\gamma_s - \gamma_w}{\gamma_w} d \tag{B.52}$$

According to (Witteveen + Bos, 2013) the maximum hydraulic head ΔH that occurs with a probability of 1/10 000 per year is 6.46 m. The density of the covering clay layer γ_s is equal to 1 800 kg/m^3, the density of water γ_w is 1025 kg/m^3 and the thickness of the covering clay layer d is equal to 3.5 m. With these values the condition for heave is met so a piping calculation is necessary.

$$6.46 \geq \frac{1800 - 1025}{1025} 3.5 = 2.65 \tag{B.53}$$

The calculation of piping is given by Equation B.48. The parameters that are used as input in this formula follow from (Witteveen + Bos, 2013) and (Forster et al., 2013). The values are given in Table B.6.

Equation B.48 can also be defined as a safety factor by dividing the left part of the equation by the right part. With the variables from Table B.6 the total safety factor for a hydraulic head with a probability of 1/10.000 per year is equal to:

$$SF = \frac{\Delta H_c/(\gamma_n \cdot \gamma_b)}{\Delta H - 0.3d} \approx 5$$

Symbol	Unit	Value
ΔH	[m]	6.46
d	[m]	3.5
γ_n	[-]	1.12
γ_b	[-]	1.1
γ_p	[kN/m^3]	27
γ_w	[kN/m^3]	10.05
θ	[°]	37
η	[-]	0.25
k	[m/s]	$3.5 \cdot 10^{-5}$
ν	[m^2/s]	$1.33 \cdot 10^{-6}$
g	[m/s^2]	9.81
d_{70}	[m]	$3 \cdot 10^{-4}$
d_{70m}	[m]	$2.8 \cdot 10^{-4}$
D	[m]	2
L	[m]	125

Table B.6: Values for variables in the piping calculation.

With this safety factor it is shown that piping is not an issue for the Afsluitdijk because it will fail due to other failure mechanisms before piping will occur. Therefore the assumption that piping is not a relevant failure mechanisms for the Afsluitdijk is valid.

C

Hydraulic structures

Outlet sluices

As we can see in Figure 4.1 there are two sluice complexes at Den Oever (Stevinsluizen)and Kornwerderzand (Lorentzsluizen). The reason that the Afsluitdijk has two sluice complexes is from a military point of view. The idea is that two complexes spread the risk in case of an attack. Including with defense mechanisms (Kazematten[1]) the choice for two complexes provides a higher safety level in case of a war.

Both the sluice complexes consist of outlet sluices and navigation locks. The outlet sluices have two main functions:

- Discharge water from the IJssel Lake to the Wadden Sea.
- Retain water from the Wadden Sea.

The River IJssel can be accounted for almost 90% of the free water supply to Lake IJssel. The remaining 10% consists of precipitation on the Lake, surplus water from the surrounding Lakes and water that is discharged out of the surrounding polders by pumping stations. Only a small part of the water in Lake IJssel is evaporated. Besides evaporating the surrounding area takes water from the Lake at scheduled times, but most of the water is discharged through the outlet sluices. These outlet sluices are used to regulate the water level on Lake IJssel. The water level on the IJssel Lake has a large influence on the safety level of the surrounding flood defences. The regulation of the water level on Lake IJssel is therefore very important for the safety against flooding of the surrounding areas of Lake IJssel.

Discharging through the outlet sluices is done by free fall. This means that the water flows from the Lake IJssel to the Wadden due to a difference in water level. During low tide at the Wadden Sea fresh water from Lake IJssel flows through the outlet sluices into the Wadden Sea. Because salt water is heavier than fresh water salt water can flow underneath the outlet flow into Lake IJssel. To prevent this from happening discharging is only done during decreasing tide.

If the outlet sluices are closed water is retained from the Wadden Sea. The water level on the Wadden Sea is higher than the water level on Lake IJssel in case of high tide and/or wind set-up. Also in extreme conditions (storm surges) the outlet sluices are able to retain the water from the Wadden Sea. In Figure C.1 a cross section of the outlet sluices is shown in an open and closed situation.

The outlet sluice complexes consist of sluice groups. Each group consists of five outlet sluices. At Den Oever three groups form a total of fifteen outlet sluices and at Kornwerderzand two groups form a total of 10 outlet sluices. Besides the foundation all these outlet sluices are identical. In Figure C.1 a cross section of an outlet sluices is shown. Each sluice has two doors and four lifting towers (two at both sides of the sluice). The two doors can be operated independently from each other. The northern doors at the Wadden Sea side (left in Figure C.1) have a certain open pattern that is designed to reduce the impact of the waves. This reduces the wave loads on the doors on the right. Because the wave

[1] A kazemat is similar to a bunker.

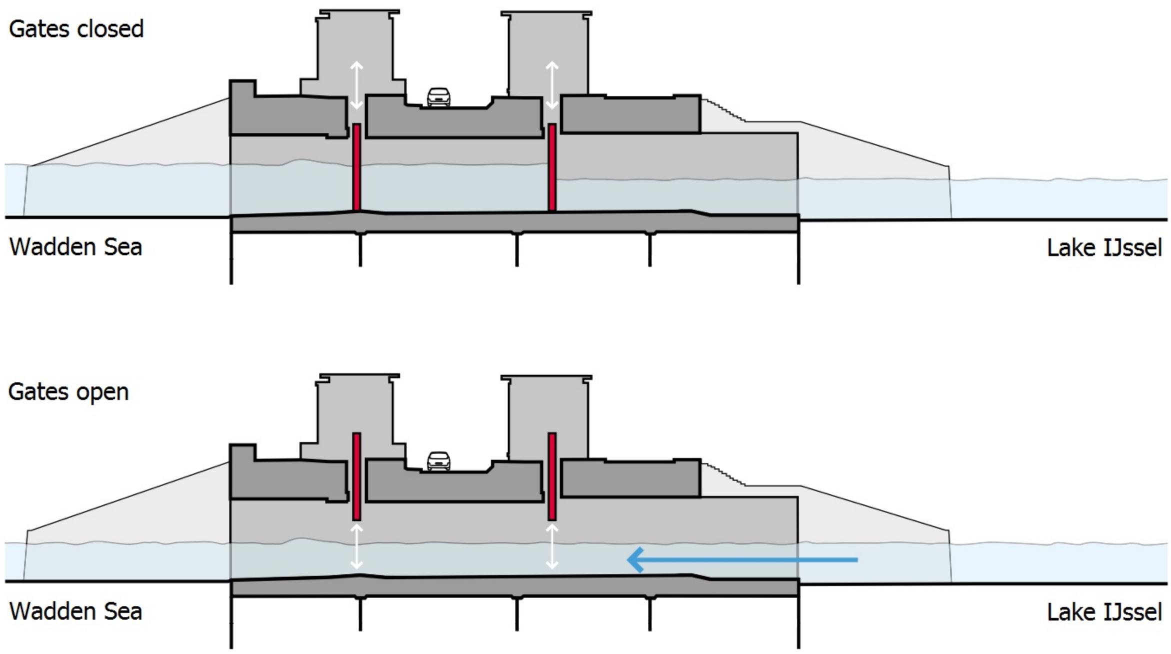

Figure C.1: Cross section of an outlet sluice in closed and open condition.

load is reduced significantly these doors only have to withstand the load of the difference in water level.

Locks

At Den Oever and Kornwerderzand navigation locks are situated that are open to commercial and recreational vessels. Besides the passage of ships the locks are also part of the primary flood defense. At both the navigation locks an outer harbor is situated with dikes at both sides of the harbor. On the side of Lake IJssel harbors are located that accommodate the navigation locks. At Kornwerderzand two navigation locks are situated in this harbor. On the dikes also fifteen houses are located and a Kazematten museum. At Den Oever one navigation lock is situated in the harbor without any other objects.

All the navigation locks have two retaining doors at both ends to retain water at high and low tides. Also on the Northern side storm surge gates are installed that are used in case of a storm surge. A top view of a navigation lock is given in Figure C.2. In this Figure the following items are depicted:

1. Storm surge gates
2. High tide gates
3. Low tide gates
4. Upper lock head
5. Locking chamber
6. Lower lock head

In Figure C.3 an overview of the sluice complex at Den Oever is shown. The red line in this Figure is the line of the primary flood defense. The three outlet sluice groups are located on the left of this picture. The bottom left part in this Figure is the Wadden Sea and at the top left Lake IJssel is shown with the navigation lock in the harbor.

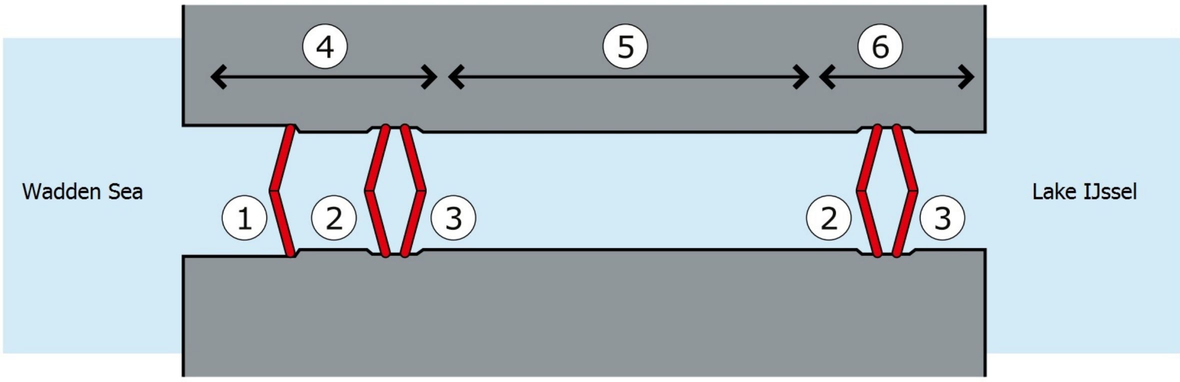

Figure C.2: Top view schematization of a navigation lock.

