Wave Overtopping Resilient ‘Afsluitdijk’

Design Procedures for Landward Slope Erosion Stability during Large Overtopping Volume Events

P.M. Landa

Master of Science Thesis

Delft University of Technology

Faculty of Civil Engineering and Geosciences (CEG)

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Master of Science Thesis

For the degree of Master of Science in Coastal Engineering at Delft University of Technology

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The undersigned hereby certify that they have read and recommend to the Faculty of Civil Engineering and Geosciences (CEG) for acceptance a thesis entitled

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'Afsluitdijk'

by

P.M. Landa

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Abstract

The Afsluitdijk is unable to withstand the future conditions that belong to an annual probability of occurrence of 1/10 000 per year for water level and wave conditions. To solve this Rijkswaterstaat wants to strengthen the dike according to the principle of the wave overtopping resilient dike (NL: Overslagbestendige dijk). Therefore the Afsluitdijk has to be able to cope with large amounts of wave overtopping (>150 $l/s/m$). The current design methods are unsuitable to prove the resistance of the Afsluitdijk against these large amounts. The goal of this thesis is to develop design procedures in order to prove the landward slopes erosion resilience of the Afsluitdijk against large amounts of wave overtopping.

Based on a theoretical study a new design procedure has been developed. This method consists of six sub-procedures, two choices and two results. The first step is to determine the wave overtopping discharge. Up to 30 $l/s/m$ good quality grass can be applied. This average wave overtopping discharge appeared a good measure for the load below 30 $l/s/m$. Otherwise the load should be expressed as the front flow velocity per overtopping wave.

It is important to express the velocity for each wave separately, because only waves resulting in a higher velocity than the critical velocity contribute to the damage. Damage only occurs if the critical velocity the landward slope is able to withstand is exceeded. These wave are the number of critical overtopping waves N_{cow} or the percentage of waves that contribute to the damage P_{cow}.

The overtopping velocities are based on empirical relations between the overtopping velocity and volume. The volume has been determined using the probability of a certain wave volume to occur (Weibull distribution with a freeboard dependent scale factor a and shape factor b).

A categorization based on the discontinuities and objects that are present on the Afsluitdijk has been created. For each category an amplification factor for the front flow velocity has been derived which varied from 1 (no influence) to 2.1 (for holes). With the velocity distributions the required critical velocity can be calculated using the cumulative overload factor. In which each wave has contributes to the damage D. The influence of the storm duration and the allowable damage number D on the critical velocity has been analyzed. As well as the relation between the critical velocity, the number of critical overtopping waves and the percentage of critical overtopping waves.

The results of the procedure are a required protective top layer or required investigations to complete the procedure. These investigations can be the development of the flow velocity on the berm, gradual transition between slope and berm, transition between dike section geometry, transition between different revetment types, influence of bushes, damage as an effect of large external structures and the resilience of revetments other than grass.

The application of this procedure to the Afsluitdijk resulted in required critical velocities varying from 5.8 (smooth crest) to 12.4 m/s (rectangular structures). From tests in the past it appeared that the top layer of the Afsluitdijk is able to withstand 6.3 m/s. This is insufficient for the conditions considered in the thesis. The limits of grass are exceeded and another revetment type should be applied. For pulsative wave overtopping currently no method exists that is able to prove the resilience of other revetments than grass, this is due to the non steady state character of wave overtopping. With this knowledge and realizing that the whole outer revetment should be replaced as well, it can be questioned if the concept of an overtopping resilient dike is the best choice for the reinforcement of the Afsluitdijk.

Preface

The report in the hands of the reader is the graduation thesis of Paul Landa executed in order to obtain the degree of Master of Science at Delft University of Technology (TU Delft). This is obtained by completing the master Hydraulic Engineering at the Faculty of Civil Engineering and Geosciences (CEG) at TU Delft, specializing in Coastal Engineering.

The interest in the subject of dikes started when I did my BSc thesis, Landa [2010]. The objective of the thesis was to design a dike ring around the small village of Urk, the Netherlands. My supervisor, C. Ramkema really helped me to get a passion for Civil Engineering and especially on the subject of dikes. Now, a few years later I am graduating on what I think one may call '*the dike*' of the Netherlands, the Afsluitdijk. I think I am very lucky to be able to graduate on this statue and piece of history of Hydraulic Engineering in the Netherlands. I therefore want to thank Witteveen+Bos (W+B) and especially ing. Coen Kuiper for giving me this opportunity and taking the initiative for this graduation topic.

I also want to thank my graduation committee, prof. dr. ir. S.N. Jonkman, ir. H.J. Verhagen, ing. C. Kuiper and ir. J.B.A. Weijers for their time and patience reviewing the submitted content. The constructive feedback during the meetings and the many comments on the preliminary content really helped me obtaining the results presented in this report.

In addition to the graduation committee I would like to thank dr. ir. J.W. Van der Meer (Van der Meer Consulting bv, owner) for supplying me with knowledge and material on the subject of wave overtopping and dikes. Also ing. H.J. Regeling (Rijkswaterstaat (RWS), managing board IJsselmeergebied) for giving me information on the background of the Afsluitdijk and the choice for the overtopping resistant Afsluitdijk. They gave me the opportunity to interview them and with that contributing to a flying start setting up the research approach for this thesis.

In addition to the people who have supervised or guided me during my thesis I would also like to thank my parents, family, friends, roommates and colleagues for supporting me during my years at TU Delft and during the execution of this thesis. A more personal thank you can be found in the Acknowledgments.

Off course I want to give a special thanks to my girlfriend who is always there to support me, cheer me up and make me laugh about the stupidest little things.

Delft, University of Technology — P.M. Landa
January 9, 2014

Acknowledgments

This acknowledgment section allows me to loosen up a little and giving a special thanks to the persons important finalizing my education at TU Delft. The amount of space in the preface is to limited to thank all the people supporting me. The colleagues at W+B and especially the group K&R deserve a thanks for involving me in the group and their activities. Also for their knowledge, time and making the office in Deventer a pleasant place to work.

Off course i want to thank my friends in Delft for allowing me to bore them with my stories about the Afsluitdijk and my thesis, for the relaxing moments we had in PSOR and late night drinks in so many bars. But also for the discussions, conversations, their view and critical words on the content of my thesis. Credits for P. Vermerris, K. Vereijken, J.W.L. Wijs, H.P.A van den Heuvel and C. Biesheuvel! A special thanks goes to P.A. Kerssens, from our BSc thesis on we have always been each others biggest motivators, he has been very helpful during the period of my thesis with his criticism and foolish jokes.

Also my roommates, from which Roelof, Erik and Tim had the most influence on my thesis, personal development and relaxing times. Roelof for his lessons about life as a student, stories about work and knowledge on Civil Engineering. Erik for the 10 euro rule, which escalated so many times, the many small talks, small beers as well as the many late night discussions we had. Tim for his enthusiasm in Civil Engineering, interest in my thesis as well as for the relaxing moments watching television shows and talking nonsense while enjoying a whisky on the side.

Maybe the largest contributors to this graduation are my parents. They gave me the opportunity to be well educated and allowed me to do this the way I thought was best. I want to thank them for their patience, support and advice along the way. Also my sister Ellen, who has reviewed parts of the thesis (surprisingly effective!).

Last but not least my lovely girlfriend, Tjitske, our mutual passion for fancy restaurants, sailing and even area of expertise gives us a great connection! Also the discussions about science, way of reporting and the content of my (and your) thesis helped me to get to the results I present here. Even seven days a week together in her 16 m^2 room for almost six months did not drive her crazy! Thank you for your love and making me feel the way I do about you!

Delft, Paul
January 9, 2014

‘*We must build dikes of courage to hold back the flood of fear*’

Martin Luther King (1929-1968)

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Chapter 1

Introduction

The subject of the thesis is about one of the biggest and most outstanding hydraulic engineering projects in the history of the Netherlands: The '*Afsluitdijk*'. In order to get an idea on this subject, the current situation and the problem definition are being described in this introduction.

1-1 Current situation

The Afsluitdijk has always done the job of keeping parts of the Netherlands safe against flooding until now. However, from the latest round of tests IVW/Waterbeheer [2011] it appears that the safety of the Afsluitdijk can no longer be guaranteed up to the required safety level, which has been increased to conditions that belong to a probability of exceedance of 1/10 000 per year. These conditions are regarding the water level, the wave height and period. This safety level mismatch is regarding the crest level, armor stability of the outer slope and stability of the inner slope. The focus of the thesis is on the last mismatch. In order to make sure that the Afsluitdijk is safe for future conditions and maintains its symbolism, *Rijkswaterstaat (RWS)* (The executive body of the Dutch Ministry of Infrastructure and Environment) in cooperation with the market has done several studies on how to adapt the Afsluitdijk to these conditions. The first phase of the study resulted in four visions and two basic solutions on how to make this adaptation Rijkswaterstaat [2009]. Out of these four visions the government formed a preferred decision which is described in a Structural Vision (Structuurvisie toekomst Afsluitdijk) IenM [2011].

The vision of the commission is a comprehensive concept on the Afsluitdijk and everything that is connected to it. In addition to the dike itself, also all the hydraulic structures near/in the dike, the possibilities for renewable energy, tourism and ecology. The main focus of the project is water safety and upgrade of the water management system, this is the dike it self and the locks and weirs at Den Oever and Kornwerderzand. The main focus of the thesis is the dike it self, nevertheless the other aspects have to be mentioned and taken into account when necessary.

The vision of the government is that the dike should be made resistant against large amounts of wave overtopping as far as the dike is concerned, this has to ensure the safety of the dike at least until 2050. The argument to select this solution in favor of the other visions is the possibility of a phased construction and low initial investment costs. This may not seem like much of a task, however applying the concept of allowing large amounts of wave overtopping, or otherwise said, '*overtopping resistant dike*', is new in the Netherlands. However, already in 1954 was spoken of unbreachable see dikes in

Edelman [1954]. The principle of dikes that can handle water overtopping instead of only heightening the dikes in order to prevent overtopping is elaborated and advised in the article. The idea as such is not new at all however, really investigating the application of this concept is. Therefore additional research has to be done on this concept. This is where the assignment of this thesis comes in play, the overtopping resistant dike. With the application of this concept the Afsluitdijk can again become the symbol it used to be and a state of art piece of hydraulic engineering!

1-2 Problem description

The safety of the Dutch primary sea defenses has to be maintained to be able to withstand conditions that belong to an annual probability, for the Afsluitdijk this probability is determined on 1/10 000 per year. The third round of safety assessment of the primary water defenses in 2006 ('Derde toets primaire waterkeringen') showed that the Afsluitdijk didn't pass the current demands for water safety IVW/Waterbeheer [2011]. This is due to new guidelines, the adaptation of the hydraulic boundary conditions and the fact that the dike was never designed to be able to withstand conditions belonging to a probability of 1/10 000 per year.

Both the dike it self and the hydraulic structures (both lock complexes) within the dike didn't pass the assessment, based on crest level, macro stability outer, and landward slope. In accordance, the government decided that an orientation should be started which resulted in different visions and basic solutions. These were composed both by private companies and RWS. The results were presented in the report 'Dijk and meer' Rijkswaterstaat [2009]. Following, the government made a vision of what they thought would be the optimal solution out of the earlier presented visions and solutions. This vision was presented in the report 'Structuurvisie toekomst Afsluitdijk' IenM [2011].

The choice of RWS to select the overtopping resistant dike as most cost effective and preferred solution was based on the feasibility and affordability of the different solutions. This together with the adaptability as well as expected esthetic's resulted in the overtopping resistant dike. One might argue whether or not this concept is totally new, currently certain amounts of wave overtopping is allowed, the quantities are however an order of magnitude smaller than considered in the 'new' concept. There are a two important aspects that have to be concerned looking at this concept:

- Strengthening of the cross section of the dike
- Reinforcement of the hydraulic structures within the dike for water safety and water management.

These are the main items concerning the overtopping resistant dike. Within these items different elements can be distinguished. Because the scope of this thesis will be mainly on the first item, the second one will not be elaborated much further.

An adaptation of the cross section might reduce the amount of overtopping. Especially in combination with a redesign of the outer slope revetment (of which approximately 1/3 failed during the safety assessment, however during an assessment of Witteveen+Bos (W+B) with the new hydraulic boundary conditions almost the whole outer revetment appeared to be insufficient). It doesn't matter what the exact amount of overtopping will be. The fact, and with that the starting point of this thesis, is that there will be a large amount of overtopping. Larger than currently allowed, used for dike design in the Netherlands the maximum allowable amount is 1 to 10 l/s/m. The amounts that the Afsluitdijk will have to deal with are an order of magnitude larger than the current allowable amounts. This results in problems and challenges that have to be investigated.

This research is related to that feasibility study. At this moment it is still unknown whether or not it is possible to convert the Afsluitdijk into an overtopping resilient dike, how and if this can be proven. The one thing that is known and with that the starting point of this research is that there will be large amounts of overtopping over the crest of the Afsluitdijk.

1-2-1 Problem definition

With the choice of Rijkswaterstaat to execute a feasibility study on the overtopping resistant Afsluitdijk new challenges are being introduced to the water safety system of the Netherlands. One of these challenges is that the crest, the landward slope and inner berm of the dike need to cope with high turbulence water flow. A lot of research has been done on the subject of wave overtopping for dikes. However, only limited on the amounts considered in this case with an order of magnitude of 100 to 200 l/s/m. Because the resilience against overtopping will depend on the weakest links, the focus has to be on these links. These can be proven using the Delta flume or the Wave Overtopping Simulator. However, currently in the Netherlands there isn't a 'standard theoretical recipe' in order to determine these weak spots and prove the resilience, or safety, against overtopping. This leads to the problem definition:

> The current predictive methods are unsuitable to prove if the landward slope and berm of the Afsluitdijk are able to withstand large amounts of overtopping water.

1-2-2 Research objectives

The main objective of the project is to prove the ability of the Afsluitdijk to resist large amounts of wave overtopping. In order to realize this, mapping the weakest links of the landward slope, subsequently develop a procedure in order to prove the resistance against wave overtopping of the weak links on the landward slope of the Afsluitdijk and how to adapt them when appearing insufficient, summarizing:

> Develop predictive procedures in order to prove the landward slope resilience of the Afsluitdijk against large amounts of wave overtopping water.

Research question

Research questions have been composed in order to solve the problem definition and reach the research objective. The main research question has been subdivided into several sub questions. The main question follows from the problem definition and research objective and can be stated as:

> *Can a predictive procedure be developed in order to prove that the landward slope and berm of the Afsluitdijk are able to withstand large amounts of wave overtopping water?*

subquestions are:

- What are the main physical processes when it comes to the loads induced by large amounts of wave overtopping? (Overtopping discharge, flow velocity, flow depth etc.)
- What are the main physical processes when it comes to the resilience against large amounts of wave overtopping? (Layer thickness, depth of roots, soil parameters etc.)
- What is the relation between the load and resistance parameters?
- What are (new) limit states that need to be taken into account when wave overtopping is allowed? (Infiltration, internal and external erosion etc.)
- For which parameters additional information is needed and with what methods can these be determined? (Delta flume, Wave Overtopping Simulator, Numerical tools)
- What are the critical points of the landward slope and berm?
- Are these critical points able to withstand large amounts of overtopping and how can that be proven?

By answering these question the main research question can be answered and implications for design can be given. When questions are being answered these will be repeated at the end of the section in which the answer can be found.

1-3 Reading guide

The report can be divided in three parts. Part 1 is an analysis of the current situation and the current state of knowledge (chapters 2 and 3). Part 2 consist of the derivation and application of the design procedure (chapters 4, 5 and 6). Part 3 is a reflection on the design procedure in which conclusions are drawn (chapters 7 and 8). A short description of each chapter can be found below.

Part 1: Current State

Chapter 2: Literature review gives a summary of the available relevant literature on the subject of the thesis, an overview of the available literature can be found at the end of each section. Also the different interviews that are held are described here. The goal of this chapter is to gain insight in the background of the subject

Chapter 3: Study area gives a description of the area on which the research is conducted, both on the relevant scientific area of interest as geographical. The goal of this chapter is to get a feeling for the background of the Afsluitdijk and its surroundings.

Part 2: Derivation and Application of Design Procedure

Chapter 4: Methodology describes the boundary conditions of the thesis, the analysis of the methods that are used to transform the input into a required top layer and the resulting design procedures. Due to the fact that the research that is conducted is mainly theoretical, there will be some overlap with the literature review. This chapter mainly describes the derivation of the design procedure.

Chapter 5: Results, in this chapter the proposed main design procedure is presented after which it is decomposed in six different boxes it consists of with a more detailed design procedure. These sub procedures will be applied to the case of the Afsluitdijk and an analysis of these results will be done.

Chapter 6: Required investigations, the proposed design procedure has some open endings. When the procedure leads to additional investigations, these investigations have to be done in order to complete the procedure. If the procedure does not lead to additional research, these investigations are not necessary.

Part 3: Reflection on application and derivation of Design Procedure

Chapter 7: Discussion will evaluate the used methodology, assumptions and limitations of this thesis. The goal of this thesis is to see which parts of the results are not jet complete or which assumptions might be not completely valid. This all with the knowledge of the completed thesis.

Chapter 8: Conclusions and recommendations will be used to evaluate the results as presented in chapter 5. In addition to this also state some useful recommendations on further research and how to approach such research. The questions stated in section 1-2-2 will be answered here, as well as the possible solution to the stated problem.