The following items are depicted in this Figure:

1. Inner port
2. Dikes along the port
3. Upper head navigation lock
4. Lower head navigation lock
5. Locking chamber
6. Outer port
7. Dams along outer port

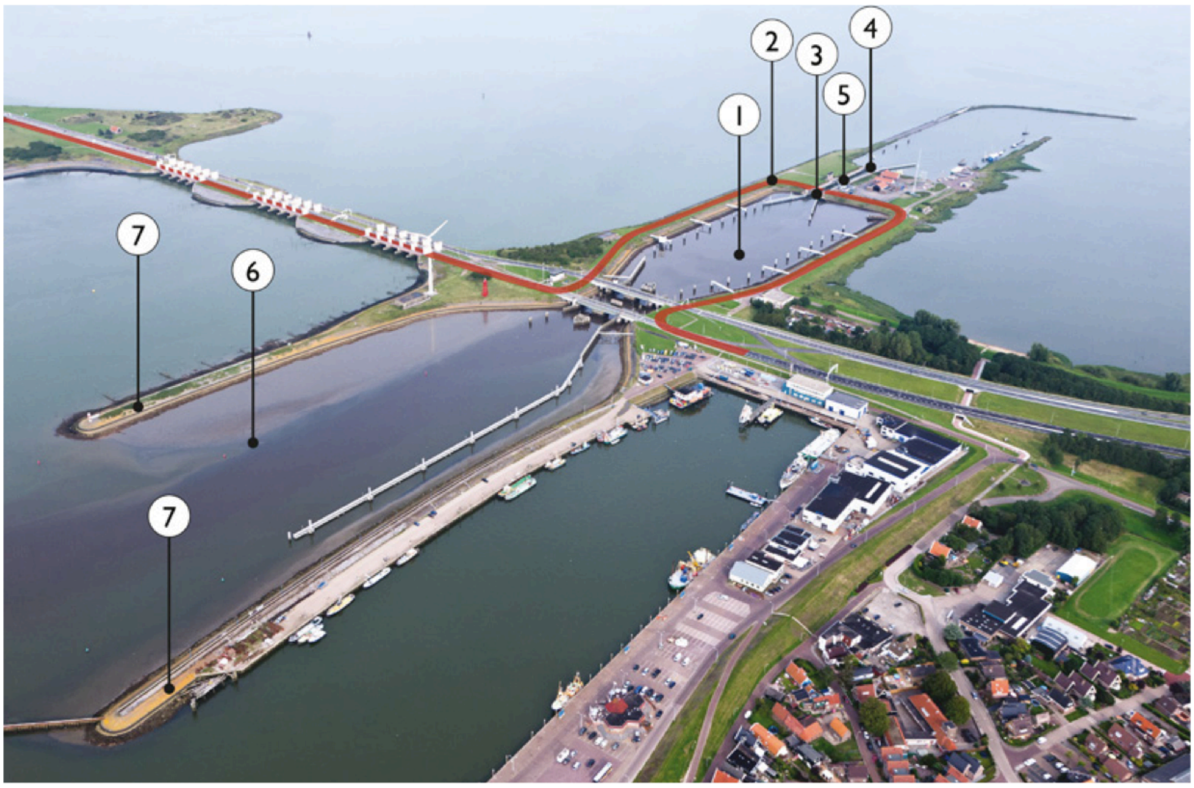

Figure C.3: Aerial view of navigation lock at Den Oever.

In Figure C.3 an overview of the sluice complex at Kornwerderzand is shown. The two outlet sluice groups are located on the right of this picture. On the left in this Figure the Wadden Sea is located and Lake IJssel on the right.

The following items are depicted in this Figure:

1. Inner port
2. Dikes along the port
3. Upper head navigation locks
4. Lower head navigation locks
5. Locking chambers
6. Houses
7. Kazematten museum
8. Outer port
9. Dams along outer port

Figure C.4: Aerial view of navigation lock at Kornwerderzand.

Assessment of the safety of the sluice complexes

In the assessments in 2006 and 2011 it was concluded that the outlet sluices are not strong enough to withstand the wave loads that occur with a probability of 1/10 000 per year. Also the reliability of closure of the gates during a critical storm condition, due to human error or technical failure, is too low because there is only one retaining door left. The foundation on piles at Kornwerderzand is not stable during a storm with a probability of 1/10 000 per year.

The upper heads of the three navigation locks are not stable during hydraulic conditions with a probability of 1/10 000 per year. Besides the instability during these conditions the retaining height of the high and low tide gates is insufficient. This is also the case for the dikes along the inner port. Another issue is that the storm surge gates in the navigation locks are not strong enough to withstand a storm with a probability of 1/10 000 per year.

In the current plans the sluice complexes will be improved to withstand storm conditions with a probability of occurrence of 1/10 000 per year until 2050. After this the sluice complexes have reached the end of their lifetime and new complexes have to be built.

In this MSc research the contribution of the failure of the sluice complexes on the total probability of failure of the Afsluitdijk is not taken into account. The aim of this MSc research is to optimize a design for the dike body taking the new safety standard into account. With the probability of failure budgets described in 3.1 the contribution of the dike body and the the hydraulic structures is taken into account. It is assumed that the probability of failure of the current configuration is too high compared to the current and the new safety standards, even when the hydraulic structures are not taken into account.

However it is recommended to investigate the contribution of the sluice complexes on the total probability of failure of the Afsluitdijk to see if improvements to the hydraulic structures are indeed needed. Also an optimization for the design of the hydraulic structures with the new safety standard in mind is recommended.

D

Design instruments for flood defences 2014 (OI2014)

In 2017 the safety assessments and design procedures for flood defences will be described legally by the WTI2017. In the Flood Protection Program (nHWBP) the WTI2017 must be taken into account to guarantee that designs are future proof. For this transition period from the current safety standard to WTI2017 the OI2014 is developed, based on results from VNK2. The OI2014 is developed in such a way that it follows the current design instruments as much as possible.

From a given standard (probability of flooding) for each dike trajectory a probability of failure for each cross section and each failure mechanism is determined. With this probability of failure the hydraulic loads and the safety factors are derived which are both part of the OI2014. With these hydraulic loads and the safety factors a flood defense can be designed with the current guides and technical reports.

One of the core ideas in the OI2014 is that the design instrument is practical, useful and applicable for all the flood defences. It aims to prevent flood defences being disapproved during the first safety assessment with the new safety standard. In practice this means that designs made according to OI2014 are conservative, but the chances of disapproval during the first safety assessment are small.

The development of the OI2014 is done by a group of experts in the fields of probability and physics. During this process the experts were supported by RWS, Deltares and the project team VNK2. This method will also be followed during the optimization of the reference alternative in this MSc research, which is described in chapter 5.

Framework

Before the development of OI2014 the following requirements were defined:

- The reference period[1] that has to be used is one year which is the same in the current design rules. An exception is the guideline Hydraulic Structures, where the reference period equals the intended lifetime.
- The design instrument is based on a semi-probabilistic approach which means that designs are made based on characteristic values and safety factors.
- Every semi-probabilistic design rule must depend on the standard and the length of the trajectory (because of the length effect).
- For every flood defense category it must be able to make a design. The subdivision in categories is not relevant for the water retaining functioning of a flood defense. This requirement is mainly to provide the hydraulic load conditions for all the primary flood defences.

[1]The reference period is the period in which the probabilities of failure are expressed, which is usually one year in hydraulic engineering.

- For ever type of flood defense it must be able to make a design taking every relevant failure mechanism into account.
- Adjustments to safety factors, resistance models and design loads are only implemented in the OI2014 if there are good reasons. In the case of lack of new data the current safety factors, resistance models and design loads are used.

In the development of the OI2014 focus was on the following starting points:

- The OI2014 only focusses on the water retaining function of the flood defences. For design aspects regarding other functions current guidelines and technical reports can be used.
- The design requirement that has to be met is defined as a probability of failure on cross-section levels. This probability of failure is derived from the standard (probability of flooding) of the dike trajectory.
- The design of a flood defense has to meet the most strict requirement that follows from the Dutch Building Decree/Eurocode or the proposed (or determined) standard (probability of flooding). For the development of the design instruments model that describe the failure mechanisms of flood defences are used. Guidelines and technical reports describe these models.

The starting points in WTI2017 are also the basis for OI2014. In WTI2017 both the safety assessment guidelines and the design instruments are described. Only the design instruments are part of the OI2014 and to prevent confusions the difference between the safety assessment guidelines and the design instruments is described. The following differences are relevant for determining the design instruments based on assessment standards:

- In the design more functions besides the water retaining function are of importance.
- In the design the strength of flood defences can be chosen between certain boundaries, in safety assessments this strength has a prefixed value.
- In the determination of the design rules most of the times it is efficient to choose a conservative (extra strength) design because of uncertainties. In the design of flood defences the fixed costs are known and only the variable costs can be changed. Given this it is more efficient to strive for a smaller probability of false positives[2] compared to safety assessments, which results in a larger probability of false negatives. In this provisional design rules a lot depends on engineering judgement which results in uncertainty if the design rules re too (un)safe.
- An economic optimal design standard is stricter (lower probability of failure) than an economic optimal assessment standard. The determination of the design standard is outside the scope of OI2014.
- Besides the safety standards for flood defences requirements are described for the structural assessment of hydraulic structures. These requirement are part of the Dutch Building Decree/Eurocode and are less strict than the design rules for new structures. Interference between the design rules and the structural assessment rules may occur, depending on the difference between them.
- In design and assessment rules different reference periods can be used. If larger periods are chosen the correlation between separate years result into lower reliability requirements. In the WTI2017 and the current assessment methods the reference period is set at one year. Because the design rules must be at least as struct as the assessment rules the reference period in OI2014 is also set at one year.
- In the design uncertainties are smaller than in assessments. Soil sampling and the choice of properties of the construction reduce uncertainties. In the assessment of existing (and old) structures uncertainties are bigger because less soil information is available and it is hard to assess the strength properties of old structures.
- In the design the expected changes and the strength during the expected life time are of importance. In safety assessments only the current strength is assessed.

In probabilistic design the flood defences are designed in a way that the probability of the load/-solicitation (S) being larger than the strength/resistance (R) is smaller than a certain probability of

[2]False negatives are designs that are disapproved during an assessment but that are actually all right. False positives are designs that are approved during an assessment but that actually are below standard.

failure(P_T):

$$P(R < S) < P_T \tag{D.1}$$

A probability of failure can be determined for every failure mechanism for each cross-section. These probabilities of failure can be combined to determine the probability of failure on a larger scale and/or for all the failure mechanisms together. A probabilistic calculation can be done with the requirements on every desired level.

In semi-probabilistic design the flood defences are designed in a way that a design value of the load/-solicitation (S_d) is smaller than the design value of the strength/resistance (R_d):

$$S_d < R_d \tag{D.2}$$

These design values are a combination of a characteristic value and a safety factor. A characteristic value is formulated as a value with a certain probability of undershooting (strength) or probability of exceedance (load). Typical values that are used are the 5% and 95% fractals of a stochastic variable. In hydraulic engineering the characteristic load values are relatively small with probabilities ranging from 1/250 to 1/10 000 per year. Because these characteristic values are already fixed the development of a semi-probabilistic method only concerns the derivation of safety factors.

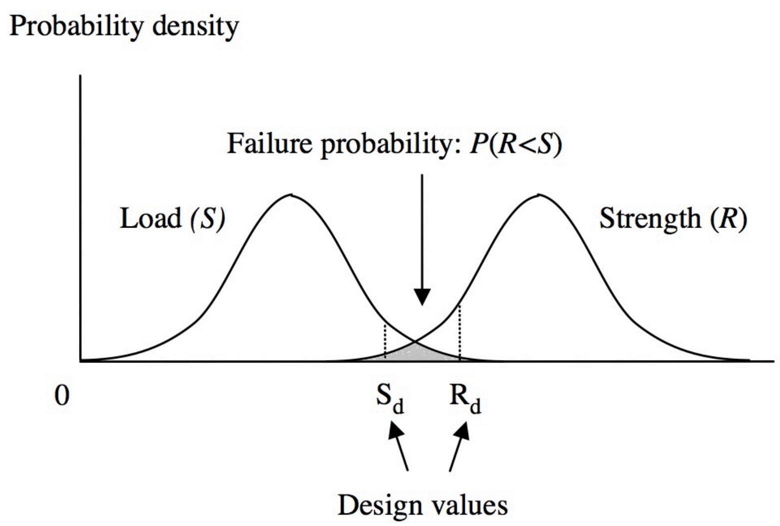

Figure D.1: The probability density functions of the load (S) and strength (R) and the design values of the load (S_d) and strength (R_d).

In Figure D.1 the probability density functions of load (S) and strength (R) and the design values of load (S_d) and strength (R_d) are given. The design values must be chosen in such a way that when the semi-probabilistic design standard is met the probability of failure is also met:

$$P(R < S) < P_T \text{ when } S_d < R_d \tag{D.3}$$

The semi-probabilistic assessments are made for each failure mechanism and cross-section. Therefore the probabilities of failure for every cross-section and failure mechanism must be known to derive the design values. The current safety standards have the appearance of a semi-probabilistic method, but the link with probabilities of failure is in many cases not clear. This is the reason why a lot of calibration studies are done for WTI2017.

In Figure D.1 t the gray area represents the failure area in a probabilistic approach. Both methods use the same limit state function (failure mechanism) and the same uncertain parameters (load and resistance stochastics). The essential difference is how is dealt with these uncertainties in the assessment of the flood defences.

To develop the semi-probabilistic method for OI2014 four steps are followed:

1. From probability of exceedance to probability of failure.

2. Determination of safety format for the semi-probabilistic design standard.
3. Determination of values for safety factors.
4. Place the results into perspective.

Step 1: From probability of exceedance to probability of failure. In Figure D.2 it is shown how the failure requirements can be derived from the safety standard (probability of flooding). For the OI2014 the design standard is a given. With the results of VNK2 probability budgets are assigned to every failure mechanism and these budgets are used in WTI2017. This probability budget is also used in OI2014 to derive the failure requirements for every mechanism on cross-section level. The result are shown in Table D.1.

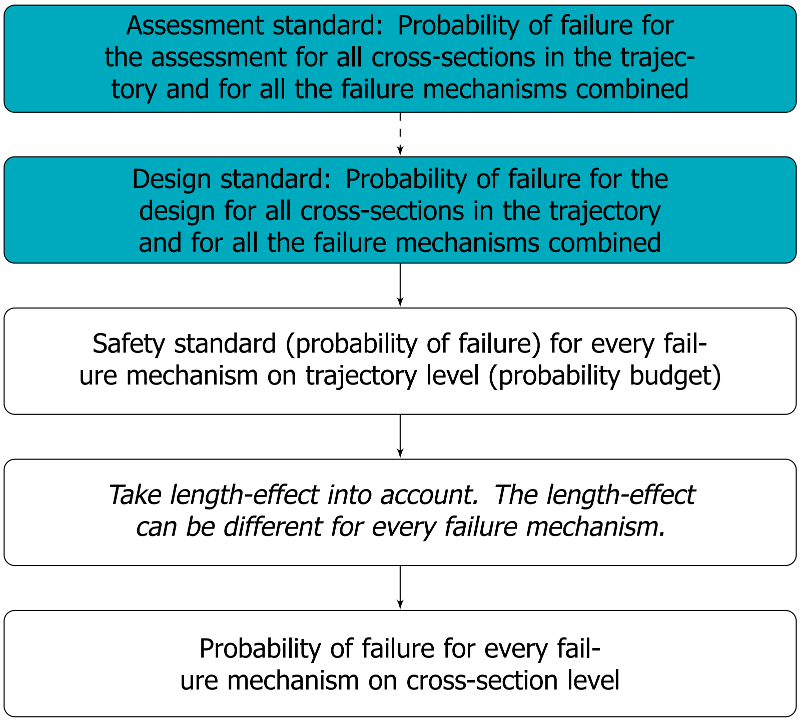

Figure D.2: Relation between the safety standard (probability of flooding) and the requirements (probability of failure) for every failure mechanism on cross-section level.

Table D.1: Probability of failure budget (maximum allowable probabilities of failure as percentages of the safety standard).

Type of flood defence	**Failure mechanism**	**Type of trajectory**	
		Sandy coast	**Remainder (dikes)**
Dike	Overflow and overtopping	0%	24%
	Piping	0%	24%
	Macro-stability inner slope	0%	4%
	Damage and erosion	0%	10%
Hydraulic structure	Non-closure	0%	4%
	Piping	0%	2%
	Constructive failure	0%	2%
Dune	Dune erosion	70%	0% / 10% [a]
Remainder		30%	30% / 20%
Total		**100%**	**100%**

[a]For trajectories that partly consist of dunes, the dune erosion contributes a relatively small percentage of failure. Proposed is to shift 10% from the remainder to dunes. This prevents that in these situations a completely new probability budget has to be used.

If the probability of failure budget is badly chosen this may result in uneconomical design, but not to trajectories that do meet the design rules but not to the design requirements on trajectory level (for all the failure mechanisms together). The use of this failure probability budget leads to a conservative design and assessment compared to a free failure probability distribution, regardless of the values chosen in Table D.1.

In Table D.1 the b-type of flood defences, including the Afsluitdijk, are missing. For the moveable barriers this is not necessary because these defences are assessed and designed in a probabilistic way. For the fixed flood defences, like the Afsluitdijk, it is not stated explicitly that the failure probability budget can be used. It is assumed that the budget differs significantly for the different b-type flood defences. However in OI2014 it is proposed to use the budget of the remainder (dikes). This is also the case for c-type flood defences. This is proposed with the idea of the relative insensitivity and the fact that information for better estimates lacks.

In this MSc research it is necessary to develop a semi-probabilistic approach for the design and safety assessment of the Afsluitdijk. Fully probabilistic methods are preferable but to take the correlation and length-effect into account a semi-probabilistic method with failure mechanism budgets is needed. Therefore the probability failure budgets that are chosen in OI2014 can be compared with the budgets and the corresponding partial safety factors that are derived in this MSc research.

Step 2: Determination of safety format for the semi-probabilistic design standard. The safety format for the semi-probabilistic design standard consists of:

1. The definition of the characteristic value of the stochastic variable.
2. The determination of the safety factors.

The characteristic values that are currently used in the guidelines and technical reports are used in OI2014. However there are some exceptions:

- When the characteristic values were not clearly defined an own definition was chosen, following the definitions derived in WTI2017.
- The design water level is chosen in such a way that the probability of exceedance of water level equals the probability of failure standard (i.e. the probability of failure requirement for all mechanisms in a whole trajectory). Attention has to be paid to:
  - In the current safety standard the probability of exceedance has a different importance than with a probability of flooding approach.
  - Characteristic values and partial safety factors are communicating barrels. the effect of low characteristic values can be compensated with higher values of safety factors.
  - By following the definition of the design load the characteristic value of the outer water level increases according to the standard level. If this would not be done, an inconsistent situation would occur where trajectories get different standards but have to use the same levels of exceedance.
  - Holding on to the current probabilities of exceedance from the Water Act is not an option. This would lead to chaotic design instruments with the new safety standard.

By determining a partial safety factor for every stochastic, the semi-probabilistic approach can follow the results of the failure calculations as close as possible. In some cases this is not the most practical solution. The current safety standard uses safety factors that are used as a bulk-parameter on strength terms. The use of a restricted amount of safety factors makes the semi-probabilistic assessment easier and reduces the chance of errors. This reduction of safety factors can result in conservatism in the semi-probabilistic assessment.