Part I

Current State

Chapter 2

Literature review

This chapter gives a summary of the available literature on the subject of the thesis. In order to get a good insight on the background of the subject and define a good problem definition, research objective and questions this literature study has been started already during the orientation phase of the thesis. A division in different topics has been made to keep a good overview on what literature is available. A list of collected literature can be found at the end of every section in this chapter. However, not all the enlisted literature is used, it might be useful for further or other research on similar topics. The literature that is actually used for this thesis can be found in the bibliography at the end of the thesis. This literature review consists of these subject:

- Current design practice
- Overtopping
- Non water retaining objects and discontinuities
- Strength of grass
- Research and reports on the Afsluitdijk
- Residual strength and breaching
- Spillways
- Interviews

2-1 Current design practice

The current design practice is based on certain limit states, these limit states are coupled to different failure mechanism. In literature the failure mechanisms are often seen as limit states, however a limit state is not the same as failure mechanism. The limit state is a limit value for the mechanism. If such a limit state occurs the dike is likely to fail. An overview of the possible failure mechanisms that can occur in case of a dike are depicted in figure 2-1 Schiereck [1998a]. The failure mechanism of overtopping is the main mechanism that needs to be taken into account, the red crossed mechanisms are mechanisms that will not be taken into account and have no direct relation with the mechanism overtopping. The dotted red crossed mechanisms are mechanisms that occur as a result of overtopping however, this is due to the infiltration, which is only partly caused by wave overtopping. These will be

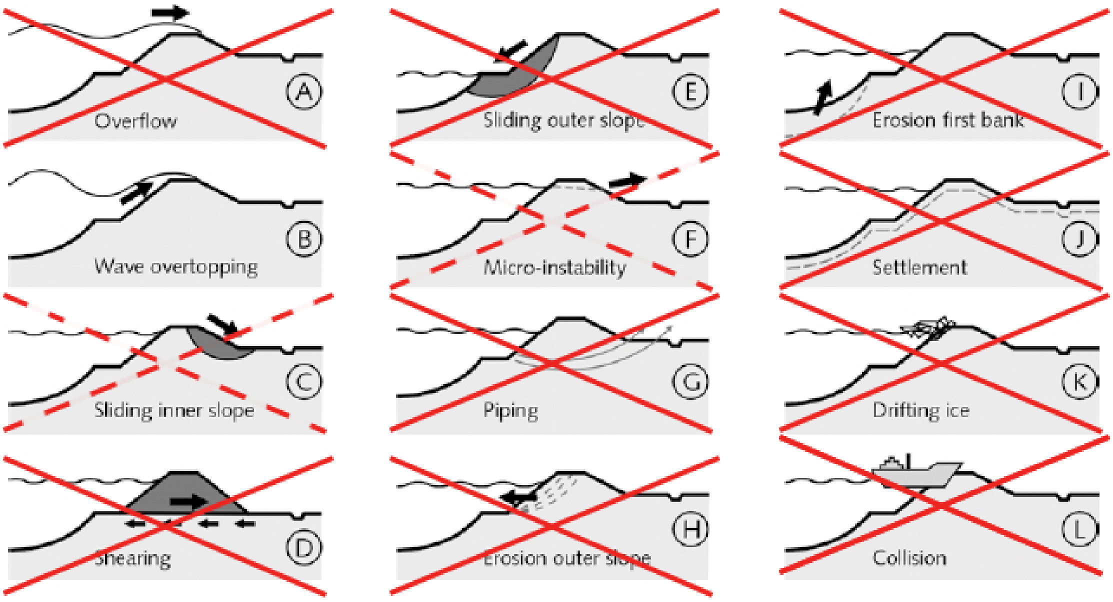

Figure 2-1: Failure mechanisms

mentioned and elaborated shortly, but this is a separate problem apart from the erosion. Currently, using these limit states often results in a traditional widening and heightening of the dike.

A: Overflow occurs when the water level on the outside of the dike is higher than the crest of the dike, inundation of the area behind the dike can occur without the dike actually failing. The crest height of the dike is simply to low.

B: In this case overtopping is allowed and therefore it isn't a limit state or failure mechanism any more as such. Often erosion of the landward slope by the water flowing over the dike causes failure, however infiltration causing shearing can also be a cause. It can happen when a combination of high water level and big waves occurs. (This phenomena will be elaborated further on)

C: The landward slope is unstable and might slide, often caused by a high water level on the outside in combination with infiltration and a raised water pressure in the subsoil. This causes a reduction of the effective pressure on the soil particles and a soil that is heavier when saturated.

D: Shearing of a part of the dike is also caused by high water pressure and a raised pressure of water in the subsoil. A whole part of the dike can slide landward. This is because the friction is reduced by the reduction of the effective stress.

E: This phenomena can occur if the water level in front of the dike drops very fast. Overpressure inside the dike might cause the sliding of the outer slope.

F: Smaller particles are being transported by seepage water, this causes instability of the landward slope. When the phreatic waterline inside the dike is being raised, so that is connects with the landward slope of the dike, then this mechanism might occur.

G: Piping is quite similar to micro-instability but it occurs underneath the cohesive layer in the subsoil. The seepage water might erode particles from underneath the dike body and forms a pipe.

H,I: Being attacked by waves and flow the outer slope and the first bank can both be eroded.

J: The soil can settle on a large scale which causes instabilities or lowering of the crest height.

K,L: Treats caused by collisions of ice and ships

Currently these mechanisms are prevented, because it is likely that if one of these failure mechanisms occurs that it might cause failure of the dike. This also counts for mechanism B. Currently the design is performed in such a way that wave overtopping is limited to small overtopping discharges (1 $l/s/m$). Other failure mechanisms than wave overtopping are not a part of this thesis, therefore only the design practice with respect to overtopping will be described.

Overtopping is the mechanism that a high water level reaches high up the outer slope of the dike, but does not flow over it. However, in combination with high waves, which causes water to run-up the outer slope, water can get over the crest of the dike and even reach the landward slope. When the water reaches the crest it can start infiltrating and will flow down the landward slope. Currently in the Netherlands wave overtopping itself is seen as a failure mechanism (limit state) Weijers and Tonneijck [2009]. The overtopping of waves is related to the water level. The amount of water the landward slope can withstand is defined as a limit state. This amount is depending on the state of the landward slope. The limit state can be different for adjacent sections of a dike, for instance if the landward slope is different. If this limit is exceeded it is very well possible that the stability of the dike is jeopardized.

The reason that the strategy of dike design in the Netherlands was to only allow small amounts of overtopping is that during the 1953 flooding, many dikes in the south west collapsed as a result of landward slope shearing following from overtopping. Therefore the crest height was chosen in such a way that only small amounts of overtopping occurred, depending on the strength of the landward profile, 0.1, 1 or 10 l/s/m. The average amount of wave overtopping that is expected to run over the dike can be calculated with equation 2-1 Verhagen et al. [2009].

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot exp\left(-2.6 \cdot \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_b}\right) \tag{2-1}$$

with:

q	average overtopping discharge	$[l/s/m]$
g	gravitational acceleration	$[m/s^2]$
H_{m0}	wave height from zero-th moment of spectrum	$[m]$
R_c	Freeboard	$[m]$
γ_f	reduction factor permeability and roughness	$[-]$
γ_b	reduction factor berm	$[-]$

The above formulation only gives an absolute maximum and the angle of the slope is not represented in the formula. To take this into account the dimensionless wave overtopping discharge can be calculated using equation 2-2

$$Q = \frac{0.067}{\sqrt{\tan(\alpha)}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.75 \cdot \frac{R_c}{\xi_{m-1,0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{2-2}$$

with:

Q	dimensionless wave overtopping	$[-]$
α	slope of the revetment	$[-]$
$\xi_{m-1,0}$	Iribarren number	$[-]$
γ_β	reduction factor angle of attack	$[-]$
γ_v	reduction factor vertical wall	$[-]$

Using equation 2-3, given the dimensionless overtopping Q, the average overtopping discharge q can be calculated.

$$Q = \frac{q}{\sqrt{g \cdot H_{m0}^3}} \tag{2-3}$$

The crest height of the dike is often defined as a function of the run-up level $R_{u2\%}$. This is the run-up level that is expected to be exceeded by 2% of the incoming waves. This can be calculated using equation 2-4 Verhagen et al. [2009].

$$\frac{R_{u2\%}}{H_{m0}} = min\left(A \cdot \gamma_b \cdot \gamma_r \cdot \gamma_\beta, \gamma_r \cdot \gamma_\beta \cdot \left(B - \frac{C}{\sqrt{\xi_0}}\right)\right) \tag{2-4}$$

with:

$R_{u2\%}$	run-up level exceeded by 2% of the waves	$[m]$
A	curve fitting parameter	$[-]$
B	curve fitting parameter	$[-]$
C	curve fitting parameter	$[-]$
γ_r	reduction factor roughness	$[-]$
ξ_0	Iribarren number with deep water wave length	$[-]$

The 2% of the waves that is exceeding this level is overtopping the crest of the dike. This run-up level now is used in order to calculate the number of overtopping waves.

Currently the allowable amount of overtopping is given by the strength on the crest and on the inside of the dike table 2-2 gives some indicative mean discharges concerning the strength of the crest and slope. For dike design in the Netherlands currently allowable overtopping criteria of 1 and 10 $l/s/m$ apply for design conditions, depending on the local situation. To get a feeling for these numbers according to Pullen et al. [2007] in table 2-1 some guidance to which certain hazard belong is given. These are tolerable discharges for the use of the area directly behind the dike. These are thus not limits related to failure of the landward slope. The amounts of water that will overtop the dike depend on a couple of important variables these are:

- The relative free board

  $$\frac{R_c}{H_{m0}} \tag{2-5}$$

  Schiereck [2001]

- Wave run-up, which depends on the surf similarity parameter (Iribarren number)

  $$\xi = \frac{\tan\alpha}{\sqrt{\frac{H}{L_0}}} \tag{2-6}$$

  Schiereck [2001], roughness of the revetment γ_r, angle of attack of the waves γ_β and the presence of a berm γ_B

The different reduction factors can be determined with the formulations below Pullen et al. [2007]:

$$\gamma_B = 1 - \frac{B_B}{L_B}\left[0.5 + 0.5 \cdot \cos\left(\pi\frac{h_B}{x}\right)\right] \tag{2-7}$$

$$\gamma_\beta = 1 - 0.0033 \cdot |\beta| \tag{2-8}$$

Table 2-1: Tolerable discharges and volumes for use of area behind a dike

Type	Hazard, reason	Mean discharge q $(l/s/m)$	Max volume $V_{max}(l/m)$
Vehicles	Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01-0.05	5-50
Pedestrians	Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only no falling jet, low danger of fall from walkway	0.1	20-50
Property	Damage to equipment set back 5-10 meter	0.4	-
Property	Building structure elements	1	-
Pedestrians	Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway	1-10	500
Property	Sinking small boats set 5-10 meter from wall, damage to larger yachts	10	1000-10000
Vehicles	Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed	10-50	100-1000
Property	Significant damage or sinking of larger yachts	50	5000-50000

Table 2-2: Limits for overtopping for damage to the defence crest or landward slope

Hazard type and reason	Mean discharge q $(l/s/m)$
Embankment seawalls/sea dikes	
No damage if crest and slope are well protected	50-200
No damage to crest and landward slope of grass covered embankment of clay	1-10
No damage to crest and landward slope of embankment if not protected	0.1

In which B_B is the width of the berm, L_B the intersection at both sides of the berm of the slope at a vertical distance equal to the significant wave height H_s from the horizontal center of the berm, h_B the water depth on top of the berm, x is twice the significant wave height and β the angle of the incoming waves.

These methods are only able to determine the amounts of water that will overtop the dike and the limits are related to some guidelines on the resistance of the dikes landward slope and activities on the inner side of the dike. It is only partly based on how much overtopping the dike can withstand. Therefore more information is needed on the subject of overtopping it self and what is caused by that.

The method described above is a combination of a deterministic approach and quasi probabilistic approach Schiereck [2001]. With a probabilistic approach the resulting function of strength and load is expressed as a limit state function $Z = R - S$ in which R is the resistance and S is the load or solicitation. $Z = 0$ is the limit state, if the value of $Z > 0$ then there will be no failure and if the value of $Z < 0$ failure will occur. There are four different levels of probabilistic approaches.

Level 0: Deterministic approach isn't a probabilistic approach, based on experience and intuition

Table 2-3: Design

Year	Title	Author(s)
1998	Fundamentals of water Defences	TAW
2006	Voorschriften toetsen veiligheid primaire waterkeringen	VenW
2010	Haalbaarheidsonderzoek semi-probablistische toetsvoorschriften	Deltares
2012	Handreiking toetsen grasbekleding op dijken tbv verlengde derde toetsronde	Rijkswaterstaat

the maximum load and minimum strength is taken. In addition to this an overall safety factor is applied. However the safety factor is misleading because it doesn't necessary mean more safety. With a different distribution the same safety coefficient can give another probability of failure.

Level I: quasi-probabilistic approach With this method not an overall safety factor is used, but a partial safety factor for every parameter involved is derived. This is often done based on a level II approach. The partial safety factors γ are based on the α (the relative importance of each parameter) and β (the required safety) values. The values of α are negative for loads and positive for strength, resulting in $\gamma > 1$ and $\gamma < 1$ for loads and strength, respectively.

Level II: approximate probabilistic approach In this method the limit state function Z is described with a normal distribution, therefore giving a probability of failure, the failure probability is derived from linearizion around the point Z=0, the so called design point. The parameters involved also have a normal distribution to describe the probability of occurrence of a certain value of that parameter. These distributions do result in the normal distribution which describes the limit state function. The mean μ and the standard deviation σ result in $\beta = \frac{\mu}{\sigma}$, which is the total required safety. In addition by using this approach also an indication on the importance of each parameter on the overall probability of failure can be given, this is expressed in the α values for each parameter, the relative importance.

Level III: fully probabilistic approach A fully probabilistic approach can be done in different ways, one of the most used is a Monte Carlo method. A probability distribution is assigned to every parameter, this can be all kind of different probability density functions. In each 'round' a value for each parameter will be drawn and a value for Z can be calculated. This is repeated several times, this results in a probability distribution for Z. The total probability of failure can be calculated using $P_F = \frac{N_F}{N}$. N is the total number of draws and N_F is the number of draws where $Z < 0$.

The above described method is actually a level 0 approach, however a certain safety factor is applied, but that is not based on the level II approach. Therefore a combination of the level 0 and level 1.

2-2 Wave overtopping processes

To gain more insight in the mechanism of overtopping, it should be split into the different problems it causes, which can lead to failure of the dike. Two main mechanisms can be distinguished:

- Water infiltrating by wave overtopping leading to an increase of the phreatic water line inside the dike. This might may cause instabilities of the landward slope.
- Water flows over crest and reaches the landward slope and causes erosion of the landward slope and berm.

Wave overtopping is induced by incoming waves that break on the slope of the dike. This causes a wave impact on the slope and a wave front will run-up the slope. Overtopping will occur when the crest level is exceeded by the run-up level. The water can infiltrate into the dike body via the

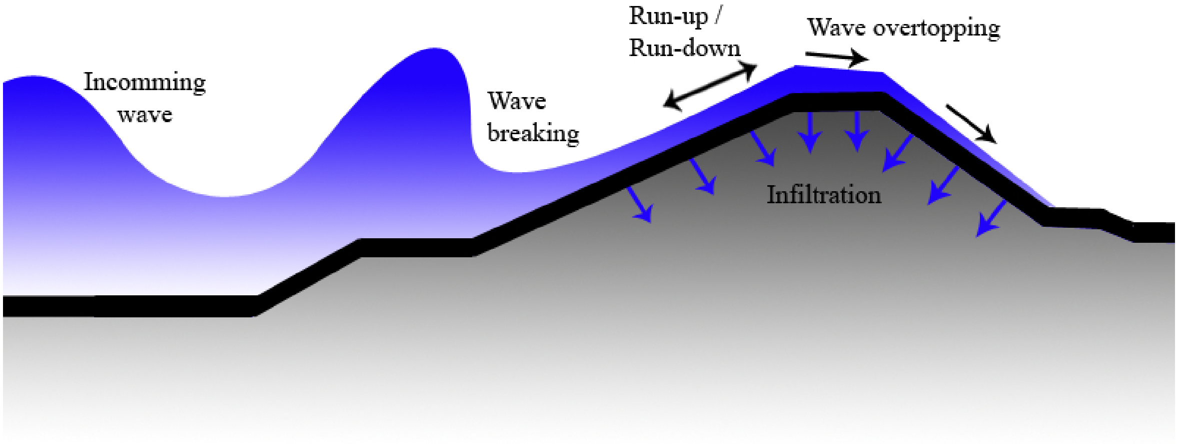

Figure 2-2: Infiltration and wave overtopping

outer slope, the crest and the landward slope. The ratio between the infiltration, water transmitted towards the landward slope and the total quantity of overtopping water is been given for rubble mount breakwaters with a sandy core by equation 2-9 and 2-10 Steenaard [2002].

$$\frac{q_2}{q_1} = \left(\frac{Q^*_{tot} - Q^*_d}{Q^*_{tot} + 0.07}\right) forQ^*_{tot} > Q^*_d \tag{2-9}$$

$$\frac{q_2}{q_1} = 0forQ^*_{tot} < Q^*_d \tag{2-10}$$

with:

q_1	discharge over the crest	$[m^2/s]$
q_2	total overtopping discharge	$[m^2/s]$
Q^*_{tot}	dimensionless total wave overtopping discharge	$[-]$
Q^*_d	limit value for the dimensionless overtopping discharge	$[-]$