In general the following types of safety factors are used in the current safety standard:

- Model factors.
- Material factors.
- Schematization factors.

- Failure requirement safety factor[3](β-dependent safety factor).

In the development of the OI2014 the following line is applied:

1. In every design rule at least one β-dependent safety factor will be used. The design rule for wave overtopping is an exception on this rule. In the case of wave overtopping only a calculation value for the critical discharge will be prescribed.
2. De current types of safety factors remain the same. If a factor could be left out it is given a value of 1,0. By not removing the factor mistakes and/or errors are prevented.
3. Safety factors will be added if it is shown in WTI2017 that it is wise to reduce the conservatism in the semi-probabilistic approach.

Step 3: Determination of values for safety factors. For the determination of the safety factors the following data is needed:

1. A reliability requirement.
2. A limit state function.
3. Probability distributions of the random variables.
4. Characteristic values of the random variables.
5. Representative influence coefficients.

As the parts 1 until 4 are already given or chosen, the determination of the safety factors consists in practice only of the fixation of the representative influence coefficients. On a main basis there are two possibilities to do this. The first is to use the standard coefficients from the Eurocode. The second possibility is to execute a lot of probabilistic analyses.

It is noticed that the current safety factors in the standards are not completely based on a probabilistic method and are derived from different starting points. This is the reason why new safety factors are determined for WTI2017.

As standard influence coefficients have to be widely applicable, their use results mostly in conservative semi-probabilistic guidelines. For WTI2017 a more developed calibration method is used that is based on a large number of probabilistic analyses.

It would be ideal to use these results from WTI2017 in OI2014. During the development of OI2014 only preliminary studies where done. These studies were focussed on the feasibility and correctness of the calibration process and the early identification of bottlenecks during the execution of the semi-probabilistic assessments with the current failure mechanism models. It is not justified to use the results of the preliminary studies in OI2014 without any further investigation. The relative importance of different random variables is however one of the insights from these studies that is used in OI2014.

The values of the safety factors are defined as follows:

- Model factors: Fixed value, derived from one reliability index requirement.
- Material factors: Fixed value, derived from one reliability index requirement.
- Schematization factors: Follows from the schematization theory and depends on the schematization uncertainty.
- β-dependent factor: Depends on the water level norm and the length-effect.

In accordance with the current damage factor for macro-stability for inner slopes and WTI2017 the β-dependent factor will have the following formulation:

$$\gamma = x + y(\beta_{eis,dsn} - z) \tag{D.4}$$

with:

$$\beta_{eis,dsn} = -\Phi^{-1}(P_{eis,dsn}) \text{ with: } P_{eis,dsn} = \frac{P_{norm} \cdot \omega}{1 + \frac{a \cdot L_{ring}}{b}} \tag{D.5}$$

in which:

[3]This is an adjustment parameter that takes the effect of water level norm and the length-effect into account (damage factor).

x, y, z	[-]	Constants
$\beta_{eis,dsn}$	[per year]	Required reliability index on cross-section level
$P_{eis,dsn}$	[per year]	Required failure probability on cross-section level
ω	[-]	Failure probability budget factor for the corresponding failure mechanism
a	[-]	Factor for the length effect that takes two phenomenons into account: • The non substantial contribution of all dike sections on the failure probability on dike ring level • The correlation between the dike sections
b	[m]	Representative length for the analysis in a cross-section
L_{ring}	[m]	Total length of the dike ring (section)
P_{norm}	[per year]	Maximum allowable probability of flooding of the trajectory

The following steps are followed to determine the safety factors in OI2014:

1. Compare the results for every safety factor from WTI2017 with the current safety standards.
2. Determine if adjustments to the safety factors is necessary/desired.
3. Investigate if current research may lead to stricter requirements on the strength of flood defences. If extra strength may be required in the future this can be a reason not to adjust/lower safety factors.

Step 4: Place the results into perspective. If adjustments are made to safety factors it is investigated what the impact would be in practice. This is not a detailed research but it is investigated more in general on a larger scale.

E

Matlab scripts

Matlab script to determine hydraulic boundary conditions

```
function [c_SWL c_Hs c_T]=HR_weibull
Data_raw=xlsread('HR_matlab.xls');
R=Data_raw(:,1);
SWL=Data_raw(:,3);
Hs=Data_raw(:,4);
T=Data_raw(:,5);
P=1-1./R;
Fe = -log(P);

Data=[Fe SWL];
t=Data(:,1);
y=Data(:,2);
F=@(x,xdata)x(4).*(log(x(2)) + (x(1)./x(4)).^x(3) - log(xdata)).^(1./x(3));
x0=[1 1 1 1];
lb=[0.001 0.001 0.001 0.001];
[c_SWL]=lsqcurvefit(F,x0,t,y,lb);

Data=[SWL Hs];
t=Data(:,1);
y=Data(:,2);
F=@(x,xdata)x(1)*log(xdata)+x(2);
x0=[1 1];
[c_Hs]=lsqcurvefit(F,x0,t,y);
Hs=c_Hs(1)*log(SWL)+c_Hs(2);

Data=[Hs T];
t=Data(:,1);
y=Data(:,2);
F=@(x,xdata)sqrt(xdata/x(1));
x0=[1];
[c_T]=lsqcurvefit(F,x0,t,y);
end
```

Matlab script Z-function for overtopping

```
function z = overtopping_x2z(varargin)

% variables derived in external control script
% critical overtopping discharge
global qc

% constants for determining the hydraulic boundary conditions
global c_Hs
global c_T

```

```
% create samples-structure based on input arguments
samples = struct(...
      'beta',           [],...    % angle of wave impact
      'm_Hs',           [],...    % modelfactor for the wave height
      'm_Tm10',         [],...    % modelfactor for the wave period
      'SWL',            [],...    % still water level
      'upper_slope',    [],...    % upper slope
      'lower_slope',    [],...    % lower slope
      'g',              [],...    % gravitational acceleration
      'hB',             [],...    % berm height
      'B',              [],...    % berm width
      'hcrown',         [],...    % crown height
      'gamma_f',        [],...    % friction coefficient
      'C1',             [],...    % coefficient
      'C2',             [],...    % coefficient
      'C3',             [],...    % coefficient
      'C4',             [],...    % coefficient
      'C5',             [],...    % coefficient
      'm_qc',           [],...    % modelfactor critical overtopping discharge
      'm_q0',           []);...   % modelfactor overtopping discharge

samples = setproperty(samples, varargin{:});
% calculate z-values
% pre-allocate z
z = nan(size(samples.beta));
% loop through all samples and derive z-values
for i = 1:length(samples.beta)

% determine the wave height and period from the water level
H=samples.m_Hs(i)*(c_Hs(1)*log(samples.SWL(i))+c_Hs(2));
if H<0.5
     H=0.5;
else H=H;
end
T=samples.m_Tm10(i)*sqrt(H/c_T(1));

% determine basic variables
dh=samples.SWL(i)-samples.hB(i);
L0=samples.g(i)*T^2/(2*pi);
s0=(2*pi*H)/(samples.g(i)*T^2);
hk=samples.hcrown(i)-samples.SWL(i);
Lberm=H*samples.lower_slope(i)+H*samples.upper_slope(i)+samples.B(i);
rB=samples.B(i)/Lberm;

% determine factor for angle of wave impact
if (abs(samples.beta(i))<80)
     gamma_beta_ru=1-0.0022*abs(samples.beta(i));
else gamma_beta_ru=1-0.0033*abs(samples.beta(i));
end
if (abs(samples.beta(i))<80)
     gamma_beta_ov=1-0.0033*abs(samples.beta(i));
else gamma_beta_ov=1-0.0033*80;
end
if (samples.SWL(i)>1.5*H)
          lowerh=1.5*H-dh;
else lowerh=samples.hB(i);
end

% determine run-up and reduction factor with an iteration
z2(1)=1.5*H;
for j=1:100
     if ((samples.hcrown(i)-samples.SWL(i))>z2(j))
          upperh=z2(j)+dh;
     else upperh=samples.hcrown(i)-samples.hB(i);
     end
Ltalud=lowerh*samples.lower_slope(i)+upperh*samples.upper_slope(i);
htalud=lowerh+upperh;
alpha=atan(htalud/Ltalud);
xi0=tan(alpha)/sqrt(s0);
if (xi0<1.8)
     z2(j+1)=samples.C1(i)*xi0*H;
```

```
else z2(j+1)=(samples.C2(i)-samples.C3(i)/sqrt(xi0))*H;
end
if (z2(j+1)>-dh && -dh>0)
      x=z2(j+1);
      rdh=0.5-0.5*cos(pi*(dh/x));
elseif (2*H>dh && dh>0)
      x=2*H;
      rdh=0.5-0.5*cos(pi*(dh/x));
elseif (dh==0)
      rdh=0;
else rdh=1;
end
gamma_b=1-rB*(1-rdh);
if (gamma_b<0.6)
      gamma_b=0.6;
elseif (gamma_b>1)
      gamma_b=1;
else gamma_b=gamma_b;
end
Lf_u=(0.5*z2(j)/samples.gamma_f(i)+dh)*samples.upper_slope(i);
Lf_l=((0.25*z2(j)-dh)/samples.gamma_f(i))*samples.lower_slope(i);
gamma_f=(samples.gamma_f_u(i)*Lf_u+samples.gamma_f_B(i)*samples.B(i)+ ...
      samples.gamma_f_l(i)*Lf_l)/(Lf_u+samples.B(i)+Lf_l);
gamma_tot=gamma_b*gamma_f*gamma_beta_ru;
if (gamma_tot<0.4)
      gamma_tot=0.4;
else gamma_tot=gamma_tot;
end
if (xi0<1.8)
      z2(j+1)=samples.C1(i)*gamma_tot*xi0*H;
else z2(j+1)=(gamma_tot/gamma_b)*(samples.C2(i)-samples.C3(i)/sqrt(xi0))*H;
end
j=j+1;
if abs(z2(j)-z2(j-1))<0.00001, break, end
end

% determine wave overtopping
Qb=(0.067/sqrt(tan(alpha)))*gamma_b*xi0*exp(-samples.C4(i)*hk/ ...
      (H*xi0*gamma_b*gamma_f*gamma_beta_ov));
Qn=0.2*exp(-samples.C5(i)*hk/(H*gamma_f*gamma_beta_ov));
if (Qb<Qn)
      Q=Qb;
else Q=Qn;
end
q0=1000*Q*sqrt(samples.g(i)*H^3);

% limit state function (with qc(=R) loaded from external control script)
z(i,:) = abs(samples.m_qc(i))*qc-abs(samples.m_q0(i))*q0;

end
```

Matlab script Z-function for determination of critical overtopping discharge

```
function z = overtopping_c_x2z(varargin)

% variables derived in external control script
% constant for determining the hydraulic boundary conditions
global Pov;

% create samples-structure based on input arguments
samples = struct(...
      'f_g',            [],...     % erosion resistance of grass
      'd_w',            [],...     % height grass roots
      'c_RK',           [],...     % factor - erosion resistance inner clay cover
      'L_Kinner',       [],...     % length of inner clay cover
      'k',              [],...     % Strickler roughness coefficient
      'inner_slope',    [],...     % inner slope
      'Ts',             []);...

```

```
samples = setproperty(samples, varargin{:});
% calculate z-values
% pre-allocate z
z = nan(size(samples.f_g));
% loop through all samples and derive z-values
for i = 1:length(samples.f_g)

% determine variables
alpha_i=atan(1/samples.inner_slope(i));
c_g=6*10^5*samples.f_g(i)^1.5;
t_RTinner=(c_g*samples.d_w(i)*(samples.Ts(i)/3600))/(c_g*samples.d_w(i)+ ...
    (0.4*samples.c_RK(i)*samples.L_Kinner(i)));
t_e=Pov*t_RTinner;
v_c=samples.f_g(i)*3.8/(1+(0.8^10)*log10(t_e));

% formula critical wave overtopping
qc=1000*(v_c^(5/2)*samples.k(i)^(1/4))/(125*(tan(alpha_i)^(3/4)));

% store qc in the limit state function
z(i,:) = qc;

end
```

Matlab script Z-function for failure of stones and filter

```
function z = prob_stbk_x2z(varargin)

% variables derived in external control script
% constant for determining the hydraulic boundary conditions
global c_T

% create samples-structure based on input arguments
samples = struct(...
    'Hs',            [],...  % significant waveheight
    'gamma_wat',     [],...  % density water
    'gamma_stone',   [],...  % density stone
    'Df15',          [],...  % 15 percent quantile size filter
    'D',             [],...  % thickness stone revetment
    'ca',            [],...  % coefficient determination Lambda
    'cb',            [],...  % coefficient determination Lambda
    'ct',            [],...  % coefficient determination Lambda
    'cgf',           [],...  % factor for strength of stone revetment
    'df',            [],...  % thickness filter layer
    's',             [],...  % width of the splits betweeen the stones
    'slope',         []);... % degree slope

samples = setproperty(samples, varargin{:});
% calculate z-values
% pre-allocate z
z = nan(size(samples.Hs));
% loop through all samples and derive z-values
for i = 1:length(samples.Hs)

% deterministic variables
g=9.81;
nu=0.0000012;   %kinematic viscosity water
n=0.35;         %porosity of the filter
Gamma=1;        %factor for the friction between the stones and flow and ineratia.
r=1;            %factor reduction roughness grass

% determine wave period from the wave height
Tp=sqrt(samples.Hs(i)/c_T(1));

% determine basic variables
alpha=degtorad(samples.slope(i));
Delta=(samples.gamma_stone(i)-samples.gamma_wat(i))/samples.gamma_wat(i);
Sop=(2*pi*samples.Hs(i))/(g*Tp^2);
Xiop=tan(alpha)/sqrt(Sop);
af=samples.ca(i)*(160*nu*(1-n)^2)/(g*n^3*samples.Df15(i)^2);
```

```
bf=samples.cb(i)*(2.2)/(g*n^2*samples.Df15(i));
% B=stone block width
% L=stone block length
% l=(B*L)/(B*s+L*s);
% split area given according to VNK method s=0.12
l=1/0.12;
rmin=max(0.5*samples.Df15(i),0.4*samples.s(i));
a2=(12*nu*l)/(g*samples.s(i)^2)+(l*samples.s(i)*af)/(pi*samples.D(i))* ...
     log((l*samples.s(i))/(pi*exp(1)*rmin));
b2=(l^2)/(2*g*samples.D(i))*((1/n-1)^2+1)+(l*samples.s(i)*bf)/(pi*samples.D(i))* ...
     ((l*samples.s(i))/(pi*rmin)-2);

% determine factor strength of stone revetment
if 1/l ≥ 0.05
     cf=5.2;
else cf=8.15;
end

% determine leakage length
k1=(-af+sqrt((af^2+4*bf)))/(0.6*bf);
k2=samples.ct(i)*(-a2+sqrt((a2^2+4*b2)))/(2*b2);
Lambda=sqrt(k1*samples.df(i)*samples.D(i)/k2);

% limit state functions for failure of the top layer and the filter layer
Zb_filter=((cf*Delta*samples.D(i)^1.67*Gamma^1.67)/(Lambda*tan(alpha))^0.67)- ...
     ((r*samples.Hs(i))/(r*Sop)^0.33);
Zb_top=(samples.cgf(i)*((tan(alpha)/sqrt(Sop)))^-(2/3))- ...
     (samples.Hs(i)/(Delta*samples.D(i)));
z(i,:) = min(Zb_filter,Zb_top);

end
```

Matlab script Z-function for residual strength of the dike body

```
function z = prob_stbk_res_max_x2z(varargin)

% variables derived in external control script
% constant for determining the hydraulic boundary conditions
global c_T c_Hs

% create samples-structure based on input arguments

samples = struct(...
     'SWL',        [],...  % still water level
     'c',          [],...  % coefficient
     'crk',        [],...  % coefficient
     'crb',        [],...  % coefficient
     'Lk',         [],...  % thickness clay cover
     'Lb',         [],...  % thickness dike sand core at MSL
     'ts',         [],...  % storm duration
     'slope',      [],...  % outer slope
     'Bcrown',     [],...  % width crown
     'alphah',     [],...  % coefficient erosion speed
     'hcrown',     []);... % crown height

samples = setproperty(samples, varargin{:});
% calculate z-values
% pre-allocate z
z = nan(size(samples.SWL));
% loop through all samples and derive z-values
for i = 1:length(samples.SWL)

% determine the wave height and period from the water level
Hs=(c_Hs(1)*log(samples.SWL(i))+c_Hs(2));
if Hs<0.5
     Hs=0.5;
else Hs=Hs;
end;
Tp=sqrt(Hs/c_T(1));
```

```
% deterministic variables
g=9.81;
r=1;

% determine basic variables
Lop=(g/(2*pi))*Tp^2;
alpha=degtorad(samples.slope(i));
dk=sin(alpha)*samples.Lk(i);

Lb=(samples.hcrown(i)-samples.SWL(i)+0.25*Hs)/tan(alpha)-samples.Lk(i)+samples.Bcrown(i);
 if Lb < samples.Bcrown(i)
      Lb=samples.Bcrown(i);
 else Lb=Lb;
end

hz=((samples.alphah(i)*Lb*dk)/cos(alpha))/((0.5*tan(alpha)*(Lb^2-samples.Bcrown(i)^2)+ ...
      (((Lb-samples.Bcrown(i))/cos(alpha)+samples.Bcrown(i))*dk)));
 if hz < 1
      hz=1;
 else hz=hz;
end

rz=(0.5*tan(alpha)*(Lb^2-samples.Bcrown(i)^2))/((0.5*tan(alpha)*(Lb^2- ...
      samples.Bcrown(i)^2)+(((Lb-samples.Bcrown(i))/cos(alpha)+samples.Bcrown(i))*dk)));
 if rz < 0
      rz=0;
 else rz=rz;
end

alphaz=6;
vzb=(1+alphaz*rz)*hz;
crb=samples.crk(i)/vzb;

% determine residual strenght
trs=57*10^3*Tp*exp(-sqrt(Hs*Lop/samples.c(i)));
trk=(0.4*samples.Lk(i)*samples.crk(i))/(r^2*Hs^2);
trb=(0.4*Lb*crb)/(r^2*Hs^2);

% limit state function for the erosion of the dike body
z(i,:) = trs+trk+trb-samples.ts(i);

end
```

F

Cross sections of the Afsluitdijk

In this Appendix the cross sections of all the dike sections that are used in this MSc research are shown. These cross sections and the cross sections of the other dike sections as can be found in (Ministerie van VW, 2009).

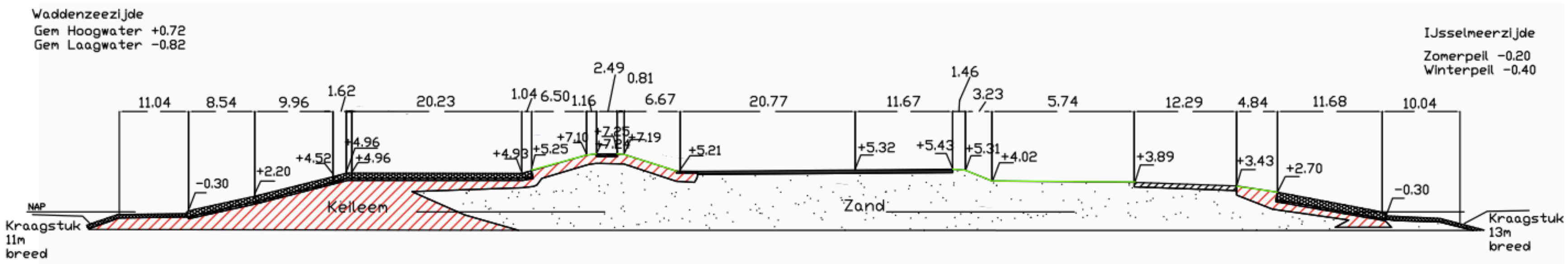

Figure F.1: Cross section of dike section 5.