One of the questions that immediately rises is whether or not this formula is valid for the case of the Afsluitdijk. The crest of the breakwater, during these tests, consisted of an armor layer with a basket for collecting the water underneath. Therefore it seems that this formula is not valid in the case of the Afsluitdijk. However, it might be interesting to look into to see what processes should be taken into account. Nevertheless a distinction between infiltration and erosion should be made.
As a continuation on the above study Lioutas [2010] has performed experiments to determine the total overtopping, the overtopping directly behind the crest and the distribution of overtopping water behind the crest. The conclusion is that the overtopping discharge at a certain distance behind the crest can be determined using equation 2-11

$$\frac{Q}{\sqrt{g \cdot H^3_{m0}}} = (0.2 - 0.133 \cdot k)\left(\frac{\gamma_b \cdot \xi_0}{\sqrt{\tan\alpha}}\right) \cdot \exp\left[-(2.6 - 2.15 \cdot k)\frac{R_c}{H_{m0}}\frac{1}{\gamma_f \cdot \gamma_b \cdot \gamma_\beta \cdot \gamma_v \cdot \xi_0^k \cdot \gamma_c}\right] \tag{2-11}$$

with

$$\gamma_c = -0.164 \cdot \frac{x}{B} + 0.677 \tag{2-12}$$

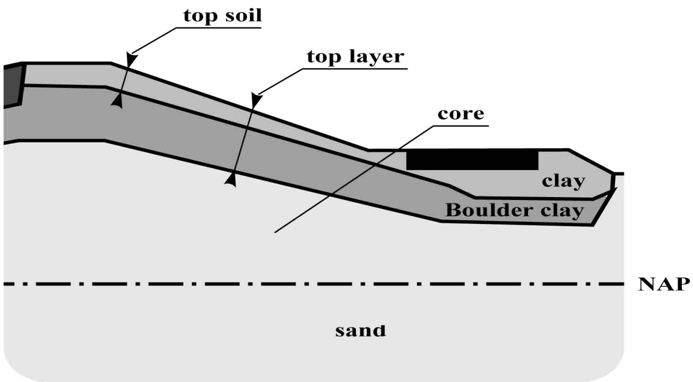

Figure 2-3: Landward slope definitions

In this equation B is the crest width, which together with the distance from the crest gives a reduction factor. For the experiments conducted with a permeable top layer, the suggested value for γ_c is 0.7 for the discharge directly behind the crest. k is the parameter which takes into account the breaking of waves (1 for breaking and 0 for non-breaking). So depending on these parameters the distribution behind the crest can be determined. In addition to the above described separated infiltration and erosion, the flowchart depicted in figure 2-4 is created. The flow chart shows how the event overtopping can lead to failure. What is considered to be failure is seen as what can be demonstrably showed. Currently this is failure of slopes or certain layers on the landward slopes. *Overtopping* can lead to three mechanisms, *occurrence of a rupture*, *infiltration* and *surface erosion*. However when a rupture occurs this can lead to infiltration or surface erosion which did not happen (or not as fast as) before the rupture occurred.
Surface erosion can cause *erosion of the landward slope top soil*, which leads to *erosion of the landward slope top layer*.

Infiltration gives a different path to failure. It can cause *internal erosion* and a *raise of the phreatic water line*, the both can lead to *shearing of the landward slope top layer*. Raise in the phreatic water line can also cause *shearing of the landward slope* as a whole or *pushing off parts of the landward slope top layer*. Shearing of the landward slope top layer, shearing of the landward slope, pushing off landward slope top layer and erosion of the landward slope top layer are often considered as a failure. When overtopping continues they will all lead to *retrogressive erosion* which can lead to *lowering of the crest level* which gives more overtopping and can eventually cause *erosion of the remaining profile*. The last is considered as a total failure of the dike.

The last three items, retrogressive erosion, lowering of the crest level and erosion of the remaining profile are part of the so called breaching process, in which the dike still maintains its function of retaining the water. One could say that the dike during that stage has a certain residual strength. The residual strength has been a topic under investigation however not very well understood up till now. The residual strength process is described in detail and methods have been evaluated in Knoeff and Verheij [2003] also in Bretler et al. [2010] an inventory has been done on this subject. The process on which the residual strength actually depends is called breaching, as mentioned before this process is being investigated for the Afsluitdijk in Visser [2002].

Infiltration instability of the landward slope

With small amounts of wave overtopping infiltration often isn't a problem to the stability of the landward slope. Only the high water level on the outer side of the dike causes water to infiltrate and only a little infiltration is present on the crest and landward slope. However with the amounts under

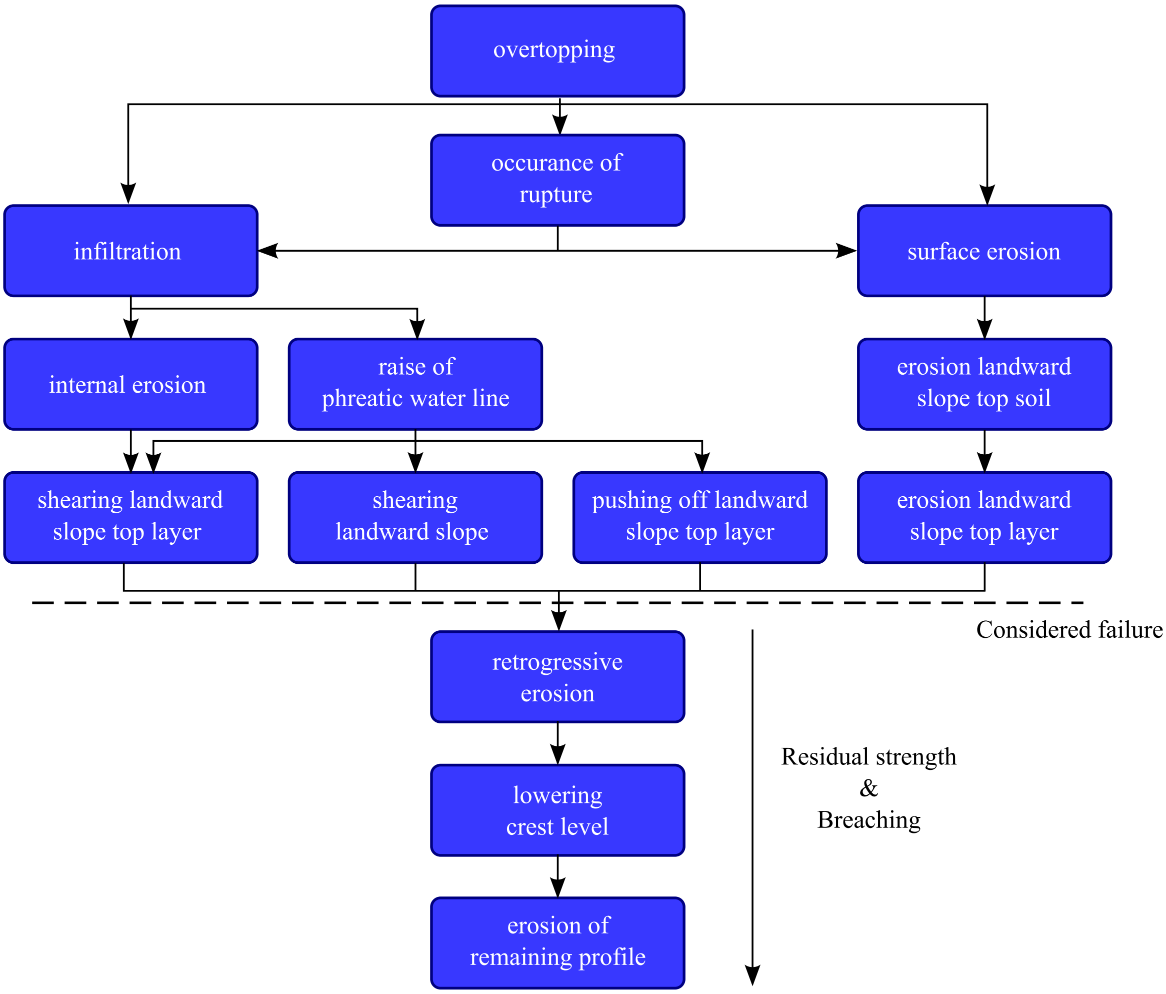

Figure 2-4: Flowchart overtopping process

consideration here, it can be problem for the stability of the dike. Especially when these large amounts of overtopping are allowed, a pulsating water flow is present during a large period of time. Which means that a small layer of water is present on the dike, which can infiltrate in the crest and landward slope. This leads to an additional contribution to the already present infiltration caused by the high outer water level. In the current guidelines for this failure mechanism, is stated that: The mechanism does not occur if the overtopping discharge is not significant, this means 0.1 l/s/m, or the slope is less steep than 1 : 4. In the case of the Afsluitdijk both criteria are not met and that means that the stability should be checked. For a clay cover on a sandy core a lot of uncertainties arise concerning the water table within the dike. A three step method is available in order to determine the pore pressure build up within a dike body. This is extensively described in van Hoven et al. [2011]:

- Determine the infiltration time (s)
- Determine the infiltration capacity (m^3/s per m^2)
- Determine the potential pore pressure build up, step 1 times step 2 this gives a volume (m^3).

Because of this in 2009, together with overtopping tests, infiltration tests where done on the Afsluitdijk by Infram commissioned by Deltares, reported in Factual report Infram [2009]. The results are interpreted by Deltares, these are presented in Sterkte en Belastingen Waterkeren (Strength and loads

on water defences) (SBW) Golfoverslag en Sterkte Grasbekleding. Deltares [2010]. The infiltration rate was 0.1 $l/s/m$ and the total rise of the phreatic water level was up to 1.9 meter in 56 hour.
Depending on the permeability, the duration and the amount of overtopping the sincerity of the problem can be determined. Infiltration can cause local instability followed by more extensive infiltration causing loss of macro stability. For a sandy dike with a top layer of clay, which the Afsluitdijk is, there are two main issues. Due to a combination of high water on the outside of the dike and infiltration of water in the crest and the landward slope, the phreatic line in the dike might rise. The tests that were done by Infram did not take into account the rising of the water level of the Waddenzee, but only infiltration. The effect of a storm in combination with overtopping therefore could be even a bigger rise of the phreatic level than measured during the tests. When this happens the soil particles might be washed out causing failure of the dike.
If pressure level under the landward slope rises higher than the boundaries of the slope it could be that the revetment is lifted up. This happens when the weight of the revetment is smaller than the hydrostatic pressure in the dike.
Another mechanism that can occur when the revetment is not jet pushed out is shearing of the inner revetment. Due to the high water pressure the effective stress on the boundary between the core and the revetment becomes less, this causes the shearing resistance of the layer as a whole to become less. One can imagine that when this happens during wave overtopping events, the erosion of the landward slope will increase a lot.
Testing for push up of the clay layer can be done using, the friction between the soil on both sides is taken into account:

$$\frac{2 \cdot c \cdot d}{\gamma_{m,c}} + \frac{\rho_g \cdot g}{\gamma_{m,\rho}} \cdot \Delta x \cdot d \cdot \cos\alpha + \frac{\rho_g \cdot g}{\gamma_{m,\rho}} \cdot \Delta x \cdot d \cdot \sin\alpha \frac{\tan\phi}{\gamma_{m,\phi}} \geq \gamma_n \cdot \gamma_d \cdot (\Delta h - \frac{1}{2}\Delta x \cdot \sin\alpha) \frac{\rho_w \cdot g}{\gamma_{m,\rho}} \Delta x \quad \text{(2-13)}$$

For shearing the following equation can be used, which again takes into account the resistance at the toe of the slope. Both equation 2-13 and 2-14 can be found in Weijers and Tonneijck [2009].

$$\gamma_n \cdot \gamma_d \cdot \left(\Delta h \cdot d \frac{\rho_g \cdot g}{\gamma_{m,\rho}} - \left(\frac{c \cdot d}{\gamma_{m,c}} + \left(\frac{\Delta h}{\tan\alpha} \cdot d \cdot \frac{\rho_g \cdot g}{\gamma_{m,\rho}} - 1/2 \frac{\rho_w \cdot g \cdot \Delta h^2}{\gamma_{m,\rho} \cdot \sin\alpha} \right) \frac{\tan\phi}{\gamma_{m,\phi}} \right) \right) \geq \frac{c \cdot d}{\gamma_{m,c} \cdot \sin\alpha} + 1/2 \cdot \frac{d^2 \cdot \rho_g \cdot g}{\sin\alpha \cdot \gamma_{m,\rho}} \quad \text{(2-14)}$$

With:

d	Assumed layer thickness (perpendicular to the slope)	$[m]$
c	cohesion for clay	$[kN/m^2]$
$\gamma_{m,c}$	(=1.250) material factor for cohesion	$[-]$
$\gamma_{m,\phi}$	(=1.1) material factor for angle of internal friction	$[-]$
$\gamma_{m,p}$	(=1.0) material factor for the volumetric mass	$[-]$
γ_d	(=1.1)	$[-]$
γ_n	(=1.1)	$[-]$
ρ_g	volumetric mass of soil	$[kg/m^3]$
ρ_w	volumetric mass of water	$[kg/m^3]$
Δx	width of slice of ground, parallel to the slope	$[m]$
ϕ	angle of internal friction	$[-]$

Erosion of the crest, landward slope and berm

There are different places where erosion due to wave overtopping can occur, at the crest, the landward slope or the inner berm of the dike. As stated earlier and depicted in figure 2-4 erosion starts with surface erosion, this results in erosion of the top soil and eventually in erosion of the top layer. One of the main question will be what is considered to be failure as a result of erosion, or what is acceptable during extreme conditions? Based on equation 2-1 and the limits depicted in table 2-1 and 2-2 it seems like the parameter q is the variable on which dike design is based. Currently this is often the case, this means that q is seen as a good measure for erosion.

From testing it appears that large quantities of overtopping are more critical for the erosion of the grass cover than small amounts, therefore it can be questioned if taking the average overtopping is good representation for the allowable load. Based on three different criteria the failure can be described Dean et al. [2010]:

Erosion due to excess velocity:	$E = K\Sigma\left((u-u_c)\,t\right)$
Erosion due to excess shear stress:	$E = K\Sigma\left(\left(u^2-u_c^2\right)t\right)$
Erosion due to excess of work:	$E = K\Sigma\left(\left(u^3-u_c^3\right)t\right)$

It appeared that erosion due to excess of work consistently had the smallest standard error if large amounts where considered, therefore this was chosen as the superior method based on these tests. If the larger amounts appear more critical the process of erosion becomes a matter of fatigue, because of this another criterion has been developed based on the number of large waves that result in a larger flow velocity than the critical flow velocity: The cumulative overload, which is defined as van der Meer et al. [2011]:

$$\sum\left(U^2-U_c^2\right) \tag{2-15}$$

The cumulative overload is based on excess of shear and not on excess of work. Next to this the duration is not included, this is because the duration is not very important for wave overtopping it is the number of waves that exceed a value of the shear stress, if overflow occurs the duration would be more important. The three equations described above are more suited for overflow and the cumulative overload is more appropriate for overtopping. In addition to this, currently there is a discussion whether or not the erosion due to excess of work is indeed the criteria with the smallest error. The cumulative overload has three different thresholds:

- First damage
- various damages
- Failure (at 3500 $[m^2/s^2]$)

The calculation of the flow velocity U depends on the geometry of the dike. The type of grass will give higher or lower values for failure. Often the flow velocity U is needed, an empirical relations based on overtopping tests:

$$U = 0.5\cdot V^{0.34} \tag{2-16}$$

Next to this the overtopping flow depth is an important parameter as well and is also given as a function of the wave volume V:

$$h = 0.133\cdot V^{0.5} \tag{2-17}$$

Note that the factor 0.5 and 0.133 are not dimensionless. However, the power of both the velocity and flow depth are still under discussion and should always be used with care. The volume of overtopping wave can be calculated using Hughes et al. [2012]:

$$P_v = 1 - exp\left[-\left(\frac{V}{a}\right)^b\right] \tag{2-18}$$

and

$$a = 0.84 \cdot T_m \cdot q \frac{N_w}{N_{ov}} \tag{2-19}$$

In which N_w is the number of waves and N_{ow} (number of overtopping waves) is given as:

$$N_{ow} = N_w P_{ov} = N_w \cdot exp\left[-\left(\sqrt{-ln0.02} \cdot \frac{R_c}{R_{u2\%}}\right)^2\right] \tag{2-20}$$

The critical flow velocity U_c can be determined according to Hoffmans [2012]:

$$U_c = \alpha_{grass,u} \cdot r_0^{-1} \cdot \sqrt{\psi_c \cdot \sigma_{grass,c}(0)/\rho} \tag{2-21}$$

In which:

$$\alpha_{grass,u} = \alpha_0 \cdot \sqrt{1 + 3 \cdot \alpha_{grass}} = 2.0 \tag{2-22}$$

For $\alpha_{grass} = 0.64$ and $\alpha_0 = 1.2$

$$\sigma_{grass,c}(0) = \frac{A_{root}}{A_1} \sigma_{root} \tag{2-23}$$

More detail can be found in Hoffmans [2012]

with:

E/K	erosion rate	$[m]$
U	flow velocity of overtopping wave	$[m/s]$
U_c	critical flow velocity	$[m/s]$
t	flow duration	$[h]$
V	volume of overtopping wave	$[m^3/m]$
h	flow depth of overtopping wave	$[m]$
P_v	probability of a certain overtopping wave volume	$[-]$
a	scale parameter	$[m^3/m]$
b	shape factor	$[-]$
T_m	mean wave period	$[s]$
N_w	number of waves	$[-]$
N_{ow}	number of overtopping waves	$[-]$
P_{ov}	probability of overtopping	$[-]$

Currently van der Meer is working on additional papers on the subject of wave overtopping. One of the papers is focusing on the factor b in equation 2-18, currently it is 0.75. The paper describes the distribution of this b value. That for a small value of b describes a small number of waves overtopping the structure however with a large volume. A large value of b means that more waves will overtop the structure with more or less the same volumes Zanuttigh et al. [2013]. The value of b is coupled to the relative free board, it appears that the value increased significantly for low crested structures. This is very clear for smooth structures, however not so good for ruble mount structures. The same analysis is also done based on the relative discharge, so the b value is directly coupled to the discharge. This can be very important because this can mean that with certain wave height the actual measurements on the Afsluitdijk might appear to actual represent an large discharge than currently assumed. To summarize the above, below some hazards and parameters that are important when looking at overtopping are being presented as stated in the Pullen et al. [2007]:

Table 2-4: Overtopping

Year	Title	Author(s)
2002	Verdeling van overslaand water op een golfbreker	Steenaard
2002	Technical Report Wave Run-up and Wave Overtopping at Dikes	TAW
2008	Direct hazards from wave overtopping	Allsop et al.
1979	Wrijvingskrachten op het binnentalud van een dijk	Kruiningen
2001	Low exceedence wave overtopping events	Gent
2005	Layer thickness and velocities of wave overtopping at sea dikes	Schüttrumpf and Oumeraci
2006	Wave overwash at low crested beach barriers	Quang et al.
2007	Velocity and depth variations during wave overtopping	Bosman
2008	Individual overtopping events at dikes	Bosman et al.
2010	Erosional equivalence of levees steady and intermittent wave overtopping	Dean et al.
2011	Destructive wave overtopping tests on flemish dikes	Steendam et al.
2011	Flow depths in describing the wave overtopping process	van der Meer et al.
2012	Improvement in describing the wave overtopping process	Hughes et al.
2012	Overtopping flow parameters on the inner slope of seadikes	Schüttrumpf et al.
2013	Eurotop revisited part 1 sloping structures	van der Meer
t.b.p.	Statistical characterisation of extreme overtopping wave volumes	Zanuttigh et al.
t.b.p.	New physical insights and design formulae on wave overtopping at sloping and vertical structures	Van der Meer and Bruce
2007	EurOtop wave overtopping of sea defences and related structures: assessment manual	Pullen
2002	Interaction of wave overtopping and clay properties for seadikes	Moller et al.
2009	Guidance on erosion resistance of inner slopes of dikes from three years of testing with the wave overtopping simulator	Van der Meer et al.
2011	Controlling of overtopping flow of embankment with vegetative barrier a flume study	Rasel et al.