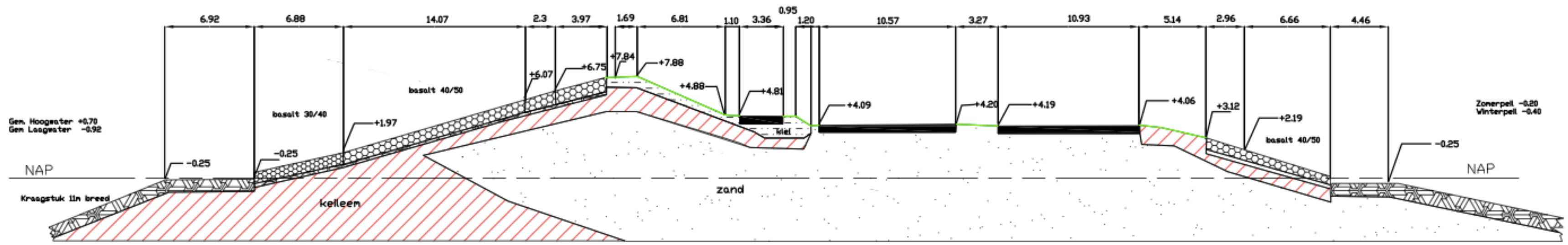

Figure F.2: Cross section of dike section 6.

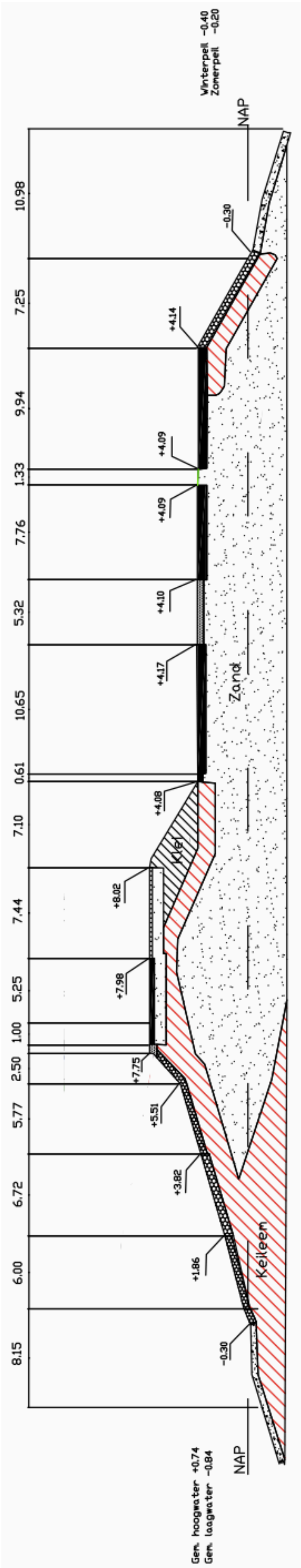

Figure F.3: Cross section of dike section 7.

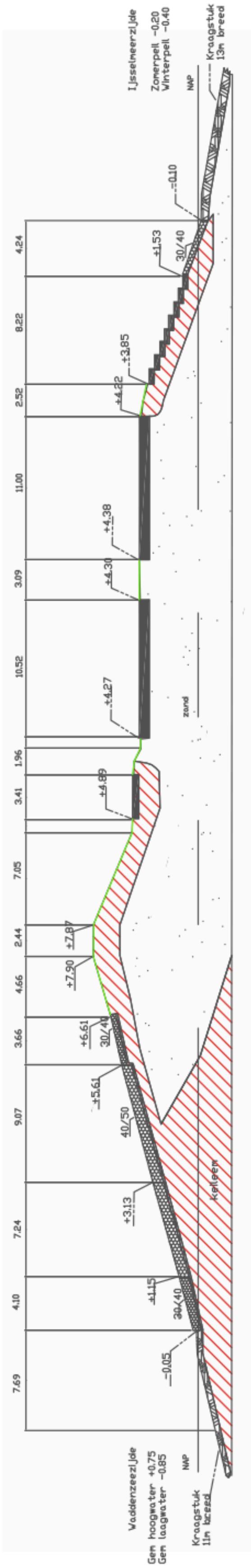

Figure F.4: Cross section of dike section 8.

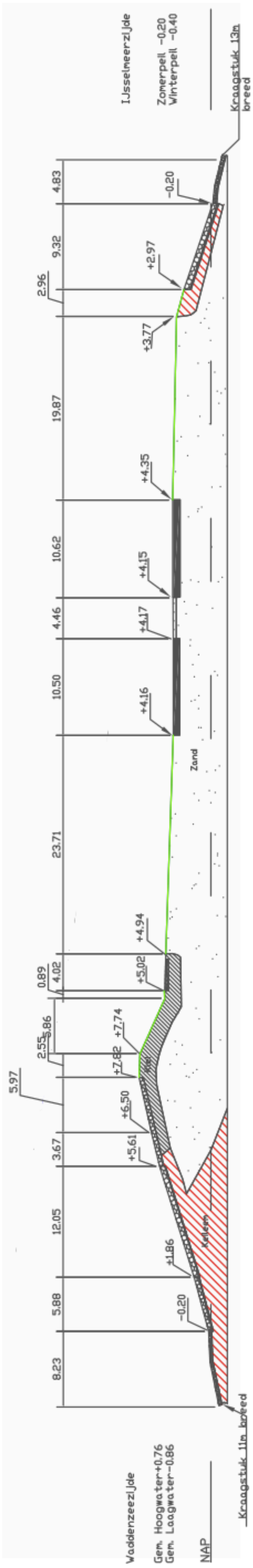

Figure F.5: Cross section of dike section 9.

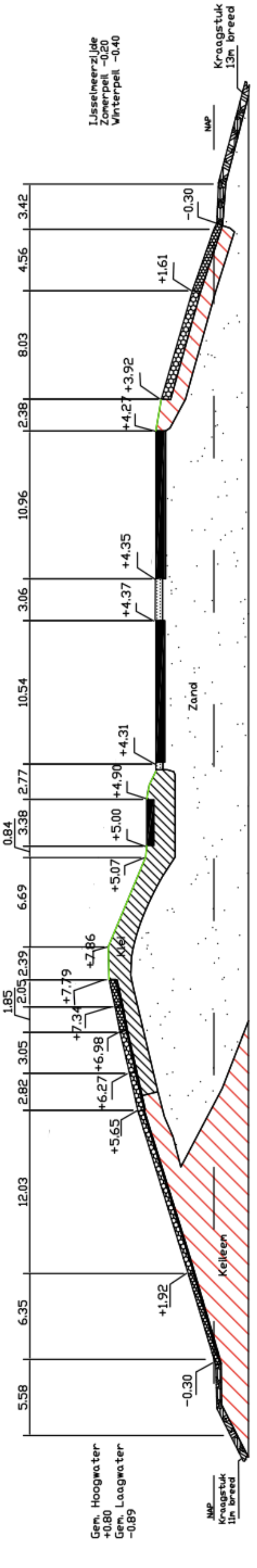

Figure F.6: Cross section of dike section 10.

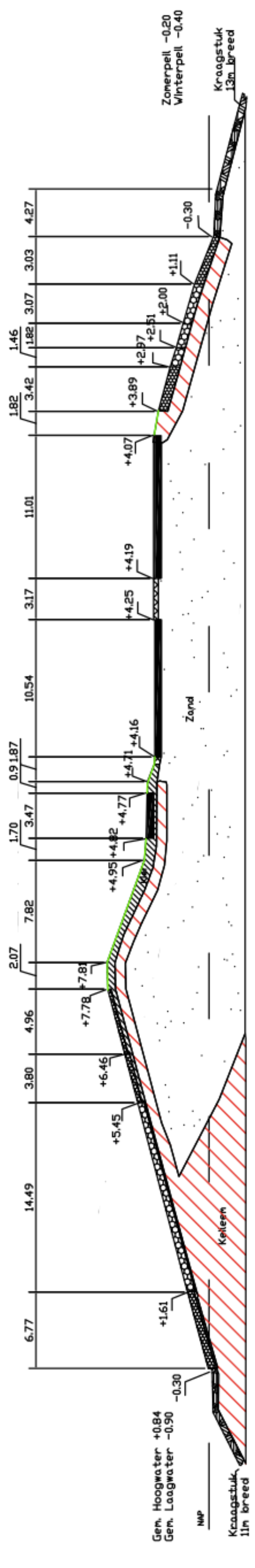

Figure F.7: Cross section of dike section 11.

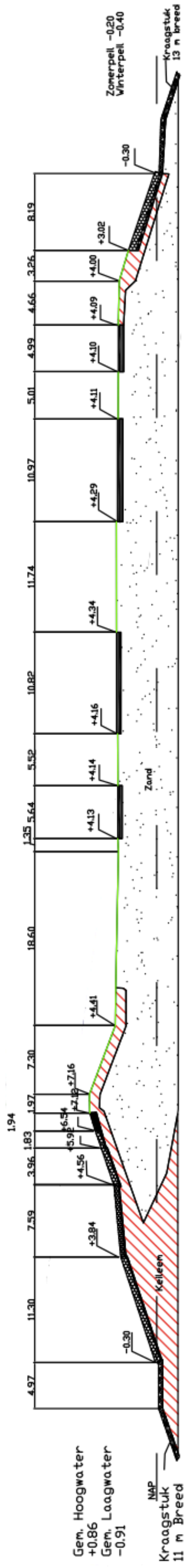

Figure F.8: Cross section of dike section 17.