- Mean overtopping discharge, q
- Individual and maximum overtopping volumes, V_i or V_{max}
- Overtopping velocities over the crest and landward slope, horizontal and vertical U_{xc} and U_{yc}
- Overtopping flow depth, again measured on crest, h_{xc}
- Overtopping falling distances x_c
- Post-overtopping wave pressures (pulsating or impulsive), P_{qs}, P_imp
- Post-overtopping flow depths, h_{xc}

2-3 Non water retaining objects and discontinuities

Objects that are being positioned in or near the protecting soil embankment but do not contribute to the water retaining function of the embankment are called non water retaining objects (In Dutch: Niet waterkerende objecten). According to the guidelines in V&W [2007a] Non Water Retaining Object(s) (NWO) introduce a disruption in the soil embankment. This can be caused by calamities like failure of the object, but can also happen during normal conditions. One might call these the first order effects of NWO. Some aspects that are important, are the location of the NWO in the cross section, the extent of the disturbance zone and the condition of the object.

In addition to the first order effects, external loads can be introduced by the presence of an NWO. A distinction can be made in several different second order effects. Groundwater flow can be influenced, flow can concentrate at the transition between soil and NWO. It can become an initiation point for erosion as well as an exit point for piping. Also a (local) increase of the phreatic line by for instance a broken pipeline or blockage of water by a object can occur. The presence of a NWO can cause concentration of surface flow round the object which causes erosion and turbulence, which is often seen as the initiation of erosion. Difference in settlement between the object and the surrounding soil can cause empty spaces which then might introduce preferred flow paths and erosion.

A different type of second order effect of the presence of NWO's is the hindered maintenance. Often done by machines which are not able to work their way round NWO's. Bad maintenance may induce erosion near NWO's or cause flow concentration on a different spot. Some research has been done on NWO's in a sandy water defense, mainly in dunes. However this concentrates on the sea side of the dune instead of the landward slope. The results are presented in Boers and Steetzel [2012] which actually is more of an inventory of NWO's in dunes and a plan of approach to find applicable test and design rules. A different report which describes a literature study with respect to NWO's in dunes and a provisional testing method is developed and described, it is based on the literature presented in the report Boers [2009]. A solution given in V&W [2007a] is to positions special water retaining structures in order to compensate for NWO's. In the same document four categories of NWO's are distinguished, vegetation, buildings, pipelines and cables and other NWO's. The complicating factor is that effects caused by the presence of a NWO are highly unpredictable. There isn't very much theoretical background on NWO's.

Another type of discontinuity of the landward slope are transitions. These are transitions between a slope and a horizontal part of the cross section or vise versa and transitions between different types of revetments. In case of a transition from a slope to a berm, the overtopping water is forming a jet that attacks the berm. Research has been done on this subject in the Master Thesis of Astrid Valk Valk [2009], in which is stated that erosion can be prevented when an gradual transition is applied. However not about how gradual it actually has to be in order to prevent this. A description of test observations can be found in Verheij et al. [2012] in which the same jet impact concept is being elaborated and is also stated that a gradual transition can prevent erosion at this spot. The transition between different type of revetments is a another critical discontinuity, especially when the transition is between a hard and a soft revetment.

Discontinuities as a effect of damage, like tracks and small holes can cause an initiation of erosion. However in Van der Meer et al. [2009] is stated that small holes from mice and moles did not initiate damage to the grass cover layer. In the same paper a summary of different tests done with the overtopping simulator can be found. This also appeared out of tests done on the Afsluitdijk, presented in the factual report Infram [2009]. Also test done in Vietnam and in the Netherlands on the Afsluitdijk and Vechtdike describe that erosion often starts at small holes and discontinuities in the slope of the dike, this is shown in Trung et al. [2011b] and Trung et al. [2011a]. For the dutch testing often the wave overtopping simulator is used. A lot of other papers can be found on the test done in the past, the most interesting in this case might be the report about the actual extensive testing on the Afsluitdijk in the period February-March 2009 the results are extensively described and interpreted in Deltares [2010]. During these test, transitions, staircases and fences are also tested with the overtopping simulator.

Most researches done on this subject only considered the surface erosion or external erosion and did not take into account internal erosion as a effect of ground water flow or piping. However one important conclusion that can be drawn from all the research and testing done on dikes is that NWO's and discontinuities are very important when it comes to initiation of erosion. The flow can really get a 'grip' on these points which is needed to get erosion in the first place. Especially the transitions between the soil embankment and the NWO is an important aspect. The resistance of a smooth slope is much larger. The main trend of the above mentioned discontinuities and NWO's is that often

Table 2-5: NWO and Discontinuities

Year	Title	Author(s)
2009	NWO toets duinen Rijnland	Deltares
2012	Voorverkenning NWO's in duinen	Deltares
2013	Programma WTI 2017, onderzoek en ontwikkeling landelijk toetsinstrumentarium	Deltares
2012	SBW-NWO validatie eenvoudige toets	Deltares
2010	Destructive wave overtopping tests on grass covered landward slopes of dikes and transitions to berms	Steendam et al.
2009	Impacts of waterjets on transitions	Valk
2012	Erosion at transitions in landward slopes of dikes due to wave overtopping	Verheij et al.
2012	WP3 Reliability of urban flood defences	Morris et al.
2013	Vulnerability of structural transtions in flood defences erosion of grass covers due to wave overtopping	Pijpers

these appear to be a initiation of erosion. This has also been a subject of extensive testing with the overtopping simulator in for instance Steendam et al. [2012b] and Infram [2009] it can clearly be seen that erosion almost always starts at these discontinuities or NWO's. This can happen already from a mean overtopping discharge of 1 l/s/m. The statement is even made that every overtopping wave leaded to an initiation of erosion and increase of damage. This is however for a bad quality grass cover. At the Afsluitdijk tests also erosion started at the toe of the dike at 30 l/s/m, while on the slope no damage occurred. Also at transitions at 10 l/s/m the first damages occurred. These test are interesting in particular because they were conducted on the Afsluitdijk it self and tested NWO's and discontinuities. In addition to the cumulative overload factor Deltares and van der Meer have proposed to adapt this for flow in the surroundings of obstacles, equation 2-24.

$$\sum \left(\alpha_M U^2 - U_c^2\right) \tag{2-24}$$

The factor α_M is the amplification factor for the increase in flow velocity due to the presence of discontinuities. For relative large obstacles and transitions it varies between 1 and 1.5. For grass to obstacles there is no amplification factor however a estimation has been done at 1.25 Steendam et al. [2012a].

In Hoffmans [2012] very clearly the strength of grass and the plunging jet principle are elaborated. The book is focused on erosion of soil by turbulence, which is interesting because the turbulence around NWO's and transition is exactly what is expected to cause erosion, the flow just transports the eroded soil. In Morris [2012] a description of different type of transitions is given and their failure modes. These are not necessary on the landward slope of the dike, it however gives a good indication for the different transition types.

2-4 Strength of grass on clay

The current limits according to Pullen et al. [2007] for crest and landward slope can be found in table 2-2, a good grass cover will provide protections against mean discharges of 1-10 $l/s/m$. Extensive testing has been done on the strength of grass on landward slopes of dikes. Out of these test it often appeared that the strength of grass was sufficient to allow larger amounts than above, however it also appeared that due to failure at certain critical spots it is not. The focus of some of the tests should have been on these critical points. The overtop manual is not a critical review on the strength but more a design purpose document, from that point of view it might be not a bad decision to take the 1-10 $l/s/m$. In Kruse [2013] a description of the strength of clay on river dikes is given, also for the landward slope. In this report statements about the thickness of the clay layer, the critical points and the allowable overtopping discharge (for good grass on clay at least 10 $l/s/m$). In principle clay layers of 0.5 m are good enough. If the thickness is increased to 1 m the extra thickness can prevent erosion after flushing of the grass. These findings are mainly based on tests executed with the wave overtopping simulator. So to see how resistant grass really is, the large scale testing with overtopping simulators should be considered. The strength of the grass is usually expressed in how much water it can resist as in an average $l/s/m$. In addition to this the quality of grass is also expressed as very poor, poor average and good. This quality is a function of the amount of roots at a certain depth V&W [2007a]. The testing has been done at a few locations in the Netherlands, but also with the stationary Colorado wave overtopping machine and testing in Vietnam. Because the testing is done on a lot of locations and by different people also a lot of literature is available on the subject. First of all, the tests done in the Netherlands up to 2009 with the Dutch wave overtopping simulator, which is a movable simulator that is able to simulate overtopping volumes of 5.5 m^3/m. The simulator was developed in 2006, tests have been performed on different sections of real dikes on a 7 locations in the Netherlands and 1 in Belgium (Tielrode). The tests have all been carried out at the end of the winter when the grass is in it's worst condition. The results of the testing of these locations gave a lot of different results Van der Meer et al. [2009], the tested slopes never failed by erosion due to overtopping for 30 $l/s/m$ or less. Only one section failed at 50 $l/s/m$ and some at 75 $l/s/m$, and different sections did not fail at all. A summary of the tests done on the locations in the Netherlands and Belgium can be found in van der Meer et al. [2011]. It appears that the Vechtdijk location was very weak, already at 10 $l/s/m$ uncontrollable failure mechanisms occurred so the tests had to be stopped. From this it can already be stated that the variability in the strength of grass is large. It appears that grass can handle more then currently is used.

The wave overtopping simulator that is being used in Vietnam is in principle the same machine as the one in the Netherlands, it is also a mobile device, so that actual dike sections can be tested. The maximum wave volume that can be simulated is also the same as the simulator used in the Netherlands, 5.5 m^3/m. In Vietnam three different dikes have been tested. At the first location three different section along a dike stretch were tested. The construction and layout was quite similar however the test results varied from 20 to 70 $l/s/m$. Which was not expected on forehand. On the second dike the resistance of vertiver grass was up to 120 $l/s/m$. The initiation of erosion showed more round small threes, small holes and transitions from slope to berm than on other points at the slope Trung et al. [2011b]. Also the relationship between the front velocity and the volume per wave was analyzed, the relation was a bit different than found by van der Meer et al. [2011] it is described in Trung et al. [2011a]. In the same paper also the cumulative overload factor is considered. The method it self is only calibrated which resulted in different values for the criteria initial damage and various damage. The difference between the results of the Vechtdike and the results described in the paper could be explained from the fact that the Vechtdike consisted of a sandy core while the Thai Binh dike had a solid clay layer with a good grass cover. A more elaborated description of the tests done in Vietnam can be found in Trung et al. [2012a]. In addition a summary of both tests in the Netherlands and in Vietnam can be found in Trung et al. [2012b].

In Colorado also a wave overtopping machine has been build and developed. The design and testing is has been described in Van der Meer et al. [2011]. In comparison to the Dutch wave overtopping simulator there are a couple of differences with the Colorado wave overtopping simulator. First of all the maximum wave volume of the Colorado simulator is larger with 17 m^3/m versus 6 m^3/m for Dutch simulator. So they are able to simulate much larger waves. However due to the lay-out of the machine, the also can still do the same simulations as the Dutch overtopping simulator. The amounts that can be tested up to now amounts of approximately 350 $l/s/m$. Next to this the overtopping simulator in the US is a permanent fixed in place simulator while the dutch overtopping simulator can be moved to practically every location. So testing can never be done on actual dikes but are always done on the same site. In order to be able to do simulations on actual dikes, sections of dikes can be cut out, packed and transported in boxes to the site of the overtopping simulator.

Another way of testing is by building the layered dike landward slope. In order to do so trays filled with a compressed clay layer with on top of that a grass layer which is put in a glass house on site. So the grass can be cultivated for a certain amount of time in order to gain some strength. However because of the controlled environment in which the grass and clay boxes and crates are cultivated, the strength might differ from what can be found on real dikes. It might be stronger so that the test results give a higher allowable amount of water. Next to this it can be found. This could be an explanation for the differences in extreme overtopping volumes that the grass can withstand at the Colorado wave overtopping simulator. From the first set of tests performed some remarkable results showed. Bermuda grass could withstand an average overtopping discharge of 370 $l/s/m$ without showing any damage. However when the same grass went dormant during winter the maximum overtopping discharge was reduced to 186 $l/s/m$. So if even one time winter condition reduces the strength of the grass by a factor two, it seems of great importance to simulate the realistic conditions. The first set of testing is described in Thornton et al. [2011]. The testing was not focused on the weakest links within the landward slope of the dike. However they did observe initial erosion at the transition from slope to horizontal. What is also very interesting to see is that bare clay already had severe erosion at 19 $l/s/m$. The difference between bare clay or clay with a grass cover layer is very big. On the one hand in seems not very attractive to use results of this overtopping simulator because of the above mentioned reasons. On the other hand comparisons of different types of grass or other revetment types might be interesting. Not because of the absolute strength but because of the relative strength. For the testing of special type of revetments (which currently is conducted according to Van der Meer) it might be a suitable facility.

In addition to normal grass a relative new development is reinforced grass. At the Colorado wave overtopping simulator also a small set of tests has been done with these mats, two different types where tested, a open weave turf reinforcement mat and a high performance turf reinforcement mat. Both were gone dormant so they were severely damaged. The first could resist 140 $l/s/m$ while the second could resist a lot more the maximum 370 $l/s/m$ Thornton et al. [2012]. Already the difference in strength is more than a factor two between the different types of turf reinforcements. Some other literature is available on this subject but is not studied so far.

Another development is the grass tension testing device. At which the actual strength of the grass is being measured by applying tension to a section of grass of 15 x 15 cm. This is a relative new device and is described in Steendam et al. [2012a].

2-5 Research on the Afsluitdijk

Because of the fact that the Afsluitdijk is the biggest dike in the Netherlands, research has been done on the strength of the dike. Partly as a part of the SBW program. A different number of investigations have been done. As a starter, the strength against breaching Visser [2002] which is described in 2-6. The wave overtopping test executed on the Afsluitdijk Infram [2009], which are extensively covered in other parts of this literature review. For the Afsluitdijk commissioned by Rijkswaterstaat (RWS) Deltares has calculated the hydraulic boundary conditions that need to be taken into account for the design of the renewed Afsluitdijk Deltares [2013]. A wave run-up investigation has been done, however not very relevant for this research. The location of the monument was part of an interdisciplinary study, this was more of an integrated design solution. Also a probabilistic safety analyses has been done on the Afsluitdijk in the past.

Table 2-6: Grass

Year	Title	Author(s)
1998	TR 12 Technisch Rapport Erosiebestendigheid van grasland als dijkbekleding	TAW
2005	Wave overtopping and grass cover layer failure on the inner slope of dikes	Young
2006	Erosiebestendigheid van grasbekleding tijdens golfoverslag	van den Bos
2007	Reinforced grass on inner dike slopes	Garcia
2009	Failure of grass cover layers at seaward and shoreward dike slopes	EroGrass
2010	Instability of grass cover caused by wave overtopping	Hoffmans
2010	Criteria voor toepassen van bekledingen op waterkeringen hulpmiddel voor ontwikkeling van innovatieve dijkbekledingen	Witteveen+Bos
2011	Erosion resistance of HPTRM strengthened levee from combined wave and surge overtopping	Pan et al.
2012	SBW Wave overtopping and grass cover strength: Predictions of prototype tests	Deltares
2012	Destructive wave overtopping and wave run-up tests on grass covered slopes of real dikes	Steendam
2011	Testing levee slope resiliency at the new Colorado state university wave overtopping test facility	Thornton et al.
2011	Design and operation of the US wave overtopping simulator	van der Meer et al.
2012	Full-Scale testing of levee resiliency during wave overtopping	Thornton
2007	Reinforced grass revetment	Garcia
2012	High performance turf reinforcement mat strengthened levee under combined wave and storm surge turbulent overtopping conditions	Amini
2010	Wave overtopping simulator test at Vietnam	Trung et al.
2010	Wave overtopping tests at Vietnam	Trung et al.
2011	Wave overtopping resistance of grassed dike slopes in Vietnam	Trung et al.
2011	Wave overtopping resistance of grassed slopes in Vietnam	Trung et al.
2012	Wave overtopping simulator tests on sea dikes in Vietnam	Trung et al.
2012	Strength of the landward slopes of sea dikes in Vietnam	Verhagen et al.

2-6 Residual strength and breaching

Because residual strength and breaching are still subjects with insufficient knowledge and many uncertainties to really take into account as a part of the strength, the subject has been looked into to see what has been investigated. Initially it will not be taken into account but there is some interesting literature available.

A description of the growth of a breach can be found in Verheij [2002]. This is thus after a breach has already occurred and has a certain initial width an depth. Knoeff and Verheij [2003] Maybe the most interesting on the growth of a breach after a break through is Visser [2002]. This because the model BRES is used on the Afsluitdijk. The conclusion is drawn that the breaches will only be shallow, but can be wide. Because of the dam of boulder clay the Afsluitdijk has a unique strength to resist the growth of a breach after a breakthrough. The expectation is that because of the above and the relative large surface of the IJsselmeer the high water levels will not occur at the IJsselmeer. There is also the SBW-reststerkte program in which more research is being done Bretler et al. [2010]. This is being described in 2-7.

Table 2-7: Afsluitdijk

Year	Title	Author(s)
1987	Golfoverslag Afsluitdijk	WL-Delft
1981a	Golfoploop onderzoek aan de Afsluitdijk	Bruins
1981b	Golfoploop onderzoek aan de Afsluitdijk	Bruins
2009	Factual Report: Overslagproeven en afschuifproef Afsluitdijk	Infram
2010	SBW Golfoverslag en Sterkte grasbekleding: fase 3D	Deltares
2012	Afsluitdijk project the monument location	Liu et al.
2013	Hydraulische Randvoorwaarden voor het ontwerp van de versterking van de Afsluitdijk	Groeneweg et al.
1988	Probablistische veiligheidsbeschouwing Afsluitdijk	Yap

2-7 Research programs

In addition to the above described topics which all have a interference with overtopping and strength of the landward slope there are some research programs which also (partly) cover this subject. These research programs are very useful because they tackle multiple subjects at once. Some relevant research programs will be described in short.

ComCoast
ComCoast stands for 'Combined functions in Coastal defence' zones. This program tries to find an answer to future problems in climate change effects and subsidences of coastal drainage areas, as a consequence the coastal defenses will be attacked more vigorously in the future. The program focuses on three main aspects: Wave attack reduction, limit wave overtopping and strengthening of the defenses The latter is the most relevant for the subject of the thesis. This is mostly reported in the WP3 reports. The aim is an adequate design criteria and innovative design approaches of heavily overtopped sea defense structures. The program did run from April 1, 2004 to December 31, 2007.

The subject of the WP3 reports vary on a broad spectrum, from a large inventory to safety analysis to reinforced grass to conceptual models. All these results are presented in 18 different reports. With report 1 a inventory, with as main conclusion has to continually update the inventory of data and report 18 a very specific report on the placement of smart grass in which additional research appears to be required. The testing is focused on new concepts, like crest drainage dikes and reinforced grass. This crest drainage dike looks like a nice concept, however a structure within the crest of the dike always introduces additional transitions which can give initiation of erosion. One of the reports is Van Gerven and Akkerman [2005]. The URL enlisted with this reference gives a direct link to all the ComCoast reports.

SBW
SBW stands for 'Sterkte en Belastingen Waterkeren' (Strength and loads on water defences). It is a program of Rijkswaterstaat with as main purpose filling gaps in the current knowledge to get a clearer view on the safety of primary defenses against flooding. The conducted research is a cooperation between different companies. There are three programs that are interesting for the subject of the thesis.
During the research on the SBW projects it appeared that the SBW program is combined together with Voorschriften Toetsen op Veiligheid (VTV) in the Wettelijk Toetsinstumentarium (WTI) 2017. This has been done in order to improve the coordination between both programs and the translation form research results into guidelines for testing. There is a lot of overlap between the different articles and the SBW reports. This is due to the fact that a lot of articles are written based on the same research as for the SBW.

Table 2-8: ComCoast

2006	Van dijkversterking naar dijkverlaging	Oedekerk, M.
2005	Sate of the art invertory	Van Gerven, K.A.J. and Akkerman, G.J.
2005	Development of alternative Overtopping-Resistant sea defences, proposal for concepts	Van Gerven, K.A.J. and Van der Meer, J.W. and Heerveld, M.A. and Akkerman, G.J.
2005	Approach to the innovative design of an overtopping resistant dike	Steenbrink, R and Pwa, S.T. and Busneili, M.M. and Karelse, M.K.
2005	Overtopping resistant dike, sandy dike	ComCoast
2005	Innovative concept for an overtopping dike	Nieuwehuis, O.E.
2005	Safety analysis of the ComCoast concept	Ter Horst, W.L.A.
2005	Development of alternative overtopping-Resistant sea defences(Elaboration of smart grass reinforcement concept	van Gerwen, K.A.j. and van der Meer, J.W. and van Heereveld, M.A. and Akkerman, G.J.
2005	Sandy dike	Koopal, A.A. and Onderwater, M.
2005	Innovative concept overtopping dike: Crest drainage dike	Nieuwenhuis, O.E.
2006	Golfoverslag en sterkte binnentalud bij dijken	Galema, A.A. and De Jong, R.H. and Prius, K.W. and Wisse, A.
2005	Conceptual model for reinforced grass on inner dike slopes	ComCoast
2007	Wave overtopping erosion tests at Groningen sea dike	Akkerman, G.J. and van Gerven, K.A.J. and Schaap, H.A.
2007	Hydraulic model tests of an innovative dike crest design	Kortenhaus, A. and Bolinger, K. and Dassayanake, D.
2007	Placement of smart grass at test sections Groningen sea dike	van Gerven, K.A.J. and Akkerman, G.J.

SBW-Wave overtopping on grass-covered dikes
One of the projects is SBW-Wave overtopping on grass-covered dikes. The main goal of this project is to gain a realistic view on the total failure mechanism on the landward side of dikes during wave overtopping. A lot of research has been done in order to gain a good insight in the failure of grass covered dikes. Extensive testing with the Wave overtopping simulator has been done to see were the weak spots are and how failure occurs and develops. These researches have resulted in a lot of different reports. It consists of 10 different phases, 1 to 7 are about testing and model testing and evaluating those tests. The phases are named after there location. 8-10 are the end results or final reports of the project. It might be clear that the most important phase for the thesis is phase 3, it is focused on the Afsluitdijk However the other reports are also very relevant. The first seven phases are:

- 1: Delfzijl
- 2: Boonweg
- 3: Afsluitdijk
- 4: Petten

- 5: Zeeland
- 6: IJkdijk
- 7: Golfoverslagbak Petten

The reports of the first five phases are available in hard copy and partly digital, however phase 6 and 7 are not. The final result should be an actualization of the rules in the current VTV and a Technical report of the findings: 'Toetsen van grasbekleding op dijken'. Based on this report and for taking into account the new knowledge for assessing grass on dikes the Handreiking toetsen grasbekleding op dijken Van der Meer et al. [2012] has been composed in which a lot of new knowledge on grass layers on dikes can be found. This report can also be seen as a summary of the different phases. In additions to this new distributions and overtopping volumes, sod openness and root density and grass modeling is reported in the predictions report of this SBW project Steendam et al. [2012a]. Topics under investigation are wave overtopping, erosion and grass tension test. The latter is a newly developed device in order to get a indication for the strength of grass.

SBW-Reststerkte
This part of the SBW program is on the residual strength of dikes after initial failure. It is the timespan between initial damage to total breaching of a dike. The target is to quantify the process after initial damage to occurrence of a breach. In addition to that develop a method to test and set up criteria on which the testing can be based. The total project of SBW-Reststerkte consist of seven subproject:

- reststerkte van keileem (residual strength of boulder clay)
- reststerkte van een dijk met steenzetting op kleilaag en zandkern (residual strength of a dike with a stone revetment on layer of clay and a sandy core)
- reststerkte van een dijk met asfalt op een zandkern (residual strength of a dike with asphalt on a sandy core)
- reststerkte van een dijk met gras langs een rivier (residual strength of a river dike with grass)
- rol van micro-instabiliteit bij reststerkte (The influence of micro-instability residual strength)
- reststerkte van de kruin en het binnentalud van gras na schade door golfoverslag (residual strength of the crest and landward slope after initial damage by wave overtopping)
- communicatietraject en samenvattend verslag (Communication process and summarizing report)

The inventory of the project and an extensive description can be found in Bretler et al. [2010] the first other report on the residual strength of a dike with a stone revetment on a layer of clay is described in Klein Breteler and Wolters [2011]. Not al the SBW-Reststerkte reports are gathered jet, because initially the residual strength will not be taken into account.

SBW-NWO
This SBW research project is still ongoing and therefore in concept phase. However, a report is available that describes the simple testing of NWO's and the description of modeling with the new program DAM with a NWO module. This is a test mainly on stability near NWO's and not so much on the erosion by wave overtopping round NWO's. It is therefore a relevant research, to keep in mind van der Kolk [2012]. The project plan for the new WTI 2017 is being described in van den Ham and van der Kolk [2013]. The NWO's will be investigated under cluster indirect failure mechanisms which is a part of WTI/SBW 2017.

2-8 Spillways

Spillways can often be seen at reservoirs, usually these are concrete streamlined structures in order to release large amounts of water in case the reservoir capacity is reached. However, due to the increasing incoming discharge as a result of climate change the capacity of the reservoir and the discharge capacity of the spillway are insufficient. In order to deal with this lack of capacity different solutions can be applied:

- Increase the storage capacity of the reservoir by raising the embankments
- Increase the capacity of the existing spillway
- Create an emergency spillways

Especially the last option is of great interest in relation to overtopping resistance. However, the dominant process is often surging overflow which doesn't have the same larger velocities and repeating loading character as for wave overtopping. The solutions could be interesting though, because the slopes still have to cope with large amounts of water. For an emergency spillway, one of the options is to adapt a part of the earthen embankment dam to be able to handle large amounts of discharging water. However, these earthen embankments are able for events with a low probability of occurrence (which is defined as 1/100 year for spillways) to withstand large amounts of water. There are several ways to increase the resistance of the earthen embankment:

- Vegetation
- High Performance Turf Reinforcement Mats (HPTRM)
- Geosynthetics
- Articulated Concrete Block system (ACB)
- Roller Compacted Concrete (RCC)

A large number of presentations has been given by the association of state dam safety officials ASDSO [2013] on the National dam safety program - Technical seminar No. 20 at which a lot of these options are clarified. Including the failure of some of these protection and what went wrong. For the use of geosynthetics a good overview is given in Haselsteiner et al. [2010] which gives different solutions for the use of geosynthetics. A large variety in the use of these geosynthetics is given, smooth and ripled slopes, and slopes divided in different segments. Each with it's own advantages and disadvantages. Especially on the use of stepped spillways a lot of research has been done. These stepped spillways are used in order to make the energy dissipate by allowing air to be caught in the water or otherwise said to introduce turbulence. In this way the load on the transition becomes much less than with a smooth slope. This has been described in Chanson [2009], together with a history and a description of two types of weirs, minimum energy loss weirs and concrete Macro Roughness elements. Often block mats of some kind are used to realize this. A different option is to choose for RCC as is described in Hunt et al. [2008]. A lot of detail on the hydraulic design of such stepped spillways and energy dissipaters can be found in Gonzalez and Chanson [2007]. In Broich [2002] the determination of the initial conditions for dam erosion is being described including some velocity limitations for different type of revetments. Also an integration in the probabilistic design is being described. The maximum allowable flow velocities for overflow can be seen in figure 2-5 Broich [2002]. This is thus for continues overflow and not for pulsating overtopping. The duration has an influence on almost every revetment type accept for the concrete blocks. Which do give the largest resistance of up to 8 m/s.

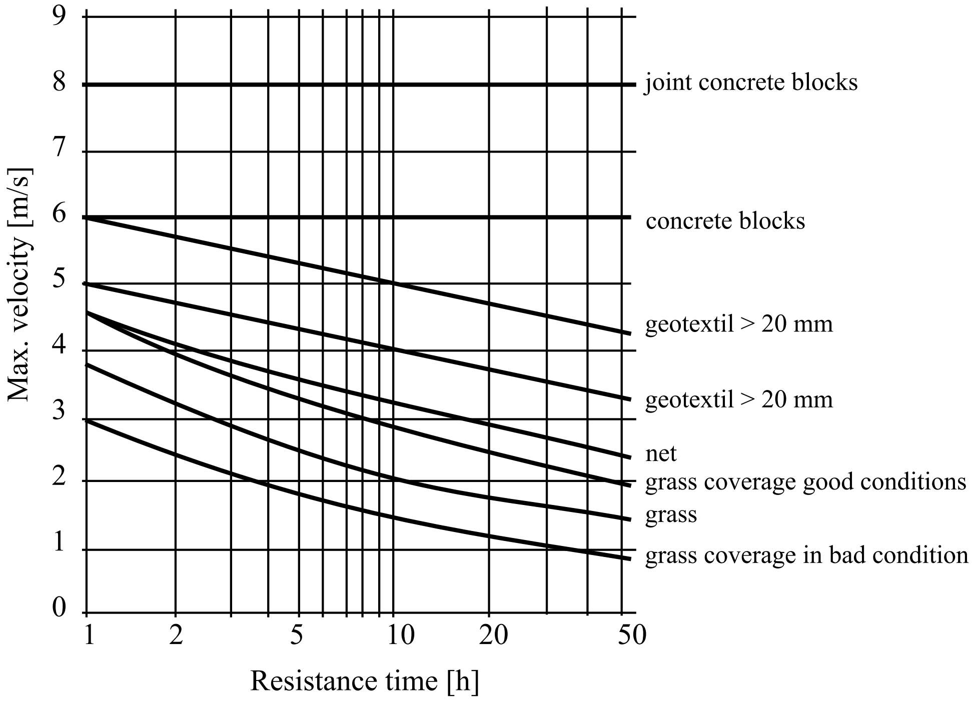

Figure 2-5: Maximum overtopping velocity versus resistance time (CIRIA experiments)

2-9 Interviews

As a part of the literature study an interview has being held with two persons on their expertise on the subject of the Afsluitdijk and wave overtopping. This is next to the conversation held with employees of Witteveen+Bos (W+B) and the Delft University of Technology (TU Delft). The goal of these interviews is to gain more insight in the research done in the past and their view on the project and aim of the thesis. Both the interviews resulted in adaptations in the research approach and additional literature that can be used. Small reports have been written (in Dutch) and can be seen in Appendix A. The interviews certainly helped to get a quick insight in the subject. The persons interviewed so far are:

- Eric Regeling (Project leader Afsluitdijk at RWS) report in section A-1
- Jentsje van der Meer (Owner Van der Meer Consulting B.V.) report in section A-2

2-10 Concluding remarks

The literature review in this chapter describes a lot of methods, formulas, theories and tests. The main question is, what can be concluded from this literature review with concern to wave overtopping resilience of the Afsluitdijk.

The theory seems to describe the loading due to overtopping quite well, an important distinction between infiltration and erosion caused by wave overtopping should be made. The current design methods are meant to heighten the dike to a level such that overtopping discharges are prevented or limited to a minimal amount. Research has been done in great extend to the loads caused by overtopping waves: the distribution between infiltration and overtopping of water, overtopping as a function of the distance behind the crest and the loads on the landward slope caused by wave overtopping are subjects of investigation. The investigations to loads caused by wave overtopping is most important for the research in this thesis. The cumulative overload factor seems to be a promising parameter, it couples the load and strength based on erosion due to excess of shear stress. Currently this factor is mainly based on testing done with the wave overtopping simulator.

From testing with the overtopping simulator also a relation between the overtopping volume per wave and the flow velocity per wave at the crest can be found. From testing it also appeared that NWO's and discontinuities are the critical points concerning overtopping. During test the transitions and objects are places at which initiation of erosion occurs. Some research has been done on the subject of transitions and NWO's, an adaptation of the overload factor with an increase in the load is proposed. Almost all research and testing has been done on grass, with the dutch overtopping simulator, the overtopping simulator in Vietnam and the one in Colorado. The last one most likely is not useful due to the fact that it is stationary and because of the controlled environment in which the grass and clay boxes are cultivated. This does not give realistic test results, for testing in the future on for other revetments than grass it might be useful.

Testing with the overtopping simulator has also been done on the Afsluitdijk, which was capable of resisting up to 75 $l/s/m$. There was damage, however the clay layer wasn't eroded away completely. Also here the transitions and NWO's appeared to be the weak spots. Research has been done on the residual strength and breaching of the Afsluitdijk. The residual strength, strength after initial failure appears to be insufficiently understood to be taken into account in order to prove the strength of the dike. The SBW and ComCoast projects are research project for which a lot of literature study and testing has been done for various dikes in the Netherlands. The analogy with spillways doesn't seem promising for the because it is a steady state process while overtopping is an intermittent process. A lot of solutions concerning the reinforcements of earthen dams can be found in the area of spillways. The strength or resistance against wave overtopping is the largest uncertainty. The least research has been done on this subject and measures that could increase the strength against large amounts of wave overtopping are nowhere to be found.