G

Input for the instability of the stone revetments from Steentoets

In this Appendix the input that is used in PC-Ring is shown. The input is from an assessment in Steentoets, which is a spreadsheet that is developed for the assessment of flood defences in the Netherlands. For the dike sections 8A up to 10B the weakest revetment sections are chosen to asses the stability with PC-Ring. A full description of Steentoets and the use of it can be found in (Klein Breteler, 2012).

STEENTOETS versie 4.03, WL / Delft Hydraulics, dec. 2004				niveau	niveau	type		helling	TOPLAAG						BOVENSTE FILTERLAAG		
Volg-nr.	Naam van dijkvak 8A	Subvakgrenzen		onder-grens	boven-grens	toplaag	onderlagen (filter, geotex-tiel, klei, etc)	te toetsen talud/berm	D	B	L	spleet	open oppervlak	soortelijke massa	b b(min): 3 cm	D15	poro-siteit
		van	tot	[m NAP]	[m NAP]			tanα	[m]	[m]	[m]	[mm]	[%]	[kg/m3]	[m]	[mm]	[-]
W83	b7.6-1a	76.61	79.50	2.000	6.000	26.00	pu vl kl	0.283	0.450				12.0		0.200	30.0	
W84	b7.6-1b	79.50	87.07	2.000	6.000	26.00	pu vl kl	0.283	0.450				12.0		0.200	30.0	
W85	b7.6-2a	76.61	79.50	6.000	6.600	28.01	kl	0.218	0.350	0.500	0.500	5.0		2200			
W86	b7.6-2b	79.50	87.07	6.000	6.600	28.01	kl	0.218	0.350	0.500	0.500	5.0		2200			
W87	b7.6-3a	76.61	79.50	6.600	7.200	28.01	kl	0.218	0.300	0.500	0.500	5.0		2200			
W88	b7.6-3b	79.50	87.07	6.600	7.200	28.01	kl	0.218	0.300	0.500	0.500	5.0		2200			
W89	b7.6-4a	76.61	79.50	7.200	7.750	26.01	kl	0.172	0.250				12.0				
W90	b7.6-4b	79.50	87.07	7.200	7.750	26.01	kl	0.172	0.250				12.0				
W91	b8.7-1a	87.07	96.50	-0.150	1.500	26.00	pu vl kl	0.347	0.350				12.0		0.150	30.0	
W92	b8.7-1b	96.50	120.00	-0.150	1.500	26.00	pu vl kl	0.347	0.350				12.0		0.150	30.0	
W94	b8.7-2a	87.07	96.50	1.500	6.000	26.00	pu vl kl	0.266	0.450				12.0		0.400	30.0	
W95	b8.7-2b	96.50	98.00	1.500	6.000	26.00	pu vl kl	0.266	0.450				12.0		0.400	30.0	
W96	b8.7-3	87.07	90.50	6.000	7.750	28.01	kl	0.202	0.350	0.500	0.500	5.0		2200			
W97	b9.0-1a	90.50	96.50	6.000	6.800	26.00	pu vl kl	0.213	0.350				12.0		0.200	30.0	
W98	b9.0-1b	96.50	98.00	6.000	6.800	26.00	pu vl kl	0.213	0.350				12.0		0.200	30.0	
W99	b9.0-2a	90.50	96.50	6.800	7.750	26.00	pu vl kl	0.194	0.250				12.0		0.200	30.0	
W100	b9.0-2b	96.50	98.00	6.800	7.750	26.00	pu vl kl	0.194	0.250				12.0		0.200	30.0	
W101	b9.8-1	98.00	106.00	1.500	3.600	26.00	pu vl kl	0.304	0.450				12.0		0.200	30.0	
W102	b9.8-2	98.00	106.00	3.600	6.000	27.10	st pu vl kl	0.240	0.400				12.0	2900	0.150	20.0	0.35
W103	b9.8-3	98.00	106.00	6.000	6.800	27.10	st ge kl	0.211	0.300				12.0	2600	0.100	20.0	0.35
W104	b9.8-4	98.00	106.00	6.800	7.750	27.10	st ge kl	0.196	0.250				12.0		0.100	20.0	0.35
W105	b10.6-1	106.00	110.00	1.500	6.000	26.00	pu vl kl	0.271	0.450				12.0		0.200	30.0	
W106	b10.6-2	106.00	110.00	6.000	6.750	26.00	pu vl kl	0.197	0.350				12.0		0.200	30.0	
W107	b10.6-3	106.00	110.00	6.750	7.750	26.00	pu vl kl	0.206	0.250				12.0		0.200	30.0	

Figure G.1: Input for the instability of the stone revetment parts for dike section 8A.

STEENTOETS versie 4.03, WL / Delft Hydraulics, dec. 2004				niveau	niveau	type		helling	TOPLAAG						BOVENSTE FILTERLAAG		
Volg-nr.	Naam van dijkvak 8B	Subvakgrenzen		onder-grens	boven-grens	toplaag	onderlagen (filter, geotex-tiel, klei, etc)	te toetsen talud/berm	D	B	L	spleet	open oppervlak	soortelijke massa	b b(min): 3 cm	D15	poro-siteit
		van	tot	[m NAP]	[m NAP]			tanα	[m]	[m]	[m]	[mm]	[%]	[kg/m3]	[m]	[mm]	[-]
W92	b8.7-1b	96.50	120.00	-0.150	1.500	26.00	pu vl kl	0.347	0.350				12.0		0.150	30.0	
W93	b8.7-1c	120.00	137.00	-0.150	1.500	26.00	pu vl kl	0.347	0.350				12.0		0.150	30.0	
W108	b11.0-1a	110.00	120.00	1.500	3.600	26.00	pu vl kl	0.280	0.450				12.0		0.200	30.0	
W109	b11.0-1b	120.00	137.00	1.500	3.600	26.00	pu vl kl	0.280	0.450				12.0		0.200	30.0	
W110	b11.0-2a	110.00	120.00	3.600	6.000	27.10	st pu vl kl	0.255	0.400				12.0	2900	0.150	20.0	0.35
W111	b11.0-2b	120.00	151.60	3.600	6.000	27.10	sl pu vl kl	0.255	0.400				12.0	2900	0.150	20.0	0.35
W112	b11.0-3a	110.00	120.00	6.000	6.750	27.10	gr ge kl	0.197	0.350				12.0		0.200	20.0	0.35
W113	b11.0-3b	120.00	151.60	6.000	6.750	27.10	gr ge kl	0.197	0.350				12.0		0.100	20.0	0.35
W114	b11.0-4a	110.00	120.00	6.750	7.750	27.10	gr kl	0.206	0.250				12.0		0.100	20.0	0.35
W115	b11.0-4b	120.00	151.60	6.750	7.750	27.10	gr kl	0.206	0.250				12.0		0.100	20.0	0.35
W116	b13.7-1	137.00	139.00	-0.400	1.500	27.10	pu vl kl	0.253	0.400				12.0		0.200	30.0	
W117	b13.7-2	137.00	139.00	1.500	3.600	27.10	pu vl kl	0.280	0.400				12.0		0.200	30.0	
W118	b13.9-1	139.00	151.60	1.500	3.600	26.00	pu vl kl	0.280	0.450				12.0		0.350	30.0	
W119	b13.9-2	139.00	151.60	-0.350	1.500	26.00	pu vl kl	0.349	0.350				12.0		0.300	30.0	
W120	b14.5-2	145.50	151.60	6.000	6.750	27.10	pu vl kl	0.197	0.350				12.0		0.200	30.0	
W121	b14.5-3	145.50	151.60	6.750	7.750	27.10	sl kl	0.213	0.250				12.0		0.300	20.0	0.35

Figure G.2: Input for the instability of the stone revetment parts for dike section 8B.