Table 2-9: Spillways

Year	Title	Author
2013	28 Presentations of National Dam Safety Program Technical Seminar No. 20	
2012	Evaluation of the Strucutre of levee Transition on Wave Run-Up and Overtopping by Physical modelling	Oaks et al.
2012	Influence of Three Levee-Strengthening Systems on Overtopping Hydraulic Parameters and Hydraulic Equivalency Analysis between Steady and Intermittent Overtopping	Pan et al.
2012	Air-water flow properties and energy dissipation on stepped spillways: a physical study of several pooled stepped configurations	Felder et al.
2012	Roller-compacted concrete dams: a brief history and their advantages	Warren
2012	Mitigation of flooding by improved dams and dykes	Heerten
2012	Performance of HPTRM strengthened levee in full-scale overtopping tests	Amini et al.
2011	Controlling of overtopping flow of embankment with vegetative barrier: A flume study	Rasel et al.
2011	Overtopping and overflow of flood protection embankments - Risk reduction of embankment dam failure by the use of geosynthetics	Werth et al.
2010	WINDAM B Earthen Embankment Overtopping analysis software	Visser et al.
2010	Application of geosynthetics for overtopping loads at flood protection dikes	Haselsteiner et al.
2009	Design of earth dams allowing temporary overtopping based on hydraulic failure experiments and flood analysis	Matsushima et al.
2009	Embankment overflow protection systems and earth dam spillways	Chanson
2008	RCC stepped spillway for Renwick Dam-A Partnership in Research and Design	Hunt et al.
2008	Overflow protection systems of flood embankments with geosynthetics	Haselsteiner et al.
2008	Levee overtopping design guidance what we know and what we need	Hughes
2007	Hydraulic design of stepped spillways and downstream energy dissipators for embankment dams	Gonzalez and Chanson
2002	Determination of initial conditions for dam erosion due to overtopping and possible integration into a probabilistic design concept	Brioch
2001	Design manual for articulated block systems	Dunlap
1989	Mechanics of Overflow Erosion on Embankments II: Hydraulic and Design considerations	George et al.
1989	Mechanics of Overflow Erosion on Embankments I: Research Activities	George et al.

Table 2-10: Additional topics

Year	Title	Author(s)
2012	Verkenning innovatieve dijken in het Waddengebied	Alterra/Jantsje
2010	Sliding stability of landward slope clay cover layers of dikes subject to wave overtopping	Hoven
1999	Golfbelasting op kruinmuren op dijken	Deiters
2007	Wave overtopping aspects of the crest drainage dike	Steeg
2012	Wave overtopping reduction by seadike crown walls in Vietnam	Quang
1999	Flooding classification and simulation	de Vries
2007	Simulating floods	Alkema
2012	Simulation of wave overtopping of maritime structures in a numerical wave flume	Tiago
2012	Notitie onderzoek Asfaltdijkbekledingen 2012-2015	STOWA

Chapter 3

Study area

This chapter gives a description of the area on which the research is conducted. The location, lay-out and cross sections are described together with the surroundings of the Afsluitdijk. The theoretical framework of the thesis is treated here as well.

3-1 History

The *Afsluitdijk* or internationally sometimes called the '*Zuiderzee enclosure dam*' was closed in 1932, it was and still is state of the art hydraulic engineering. It also has become an attraction for both the Dutch and foreign tourists. The plans for a closure of the '*Zuiderzee*' did exist for a very long time. Already in the 17th century, the to most readers known, *Hendrik Stevin* had an idea to close the *Zuiderzee* in order to protect the Netherlands from flooding and prevent siltation of agricultural grounds. However at that time the technology wasn't ready for such a closure.

It was all the way up to 1880 that several people had different plans for reclamations in the Zuiderzee. Following this trend in 1886 the '*Zuiderzee Vereniging*' was founded. It was an initiative from several influential individuals and representatives from provinces, municipalities and water boards. *ir. Cornelis Lely* was appointed advisor of this association. He was given the task to perform a technical and financial research of the closure. This was and still would be a hard task. Especially when one bears in mind that such a closure was never done before.

In 1891, the same year as Lely finished his plans for the reclamation of the Zuiderzee, he also did become Minister of public works. It was only until 1913 that the decision was made that the plans where going to be executed. However, one year later World War I started and the government had other problems to deal with. In combination with the 1916 '*Zuiderzee flood*' and the importance of the supply of grain for the Netherlands, Lely's plans gained new attention. In 1920 the start with the actual calculations was made by the committee *Lorentz*. It took them up to 1926 to finish. The start with the actual construction of the Afsluitdijk was made in 1927 and it took five years to complete. In figure 3-1[1] the construction of the boulder clay dam is depicted.

From then on the Afsluitdijk was a symbol of Dutch hydraulic engineering tradition and made the Dutch world famous when it comes to engineering, and it still does.

[1] www.nieuwlanderfgoed.nl/beeld/Pers/Beeldbank/3%20afsluitdijk.jpg

Figure 3-1: Construction Afsluitdijk

3-2 Location

The Afsluitdijk is located between Noord-Holland (Den Oever) and Friesland (Zürich) it is the barrier between the 'Waddenzee' and the 'IJsselmeer' 3-2[2]. The location of the Afsluitdijk has been chosen in such a way that the tidal flow velocities during construction where as low as possible. In the system there was a place where the tidal flow before the closure was minimal, the location of the Afsluitdijk has been chosen near that position, only a little bit more seawards Wang et al. [2009]. The construction of the Afsluitdijk has resulted in a sediment importing Waddenzee. This is due to the fact that the tidal prism has been reduced as an effect of the closure. One of the unique aspects of the dike is that on the outside there is a salt water tidal basin and on the inside there is a big fresh water lake the Waddenzee and the IJsselmeer, respectively. The latter is considered as an important strategic fresh water basin for the Netherlands. It slowly desalinated after the construction of the dike. The dimensions of the Afsluitdijk are of huge proportions with it's length of about 32 km, an average width of about 90 m at the waterline and a crest height of Normaal Amsterdams Peil (NAP)+ 7.50 to 7.80 m.

3-3 Lay out

The Afsluitdijk consist of a few different sections. These are schematically depicted in figure 3-3[3] IenM [2011]. To start with the connection of the dike to the province of 'Noord-Holland' at Den Oever. At this location there are ship locks and discharge sluices. The locks as a whole are named the 'Stevinlocks', in this report Den Oever lock complex. Further along the dike the monument can be found, at which the dike has a different lay-out. This is the location at which the closure of the dike was completed. The old working harbor which was in use during the construction of the dike still has a special position within the dike. This includes a few houses. It is called 'Breezanddijk'. At the north side of the dike there is a bend in the dike after which another set of locks is located. This bend is the result of that one of the locations where the discharge sluices had to be placed appeared

[2] source: maps.google.com
[3] Translated version of the original

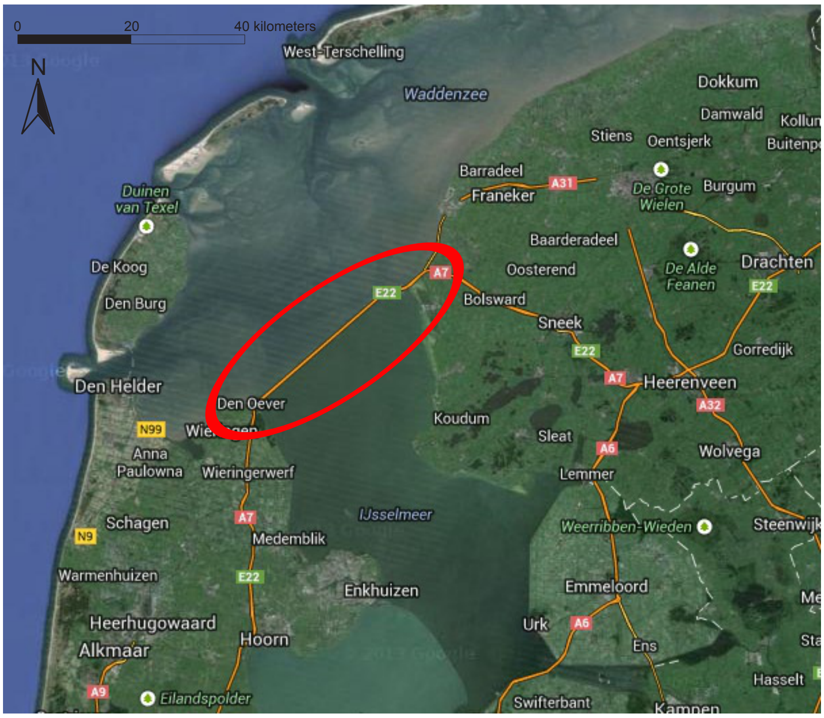

Figure 3-2: Location Afsluitdijk

unsuitable during construction. Therefore the decision was made to adapt the alignment towards the north. This location is called 'Kornwerderzand', the locks are named 'Lorentzsluizen', in this report Kornwerderzand lock complex. This complex also consists of a ship lock and a set of discharge sluices. After this set of locks a small stretch of dike follows which is connected to the province of Friesland at Zürich.

3-4 Cross section

The Afsluitdijk has a few different cross sections along its course. The cross section that can be found for most dike sections is depicted in figure 3-4[4]. Deviating cross sections can be found at the Den Oever lock complex, at the location of the monument, the section Breezanddijk and the segment in between Kornwerderzand lock complex and the end of the dike at Zürich. In table 3-1 the division of the Afsluitdijk in different sections can be found. These will be used later to describe the governing section. The division is based on the layer (in Dutch: 'legger') of the Afsluitdijk. Each section is defined from a certain distance from Den Oever (so 0.00 km is at Den Oever) to another distance. Also the point at which the output of the Hydra-k model is given can be found in the last column of the table. The definitions of these points can be found in Deltares [2013].

The characteristics of the dike are: the angle of the outer slope varies between 1 : 3.5 and 1 : 4.5. The landward slope has an angle varying between 1 : 2.2 and 1 : 2.7. The crest of the dike has a width of 2 m and the height varies between NAP+7.5 and 7.8 m. The subsoil on which the dike is constructed is NAP-3.7 m on average, but varies as well. At this level the dike has a width of approximately 140 meter. As a part of the foreshore of the Afsluitdijk the bottom of the Waddenzee is important. This is because the foreshore partly determines what kind of distribution waves at the toe of the dike will

[4]Large version can be found in Appendix B, figure B-1.

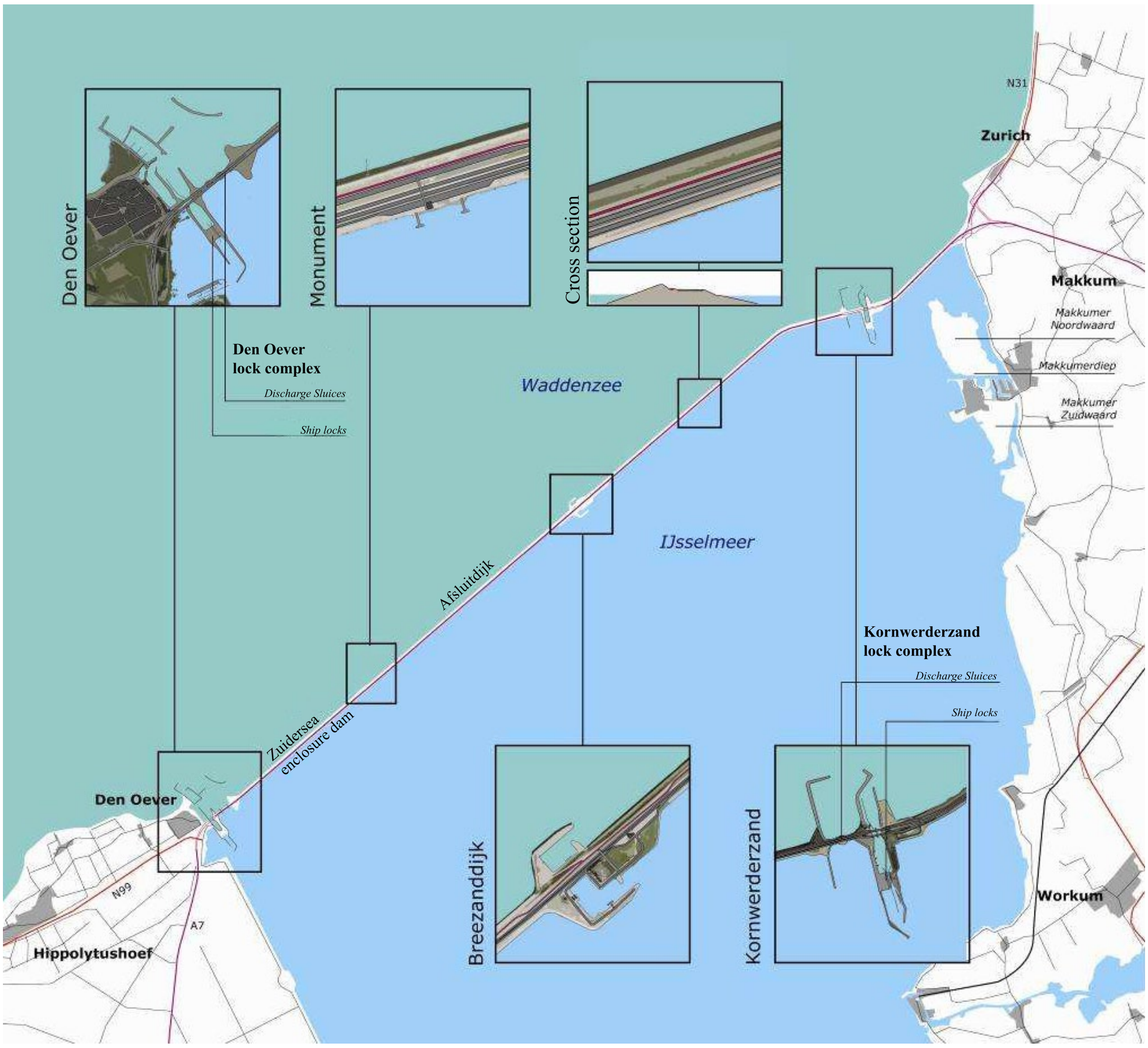

Figure 3-3: Lay out

have. The foreshore near the Afsluitdijk is variable, this is along the dike and also in time. In front of the Afsluitdijk various gullies are present which move over time.

This clearly can be seen in the 'vakloding' data of Rijkswaterstaat (RWS)[5]. This means that there is a large uncertainty in the foreshore slope and depth. The data is plotted for the years 1971 and 2009 in figures 3-5 and 3-6, respectively. The straight line on the low right of the figure represent the location of the dike. The axis are given in 'Rijksdriehoekscoordinaten' (RD-Coordinates), which gives the relative position from the center of the system at Amersfoort, in meters. The depth variation is given by the different colors which can be seen on the scale on the right of the figure, these are in meters as well. The figures give a good indications on how much the conditions per dike section can vary over a few decades. Some of the gullies have become less deep while new gullies are present and some existing have moved. So for a lifetime of 50 years it most likely will also vary a lot. Close to the Afsluitdijk various deep gullies are present. Which can mean that a relative steep foreshore is present at some locations along the course of the dike. Depending on whether or not there is a shallow

[5] opendap.deltares.nl/thredds/catalog/opendap/rijkswaterstaat/vaklodingen/catalog.html (KB124_1918, KB124_2120, KB125_1918, KB125_2120)

Table 3-1: Dike sections based on the 'layer' of the Afsluitdijk (start 0.00 km at Den Oever)

section	from (km)	to (km)	description
1	0.30	0.90	outer harbor Den Oever (west)
2	0.91	1.47	outer harbor Den Oever (east)
3	1.47	4.60	between bridge and discharge locks Den Oever
4	4.60	2.07	Den Oever lock complex
5	2.07	2.50	connection to structures
6a	2.50	4.40	Afsluitdijk
6b	4.40	6.90	Afsluitdijk
7	6.90	7.60	monument
8a	7.60	11.00	Afsluitdijk
8b	11.00	15.05	Afsluitdijk
9	15.05	17.53	Breezanddijk
10a	17.53	19.50	Afsluitdijk
10b	19.50	21.03	Afsluitdijk
11a	21.03	23.90	Afsluitdijk
11b	23.90	25.90	Afsluitdijk
12	25.90	26.22	Kornwerderzand lock complex (west)
13	26.22	26.49	Kornwerderzand lock complex
14	26.49	26.70	Kornwerderzand lock complex (east)
15	26.70	27.30	outer harbor Kornwerderzand (west)
16	27.30	27.90	outer harbor Kornwerderzand (east)
17	27.90	31.92	Kornwerderzand

foreshore the waves do have a different distribution or otherwise said spectrum. The dike actually consist of two parts, the Waddenzee side at which the actual closure dam is constructed. This dam consist of boulder clay, the construction height was NAP+3.5 m. This dam is 42 m wide at the bottom. On top of the core is a layer of boulder clay and a revetment of basalt. The combined thickness is approximately 1 meter. The revetment of the crest and the landward slope is also a layer of boulder clay, 0.75 meter with on top of that a layer of clay varying between 0.25 and 0.70 meter with a grass cover. Below the crest and the inner berm is a sandy core, the second part. The level of the inner berm is NAP+3.5 m on which the high way A7 of 2x2 lanes is constructed. The cycling lane which is positioned more towards the crest is slightly higher at approximately NAP+4.0 m. The slope on the IJsselmeer side is 1 : 2.7 and has a revetment of basalt. It ends on a toe structure with a slope of 1 : 6 and a length of about 13 meter covered in riprap. The cross section of the Afsluitdijk might be adapted in the future. However, this is not of great importance for this thesis, because the focus of the thesis is on how to get to a overtopping resilient Afsluitdijk the current situation is the starting point. Off course these adaptations might be suggestions given as a results of the research done here.

3-5 Dike or dam

The question whether or not the Afsluitdijk is actually a dike often appeared during the execution of the thesis. Well the answer to that is not as clear as it seems. The discussion was mainly concerning calling the Afsluitdijk a dam instead. The name Afsluitdijk, implies that it is a dike. The primary function of a dike and a dam are defending the hinterland against open water and retaining water,

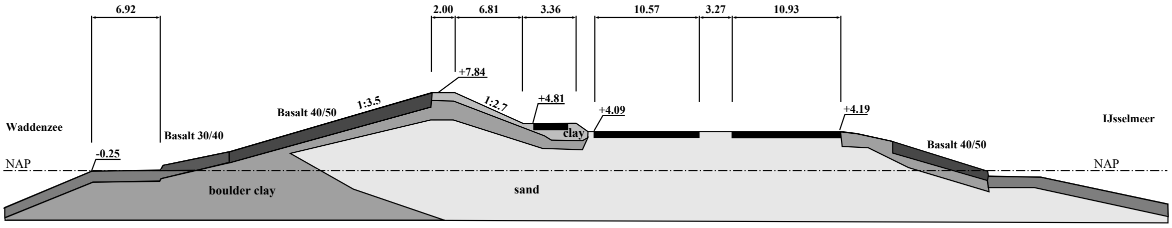

Figure 3-4: Typical cross section Afsluitdijk

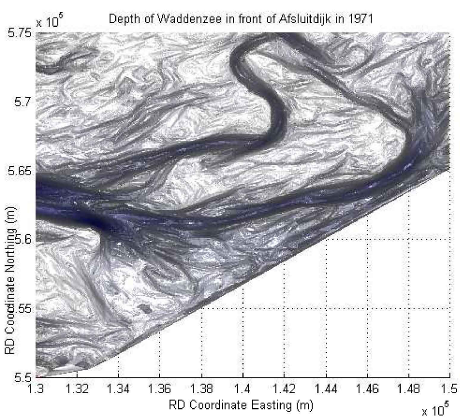

Figure 3-5: Gully development Waddenzee in front of Afsluitdijk 1971

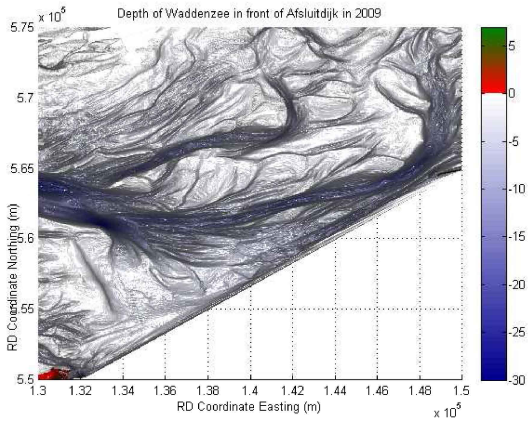

Figure 3-6: Gully development Waddenzee in front of Afsluitdijk 2009

respectively. Initially the function of the Afsluitdijk was to protect the hinterland against flooding. This would result in using the term dike. However, in time the IJsselmeer developed to be a strategical fresh water reservoir with a controllable water level, contained by the Afsluitdijk. It could thus be stated that the Afsluitdijk is a dam. In addition to this, a dam most of the time runs through water and a dike has water on one side and land on the other side. The Afsluitdijk is still considered a primary flood defense. This is still the primary function of the Afsluitdijk, it is a dike with functions of a dam. In addition to this the protective function of the Afsluitdijk is under consideration in the thesis. Therefore the Afsluitdijk is called a dike and not a dam in this thesis.

3-6 Definition of failure

Failure can be defined in a lot of different ways. Because failure is used as a limit for the overtopping resistance, one of the questions is when does the dike fail as result of wave overtopping. The failure of the landward slope is considered as failure due to overtopping. As stated earlier residual strength is not taken into account. Residual strength is the remaining strength after initial failure. Therefore initial failure is considered to be failure of the landward slope. This has been mentioned in a lot of literature and is often defined as *the failure of the landward slope top layer*. This will be considered as failure in this thesis as well. The landward slope has failed if the landward slope top layer has failed. Applied to the Afsluitdijk, it means that if the clay layer is eroded and the sand core is exposed, initial failure has occurred and thus failure of the landward slope top layer.

3-7 Focus of the thesis

To clarify the focus of this graduation thesis, the starting points and boundary conditions will be mentioned here. Most starting points have already been described earlier or will be explained in the next sections.

Only the dike of the Afsluitdijk is under investigation, the structures within the dike are not. The focus is on the cross section.

A smooth landward slope is defined as a slope which is uninterrupted and has no transitions at all. The slope is ending below the backward water level, so is running in the water directly without a berm.

The decision for the concept of the overtopping resistant dike is a political decision, other concepts have been evaluated by RWS, this choice will not be a point of discussion in the thesis.

Overtopping resistance is defined by RWS as being able to resist maximum wave overtopping for high water level belonging to 1/10 000 year conditions without taking into account residual strength, expected is wave overtopping discharges of >200 $l/s/m$.

The safety norm is an annually probability of failure of 1/10 000 year. This means that the dike has to be able to withstand conditions that occur with the same annual probability of 1/10 000 year. The choice for this safety norm is a political decision.

The solution has to be able to function at least until 2050, future reinforcements may not be made impossible. This includes increases in sea level rise.

The definition of failure as explained in 3-6 will be used. An additional step might be to show what is allowable damage during extreme conditions and how does it develop.

The Afsluitdijk should be designed with the W+ scenario of the Koninklijk Nederlands Meteorologisch Instituut (KNMI). This is for 2050 and 2100 a sea level rise of 0.35 and 0.85 m, respectively.

It is assumed that measures will be taken in order to prevent (the problems caused by large amounts of) infiltration of water in the dike.

The focus will be on the mechanism of wave overtopping and directly related failure mechanisms.

Wave transmission caused by overtopping will not be taken into account.

Only pulsative wave overtopping flow is considered, overflow and combined wave overtopping and overflow aren't considered.

The determination of the correct overtopping volumes is not the focus of the thesis. It assumes large overtopping values rather than calculating these volumes.

The hydraulic boundary conditions that belong to an annual probability of 1/10 000 year as determined by Deltares will be used in the thesis Deltares [2013].

3-7-1 Safety standard

The Afsluitdijk has to be able to withstand conditions belonging to a probability of exceedanc of 1/10 000 per year, which was confirmed in V&W [2007b]. In V&W [2004] the normative frequency is adapted to this 1/10 000 per year. This is instead of the 1/1430 per year, which was used as the hydraulic boundary condition for the first round of tests. This 1/1430 per year is the same as the Delta committee had proposed for the Afsluitdijk. However, during the preparation for the second round of tests the dike ring areas in the hinterland area of the Afsluitdijk where also considered. The choice was

Table 3-2: Normative frequency Afsluitdijk and surrounding areas

dike ring area	name	frequency per year	category
6	Friesland and Groningen	1/4000	a
7	Noordoostpolder	1/4000	a
8	Flevoland	1/4000	a
12	Wieringen	1/4000	a
13	Noord-Holland	1/10 000	a
1b	Afsluitdijk	1/10 000	b
4b	Houtribdijk	1/10 000	b

made to make the Afsluitdijk as safe as the normative dike ring area in the hinterland of the Afsluitdijk. This is dike ring area 13, that has a safety norm of 1/10 000 per year. The norm of the Afsluitdijk has thus increased with a factor 7. The Afsluitdijk appears to be a difficult case concerning the safety norm, the primary flood defenses of the Netherlands are divided into three different categories which can be found in V&W [2007a]. These are a, b, c and d:

a. primary flood defenses which belong to a system which surrounds dike ring areas, with or without high grounds, and protects directly against open water.

b. primary flood defenses protecting dike ring areas or connect different dike ring areas and protects directly against open water.

c. primary flood defenses which belong to a system which surround dike ring areas, with or without high grounds, and do not protect directly against open water.

d. primary flood defenses which belong to a system which surround dike ring areas, with or without high grounds (like category a and c), or protecting other dike ring areas or connect different dike ring areas (like b) but are out of the country borders.

The Afsluitdijk belongs to category b, it protects against open water (Waddenzee), it connects two dike ring areas (6 and 12) and protects several other dike ring areas (7, 8, and 13) as well as the Houtribdijk. The dike ring areas in the surrounding of the Afsluitdijk can be seen in figure 3-7[6] and are numbered. The normative frequencies are given in table 3-2 together with their name, number and category. The normative frequencies for the dike ring areas in the hinterland of the Afsluitdijk are almost all lower than 1/10 000 per year, only the Houtribdijk and dike ring area 13 are as high as the Afsluitdijk. Probably the Houtribdijk has this normative frequency because it protects dike ring 13 as well. This exact subject is under discussion at this moment. Some parts of the government want to work with the so called 'meerlaagse veiligheid' (multiple safety layers). In which the primary defenses can get a reduced safety level because humans can survive certain floods when well prepared, evacuation plans and survival packages are examples of measures that could be taken. The layers are: prevention, durable spatial planning and disaster control[7].

Another approach is, instead of speaking of a certain normative frequency for dike ring areas, is using risk. A risk approach is based on the fact that each dike ring area should have the same risk. Because risk is the probability times the consequences, not only the normative frequency is taken into account but also the consequences expressed in terms of money. The term cost-benefit analyses is mentioned as an option to determine the required safety levels of dike ring areas and the Afsluitdijk. If this is not complicated enough, an additional question is what is the value of a human life and can this be expressed in terms of money and thus risks. The risk approach is for instance mentioned in Grevers

[6] slightly adapted version of 'figuur 1-1' in V&W [2007b]
[7] deltaproof.stowa.nl/projecten/Projectdossier_Meerlaagse_Veiligheid.aspx?pId=23

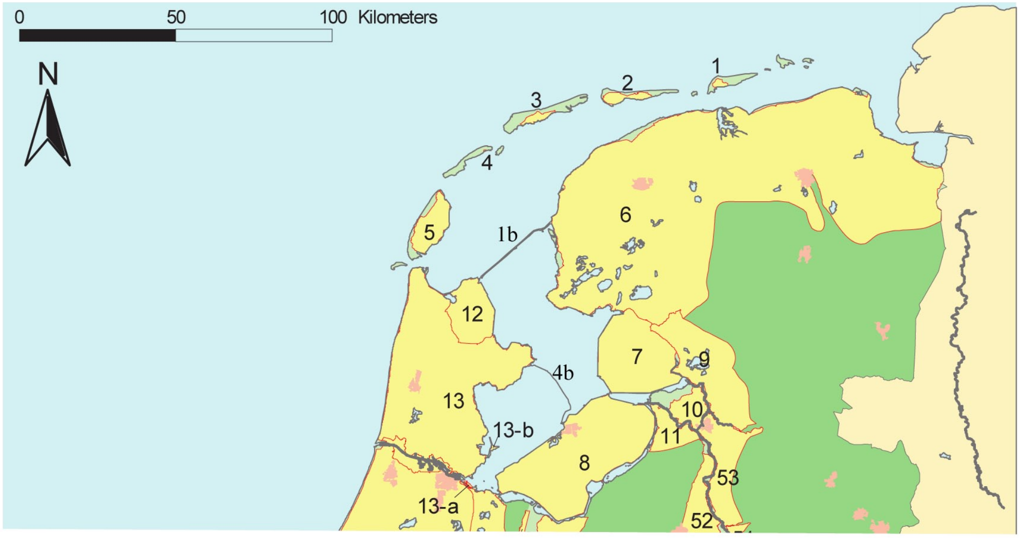

Figure 3-7: Dike ring areas

and Zwanenveld [2011]. Also the minister of Infrastructure and the Environment writes in her letter of 26 april 2013[8] that she is in favor of the risk based approach. The 1/10 000 level seems a high level for a dike which 'only' protects water, in the form of the IJsselmeer and other dikes.

However, to get a good insight in what the risk is, the consequences of a breakthrough for the hinterland dike ring areas should be investigated. What is the consequence of a gap in the Afsluitdijk? is the main question that should be asked. The water level of the IJsselmeer may rise but how much and how fast? What water level can the dikes at the hinterland of the Afsluitdijk deal with? and is this within acceptable limits? are all questions that should be answered. For other dikes an attempt has been made to execute a risk based approach, this can be seen in for instance Deltares [2011]. This will not be done in this thesis but it is important enough to mention and be aware off. A change in the normative frequency means a change in the hydraulic boundary conditions, this could mean that the strength of the dike can be reduced. The normative frequency as established for the third round of safety assessments will be used here, this is 1/10 000 per year.

[8]http://www.tweedekamer.nl/downloads/document/index.jsp?id=6e7540fd-1e1f-4e96-b3ce-606155c9a919&title=Verbetering%20van%20de%20normering%20voor%20waterveiligheid.pdf

Part II

Derivation and Application of Design Procedure

Chapter 4

Methodology

This chapter consist of three different aspects, eventually leading to the design procedure. The first aspect is the description of the input that should be used for the case of the Afsluitdijk, or better said the boundary conditions. The second part is analyzing the methods that are used to transform the input into a required top layer. The third part is the procedures, resulting from the analysis of the methods used. These procedures can be merged into the Main Design Procedure which will be presented in chapter 5.

4-1 Hydraulic boundary conditions

For the hydraulic boundary conditions used to determine the wave overtopping discharges and velocities the hydraulic boundary conditions for designing the strengthening of the Afsluitdijk as determined in Deltares [2013] are used. The boundary conditions include the following two supplements:

- Supplement taking into account uncertainties
- Supplement for the increase of the load during the lifetime as a result of climate changes

These two consist of:

- 10% surcharge on wave height and wave period
- 0.10 m surcharge on water level
- Absolute sea level rise in 2050 with respect to 1990: 0.35 m
- Absolute sea level rise in 2100 with respect to 1990: 0.85 m (IPCC report: 0.26-0.82 m[1])
- Taking into account the effect of relative sea level rise (including settlement) on the wave height by an increase of the wave height with 0.45 times the water level increase.
- Effect of land subsidence Waddenzee for 2050 en 2100: 0.00 m
- Effect of subsidence Afsluitdijk and dams Den Oever 2050: 0.05 m
- Effect of subsidence Afsluitdijk and dams Den Oever 2100: 0.10 m

[1] www.knmi.nl/klimaat/IPCC/SPM/H_5.php

Table 4-1: Hydraulic boundary conditions, given 1/10 000 in 2050

section	water level h [NAP+m]	wave height H_s [m]	mean period $T_{m-1,0}$ [s]	peak period T_p [s]	wave direction [degree.N]	orientation [degree.N]
1	-	-	-	-	-	-
2	-	-	-	-	-	-
3	-	-	-	-	-	-
4	5.30	2.70	5.42	6.30	328	325
5	5.26	2.58	5.49	7.02	318	319
6a	5.23	2.93	5.72	7.40	317	320
6b	5.26	3.09	5.92	7.51	321	319
7	5.29	3.12	5.95	7.54	321	319
8a	5.38	3.42	6.03	7.50	314	319
8b	**5.28**	**3.92**	**6.27**	**7.77**	**301**	**319**
9	5.30	3.84	6.33	7.81	300	319
10a	5.31	3.83	6.17	7.56	297	319
10b	5.33	3.65	5.99	7.27	297	319
11a	5.35	3.56	5.82	6.87	298	319
11b	5.39	3.08	5.77	7.01	300	324
12	5.43	3.45	6.00	7.40	313	346
13	5.43	3.33	5.88	7.08	312	345
14	5.47	3.17	5.64	6.84	305	346
15	5.46	3.03	5.60	6.97	301	337
16	5.46	3.03	5.60	6.97	301	320
17	5.46	3.13	5.80	7.36	295	315

The last two items, the effect of subsidence of the Afsluitdijk and dams at Den Oever are not jet taken into account in the model it self. They have to be taken into account for the design of the Afsluitdijk. The water levels and wave conditions are determined for each dike section, these sections can be found in table 3-1. Hydra-k (version 3.6.5) is used to do so, the sea level rise has been taken into account. For the year 2050, the year of interest, the values of the hydraulic boundary conditions are given in table 4-1. These are the conditions per dike section for 2050, with 1/10 000 year conditions. The governing dike section is dike section 8b (bolt). The water level for section 8b is not the highest of all the dike sections, this is for section 14 with 5.47 m. However, the significant wave height is the largest with 3.92 m. The mean spectral wave period is the second largest with 6.27 s. This combination of water level, wave height and mean period is governing for wave overtopping. For section 1, 2 and 3 Hydra-K output has not been determined, because these are a part of the Den Oever lock complex and do not have a direct attack from the Waddenzee. In table 4-2 the results for section 8b are shown for the years 2020, 2050 and 2100. These are the 1/10 000 year conditions. The table also shows the location of the normative cross section as well as the x-, and y-coordinates. The water level, wave height and wave period increase over time as can be expected with sea level rise. The year for the rehabilitation of the Afsluitdijk for Rijkswaterstaat (RWS) is 2050 and the data for this year will be used in the thesis.