STEENTOETS versie 4.03, WL / Delft Hydraulics, dec. 2004				niveau	niveau	type		helling	TOPLAAG						BOVENSTE FILTERLAAG		
Volg-nr.	Naam van dijkvak 9	Subvakgrenzen		onder-grens	boven-grens	toplaag	onderlagen (filter, geotex-tiel, klei, etc)	te toetsen talud/berm	D	B	L	spleet	open oppervlak	soortelijke massa	b b(min): 3 cm	D15	poro-siteit
		van	tot	[m NAP]	[m NAP]			tanα	[m]	[m]	[m]	[mm]	[%]	[kg/m3]	[m]	[mm]	[-]
W122	b15.1-1	151.60	153.60	-0.100	2.000	26.00	pu vl kl	0.300	0.350				12.0		0.200	30.0	
W123	b15.1-2	151.60	153.60	2.000	5.600	26.00	pu vl kl	0.327	0.450				12.0		0.200	30.0	
W124	b15.1-3	151.60	153.60	5.600	6.900	26.00	pu vl kl	0.217	0.350				12.0		0.200	30.0	
W125	b15.1-4	151.60	153.60	6.900	7.750	26.00	pu vl kl	0.170	0.250				12.0		0.200	30.0	
W126	b15.3-1	153.60	155.10	-0.300	0.700	26.00	pu vl kl	0.333	0.350				12.0		0.200	30.0	
W127	b15.3-2	153.60	155.10	0.700	6.000	27.10	pu vl kl	0.265	0.400				12.0	2900	0.200	30.0	
W128	b15.3-3	153.60	155.10	6.000	6.900	27.10	sl kl	0.225	0.350				12.0		0.300	20.0	0.35
W129	b15.3-4	153.60	155.10	6.900	7.750	27.10	sl kl	0.170	0.250				12.0		0.300	20.0	0.35
W130	b15.5-1a	155.15	156.00	-0.300	2.000	26.00	pu vl kl	0.329	0.350				12.0		0.200	30.0	
W131	b15.5-1b	156.00	157.20	-0.300	2.000	26.00	pu vl kl	0.329	0.350				12.0		0.200	30.0	
W132	b15.5-2a	155.15	156.00	2.000	4.000	26.00	pu vl kl	0.308	0.450				12.0		0.200	30.0	
W133	b15.5-2b	156.00	157.20	2.000	4.000	26.00	pu vl kl	0.308	0.450				12.0		0.200	30.0	
W134	b15.5-3a	155.15	156.00	4.000	5.600	27.10	sl pu vl kl	0.286	0.400				12.0	2900	0.100	20.0	0.35
W135	b15.5-3b	156.00	157.20	4.000	5.600	27.10	sl pu vl kl	0.286	0.400				12.0	2900	0.100	20.0	0.35
W136	b15.5-4a	155.15	156.00	5.600	6.600	27.10	sl kl	0.250	0.350				12.0		0.200	20.0	0.35
W137	b15.5-4b	156.00	157.20	5.600	6.600	27.10	sl kl	0.250	0.350				12.0		0.200	20.0	0.35
W138	b15.5-5a	155.15	156.00	6.600	7.700	27.10	sl kl	0.220	0.250				12.0		0.200	20.0	0.35
W139	b15.5-5b	156.00	157.20	6.600	7.700	27.10	sl kl	0.220	0.250				12.0		0.200	20.0	0.35
W140	b157-1	157.20	161.30	-0.300	1.400	26.00	pu vl kl	0.315	0.350				12.0		0.200	30.0	
W141	b157-2	157.20	161.30	1.400	3.600	26.02	pu vl kl	0.310	0.350				12.0		0.200	30.0	
W142	b157-4	157.20	161.30	1.600	3.600	28.02	pu vl kl	0.381	0.450	0.500	0.500	5.0		2200	0.200	30.0	
W143	b15.7-5	157.20	158.35	1.600	3.800	27.10	sl pu vl kl	0.286	0.350				12.0		0.100	20.0	0.35
W144	b15.7-6	157.20	159.00	3.800	6.000	27.10	sl pu vl kl	0.286	0.250				12.0		0.100	20.0	0.35
W145	b157-11	158.00	158.20	1.500	3.600	26.02	kl	0.183	0.450				12.0				
W146	b15.8-1	158.35	160.65	1.600	3.800	27.10	sl pu vl kl	0.286	0.350				12.0		0.100	20.0	0.35
W147	b15.9-1	159.00	161.00	3.800	10.000	27.10	sl pu vl kl	0.332	0.250				12.0		0.150	20.0	0.35
W148	b160-1	160.50	160.80	-0.400	3.500	28.00	pu vl kl	0.312	0.350	0.500	0.500	5.0		2200	0.200	30.0	
W149	b16.0-2	160.80	161.00	-0.400	2.000	28.00	pu vl kl	0.343	0.350	0.500	0.500	5.0		2200	0.200	30.0	
W150	b16.0-3	160.80	161.00	2.000	3.500	26.00	pu vl kl	0.188	0.450				12.0		0.200	30.0	
W151	b160-5	160.50	160.80	1.800	3.500	28.00	pu vl kl	0.391	0.250	0.400	0.400	5.0		2200	0.200	30.0	
W152	b160-7	158.35	161.30	-0.350	1.500	28.00	pu vl kl	0.394	0.250	0.400	0.400	5.0		2200	0.200	30.0	
W153	b160-9	160.50	160.50	-0.360	3.500	26.00	pu vl kl	0.297	0.350				12.0		0.200	30.0	
W154	b16.1-1a	161.00	200.00	-0.350	2.000	26.00	pu vl kl	0.336	0.350				12.0		0.200	30.0	
W157	b16.1-2a	161.00	200.00	2.000	5.550	26.00	sl vl kl	0.282	0.450				12.0		0.120	20.0	0.35
W159	b16.1-3	161.00	171.00	5.550	6.500	28.01	kl	0.241	0.450	0.500	0.500	5.0		2200			
W160	b16.1-4	161.00	171.00	6.500	7.750	28.01	kl	0.223	0.350	0.500	0.500	5.0		2200			
W161	b17.1-1	171.00	179.20	5.670	6.100	27.10	sl vl kl	0.226	0.400				12.0	2900	0.200	20.0	0.35
W162	b17.1-2a	171.00	179.20	6.100	6.500	27.10	sl vl kl	0.200	0.350				12.0		0.150	20.0	0.35
W163	b17.1-2b	171.00	179.20	6.500	6.900	27.10	sl ge kl	0.200	0.350				12.0		0.150	20.0	0.35
W164	b17.1-3	171.00	179.20	6.900	7.750	27.10	sl ge kl	0.213	0.250				12.0		0.200	20.0	0.35

Figure G.3: Input for the instability of the stone revetment parts for dike section 9.

STEENTOETS versie 4.03, WL / Delft Hydraulics, dec. 2004				niveau	niveau	type		helling	TOPLAAG						BOVENSTE FILTERLAAG		
Volg-nr.	Naam van dijkvak 10A	Subvakgrenzen		onder-grens [m NAP]	boven-grens [m NAP]	toplaag	onderlagen (filter, geotex-tiel, klei, etc)	te toetsen talud/berm $\tan\alpha$	D [m]	B [m]	L [m]	spleet [mm]	open oppervlak [%]	soortelijke massa [kg/m3]	b b(min): 3 cm [m]	D15 [mm]	poro-siteit [-]
		van	tot														
W154	b16.1-1a	161.00	200.00	-0.350	2.000	26.00	pu vl kl	0.336	0.350				12.0		0.200	30.0	
W157	b16.1-2a	161.00	200.00	2.000	5.550	26.00	sl vl kl	0.282	0.450				12.0		0.120	20.0	0.35
W159	b16.1-3	161.00	171.00	5.550	6.500	28.01	kl	0.241	0.450	0.500	0.500	5.0		2200			
W160	b16.1-4	161.00	171.00	6.500	7.750	28.01	kl	0.223	0.350	0.500	0.500	5.0		2200			
W161	b17.1-1	171.00	179.20	5.670	6.100	27.10	sl vl kl	0.226	0.400				12.0	2900	0.200	20.0	0.35
W162	b17.1-2a	171.00	179.20	6.100	6.500	27.10	sl vl kl	0.200	0.350				12.0		0.150	20.0	0.35
W163	b17.1-2b	171.00	179.20	6.500	6.900	27.10	sl ge kl	0.200	0.350				12.0		0.150	20.0	0.35
W164	b17.1-3	171.00	179.20	6.900	7.750	27.10	sl ge kl	0.213	0.250				12.0		0.200	20.0	0.35
W165	b17.9-1a	179.20	200.00	5.500	6.280	28.01	kl	0.229	0.450	0.500	0.500	5.0		2200			
W167	b17.9-2a	179.20	200.00	6.280	6.910	28.01	kl	0.217	0.350	0.500	0.500	5.0		2200			
W169	b17.9-3a	179.20	200.00	6.910	7.310	27.10	kl	0.200	0.350				12.0				
W171	b17.9-4a	179.20	200.00	7.310	7.750	27.10	kl	0.220	0.250				12.0				

Figure G.4: Input for the instability of the stone revetment parts for dike section 10A.

STEENTOETS versie 4.03, WL / Delft Hydraulics, dec. 2004				niveau	niveau	type		helling	TOPLAAG						BOVENSTE FILTERLAAG		
Volg-nr.	Naam van dijkvak 11A	Subvakgrenzen		onder-grens [m NAP]	boven-grens [m NAP]	toplaag	onderlagen (filter, geotex-tiel, klei, etc)	te toetsen talud/berm $\tan\alpha$	D [m]	B [m]	L [m]	spleet [mm]	open oppervlak [%]	soortelijke massa [kg/m3]	b b(min): 3 cm [m]	D15 [mm]	poro-siteit [-]
		van	tot														
W154	b16.1-1a	161.00	200.00	-0.350	2.000	26.00	pu vl kl	0.336	0.350				12.0		0.200	30.0	
W155	b16.1-1b	200.00	228.50	-0.350	2.000	26.00	pu vl kl	0.336	0.350				12.0		0.200	30.0	
W156	b16.1-1c	228.50	238.00	-0.350	2.000	26.00	pu vl kl	0.336	0.350				12.0		0.200	30.0	
W157	b16.1-2a	161.00	200.00	2.000	5.550	26.00	sl vl kl	0.282	0.450				12.0		0.120	20.0	0.35
W158	b16.1-2b	200.00	210.34	2.000	5.550	26.00	sl vl kl	0.282	0.450				12.0		0.120	20.0	0.35
W165	b17.9-1a	179.20	200.00	5.500	6.280	28.01	kl	0.229	0.450	0.500	0.500	5.0		2200			
W166	b17.9-1b	200.00	210.34	5.500	6.280	28.01	kl	0.229	0.450	0.500	0.500	5.0		2200			
W167	b17.9-2a	179.20	200.00	6.280	6.910	28.01	kl	0.217	0.350	0.500	0.500	5.0		2200			
W168	b17.9-2b	200.00	210.34	6.280	6.910	28.01	kl	0.217	0.350	0.500	0.500	5.0		2200			
W169	b17.9-3a	179.20	200.00	6.910	7.310	27.10	kl	0.200	0.350				12.0				
W170	b17.9-3b	200.00	210.34	6.910	7.310	27.10	kl	0.200	0.350				12.0				
W171	b17.9-4a	179.20	200.00	7.310	7.750	27.10	kl	0.220	0.250				12.0				
W172	b17.9-4b	200.00	210.34	7.310	7.750	27.10	kl	0.220	0.250				12.0				

Figure G.5: Input for the instability of the stone revetment parts for dike section 10B.