4-2 Available procedures

Different methods can be used to solve the problem definition as stated in section 1-2. Several methods that can be used are treated in this section, some methods are unsuited to be used for solving the

Table 4-2: Hydraulic boundary conditions dike section 8b, 1/10 000

Dike section	8b				
Normative cross section	Afsluitdijk				
X-coordinate	141754	[m]			
Y-coordinate	558496	[m]			
Dike orientation	319	[gr.N.]			
year	water level h [m+NAP]	wave height H_s [m]	wave period $T_{m-1,0}$ [s]	peak period T_p [s]	wave direction [degree.N]
2020	5.00	3.85	6.20	7.68	301
2050	5.28	3.92	6.27	7.77	301
2100	5.77	4.06	6.41	7.93	301

problem or not useful for other reasons. These will be described here together with the reason why these will not be used as method in this thesis.

Model testing is one of the tools in order to prove the overtopping resilience of the Afsluitdijk. These tests need to be large scale tests because grass cannot be scaled down properly. Large scale test should therefore be used to prove the resilience of the final design. In addition to this, full scale testing takes a lot of time and is not possible within the time span of the thesis. However, it appeared that several knowledge gaps are unsolvable using theoretical methods as presented later. Therefore model tests will have to be performed in order to get full knowledge on the overtopping resilience. This can for instance be done with the wave overtopping simulator as described in section 2-4. The use of the overtopping simulator is a part of the procedure as presented in chapter 6 but has not been done in this thesis because of the before mentioned reasons.

Computer models can be used as an alternative to full scale model testing, in order to simulate the situations under consideration. There are certain methods available that might get a reasonable result, one could think of the use of a CFD (computational fluid dynamics) based model or a boussinesq based model. There are a lot of uncertainties that need to be investigated, the strength of the top layer is one of them. If the resilience is unknown it cannot be modeled. Therefore for this research it seems better to use this as an assisting method or an addition. It might not necessarily improve the insight in the mechanisms behind the resilience against wave overtopping and this is one of the goals of this thesis. In addition to this, free surface flow is difficult to model with a CFD based model. Also the transition from waves to free surface overflow is difficult to model. The calculation time of these models is long, for complex geometries it takes about a week to run such a model. The reduction of density as well as the turbulence in the water flow is hard to model with this kind of models. Summarizing it can be stated that numerical models can be used to predict the load and physical model tests are used to test the strength.

Because one of the main goals of the thesis is to understand what happens during extreme wave overtopping events, the choice has been made to mainly perform a theoretical study on the subject. The research approach or method is not one particular method but is a combination of different methods. The easiest way to describe the used method is *a theoretical approach*. A more extensive research description can be found below. Due to the fact that the execution of the thesis was an integrated process at which methodology, results and conclusion were simultaneously developed it is difficult to describe this process in a structured way. The procedure has been decomposed in order to be able to describe it as clearly as possible to the reader.

4-3 Used methods

The first step of the research was a review of available literature on the subject of wave overtopping. This has partly been conducted in order to formulate the problem definition, the objective and the research questions. During the execution of the thesis the literature review was continuously updated with new information. It is required to see what has already been done and on which part of the problem additional information is required. As a part of the literature review it is interesting to see whether there are methods that might show a positive outcome of the resilience against wave overtopping of the dike. Another important part is to see how resilient the current Afsluitdijk actually is against overtopping.

The review first focused on the Dutch design rules and methods followed by international design rules. This consists of collecting literature and giving a short description on what is available. This has partly been done in order to compose the research approach for the thesis. A description of the currently available design, test methods and the procedures that are being developed at this moment in time are known from the literature review. Also the practical procedures that can be used in order to prove the resilience against wave overtopping is described to see what has been tested and if that can be used to determine additional information on certain parameters.

The second step is to investigate if and where gaps of knowledge are present. A part of this investigation is interviewing different persons that have a lot of experience on the Afsluitdijk or the subject of wave overtopping. Also subjects that have a certain overlap or interface with the concept of an overtopping resilient dike will be investigated, one can think of breakwaters or spillways. The literature review focuses on two parts, on one hand investigating the parameters that are important when it comes to loads caused by the overtopping wave and on the other hand the parameters that are important for the resilience against these loads. A third aspect is to find a coupling between the load and resilience. Parallel to the literature review, a description of the study area has been made in order to gain all necessary boundary conditions, both areal and theoretical framework will be described.

The third step of the research has been to gain insight on the loads caused by an overtopping wave and how these loads will develop along the landward slope and berm, especially the new developments on the load side are interesting. It can result in a different way of looking at loads caused by overtopping.

The fourth step has been to investigate the resilience part. Less knowledge is available concerning the resilience, it was difficult to find any theory or literature on this subject for pulsating wave overtopping. Mainly it could be found in other area's that have certain parallels with the concept of the overtopping resistant dike.

The last step is to couple both insights gained on loads and resilience to see if the load can be coupled directly to the resilience. From this analysis it appeared on which parts additional research should be done and for which part a procedure can be developed. The procedure will be developed and applied to the Afsluitdijk.

4-4 Critical points

To be able to make a statement about the load or the strength first the points at which the combination of load and strength is the weakest should be known. In order to do so an analysis has been made based on theory and on tests that have been executed in the past.

Failure, as described in section 3-6, will most likely occur at the weak, or so-called critical points along the slope. It is important to know where these points are located, what these are and how these points influence the strength of the dike. These points do not have to be weak, also an increased load at the spot can make such a point critical. The criticality is caused by weakening of the landward slope top layer, introduce an extra load, a combination of the both or if the load it self is maximal. Often

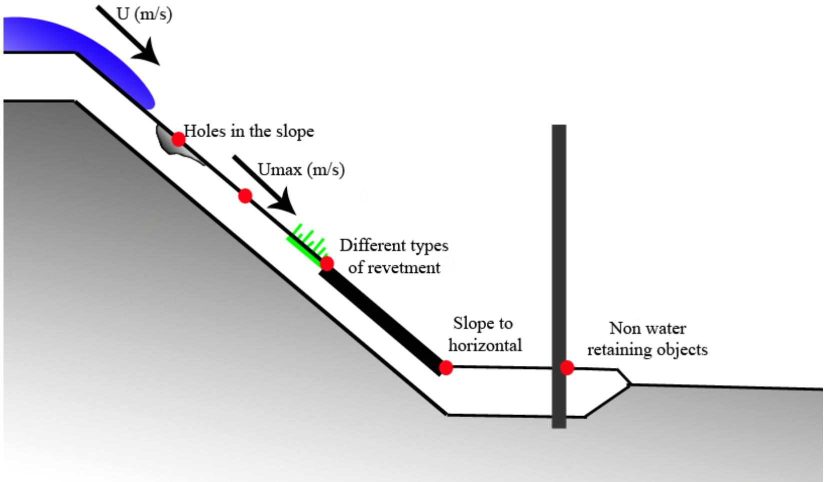

Figure 4-1: Critical points on landward slope

these weak spots are the points at which the initiation of erosion occurs. Preferably every dike should have a smooth slope, which runs directly into water. This means there are no transitions and the only critical point is the point on the slope where the flow velocity of the overtopping wave is the highest. Realistically this is impossible, at every dike objects or transitions and therefore critical points are present. The critical points on the landward slope of the Afsluitdijk are determined based on literature and on test executed on the Afsluitdijk and other dikes. These are:

- The point at the landward slope were the velocity is the highest.
- Holes in the landward slope or berm.
- Transition between different dike segments (longitudinal)
- Transition between slope and berm
- Transition between different types of revetment
- Transition between dike and Non Water Retaining Object(s) (NWO)

To clarify this, in figure 4-1 the critical points are shown schematically. In figure 4-2 the devision between the different critical points can be seen together with the influence on the load and strength of each point and how the failure is caused. The thick black lines in the figure lead directly to retrogressive erosion. One of the possible methods to asses the strength of the landward slope is the cumulative overload factor. As stated earlier in chapter 2 for transitions and obstacles the cumulative overload factor has been adapted with the factor α_m in equation 4-1 as stated in Steendam et al. [2012a].

$$\sum_{i=1}^{N} \left((\alpha_M U)^2 - U_c^2 \right) \tag{4-1}$$

The factor α_M depends on the kind of discontinuity or obstacle, if this method is a possibility to asses a certain critical point, it will be described based on theory or experiments. This concept is quite new and only one of the ways to asses the resilience of the critical points. Not for all critical points a certain value for α_M can be derived. Next to this the statement should be made that currently the cumulative overload factor is only based on the resilience of grass, not on other types of revetments

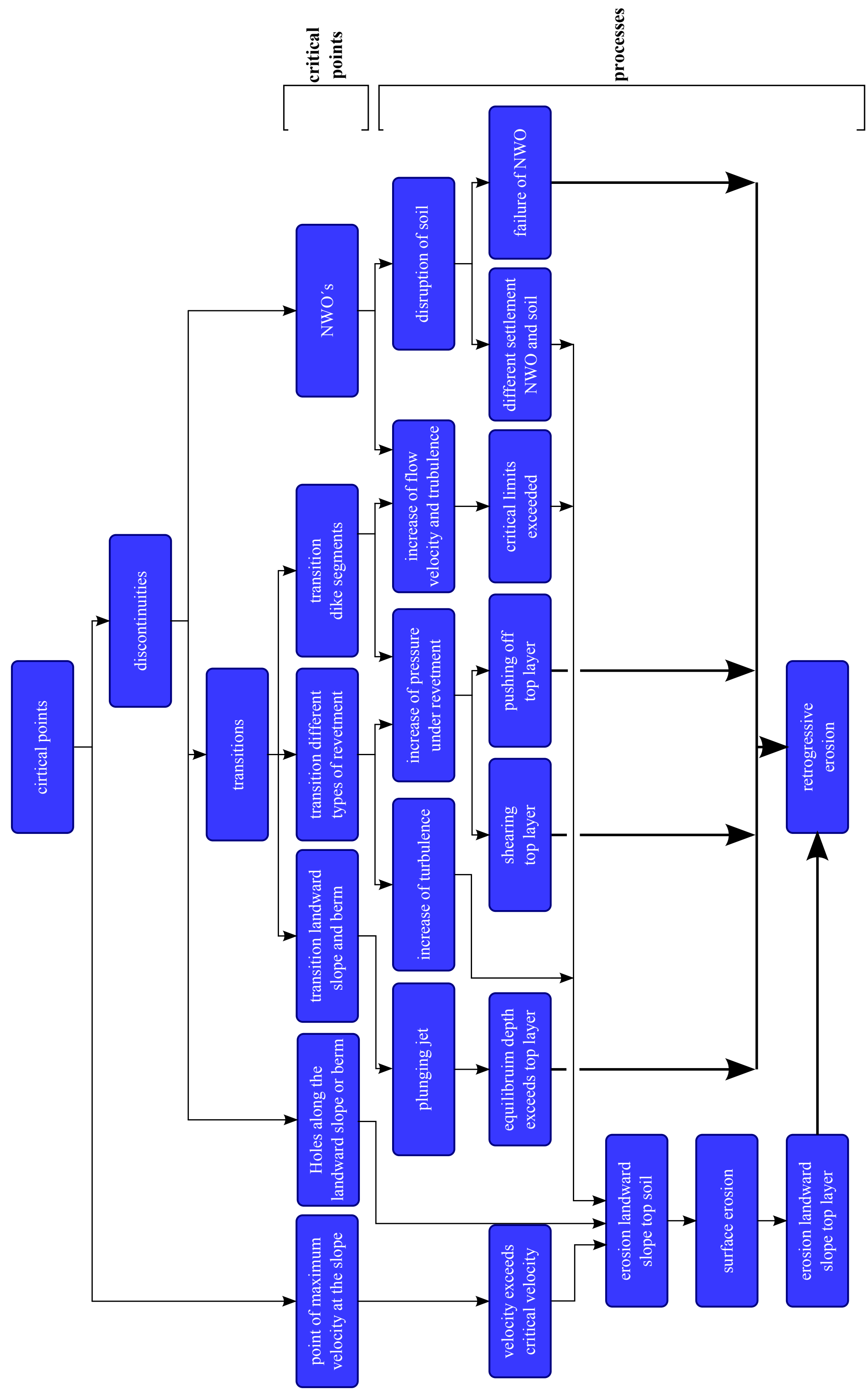

Figure 4-2: Critical Points

Figure 4-3: Damage maximum velocity at landward slope

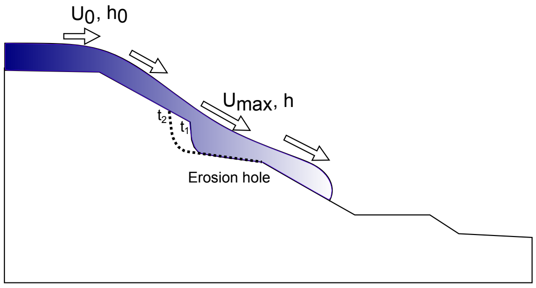

Figure 4-4: Maximum velocity at landward slope

and only on a couple of tests. In addition to this as described in section 2-2 the velocity to the second power is not used by everyone, instead the velocity to the power three (based on excess of work) is sometimes used. For each critical point the following aspects are treated in line with Morris [2012]:

- Type of critical point
- The potential failure modes and mechanism linked to these critical points
- Detect these critical points
- critical design aspects
- How to avoid problems
- Issues and questions
- Examples of these critical points

4-4-1 Point of maximum velocity at the landward slope

Description

This is a critical point that cannot be seen when standing on a dike. It is the location at which the velocity is largest, the exact location along the slope is hard to define. However, this isn't a problem because the revetment at the landward slope is usually uniform over the height of the dike. From measurements it appeared that the velocity along the slope increases down the slope and eventually decreases again. If no irregularities are present then this spot is most likely to erode as the first points on the landward slope. This is observed in full scale tests executed with the wave overtopping simulator on several real dikes.

Failure mode

The main failure mode is erosion of the top layer due to an excess of shear stress or velocity. Damage only occurs if a certain critical velocity or shear stress is exceeded. This is because the load is the largest at this point and not because the strength is decreased.

Figure 4-5: Initial situation holes in landward slope

Figure 4-6: Damage holes in the landward slope

Critical design aspects

Problems can only be seen when the dike is overtopped by waves and the damage starts. It is known that there will always be a point of maximum velocity along the landward slope, therefore anticipation of this larger velocity is possible and problems can be prevented when designing with an anticipation on this higher velocity. The problems are indicated when after a storm at a certain point along the slope the cover layer is damaged. Indicators for problems can be:

- observation of a bulge underneath the grass
- small spots of grass eroded away

4-4-2 Holes in the landward slope

Description

Holes can be found anywhere on the landward slope. These can be small holes, caused by small animals, or larger holes that are caused by settlements or an incident. The hole can be seen as a weakening of the top layer and a point at which the initiation of erosion occurs. There can be a certain stage before a hole has developed, when initially bare spots are present on the landward slope. These bear spots are a weak point at which a part of the protective top layer is missing or damaged. This can happen for instance with a grass layer if the grass is missing at some spots, these will most likely develop into holes, therefore the choice has been made not to make a separate critical point for the bear spots, but to treat them as holes.

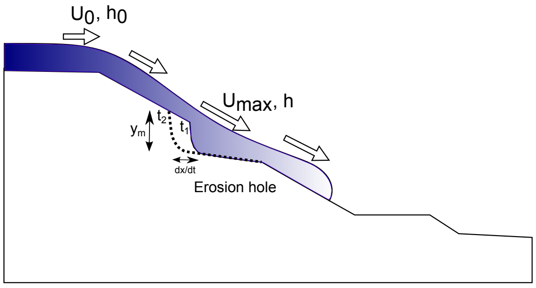

Figure 4-7: Head cut erosion

Failure mode

Damage is already present as in the initial hole. Depending on the flow depth, velocity, size of the hole and slope properties the damage will grow. The growth can be in downstream direction and in upstream direction. The failure modes in relation to these directions are the development of gullies and head cut erosion as in figure 4-7, respectively.

Critical design aspects

Problems can be detected by observations, initially a hole doesn't have to be a weak spot, especial very small holes do not have to be a problem. Larger holes usually are a problem and can be observed in situ. According to Van der Meer et al. [2009] indicators for problems can be:

- Holes larger than 15 cm
- Development of gullies around holes
- Enlargement of holes in time

Problems with holes can be prevented via regular maintenance in order to be able to repair holes before a large storm occurs and good monitoring during storms for the development of the holes.

4-4-3 Transition between dike segments

Description

Dike segments are different configurations of the dikes cross section, the transition is the point where two segments are joint together. The segments can be different in height, width, revetment type or a combination of these three. Depending on the type of dike segments, the transition gives different problems and therefore different failure modes. Because of the fact that the Afsluitdijk consists of 17 dike sections (for which the hydraulic boundary conditions are determined, so not all are transitions between different segments), there are a lot of transitions between different dike segments. Most of these transition are not visible when driving across the dike. These are transitions between different dike sections, which means that the layout of these sections are different. Some parts are just different segments because the load appears to be different and the dike has no other configuration what so ever. Another type of transitions are driveways at the landward slope of the dike, which oblique run up (or down) the landward slope, paved or unpaved as can be seen in figure 4-8. The fact that the transition it self varies along the profile of the dike and it can be paved or unpaved makes it a complicated transition. In addition to this the water also runs on the driveway it self. There are also parts of the dike on which plates are placed on the outer, landward slope and crest of the dike. This looks like a driveway as well, the combination of both the change in slope as well as the revetment type makes it a combination of the two types of transitions.

Figure 4-8: Initial situation transition dike segments in landward slope

Figure 4-9: Damage transition dike segments in the landward slope

Failure mode

The failure mode depends on the type of segments that are under consideration. A difference in height gives other problems than a difference in width or revetment type. The transition can introduce flow concentration or unexpected flow patterns. In addition to this also the interface between soft and hard revetments causes erosion. It is due to a excess of shear stress or velocity.

Critical design aspects

Problems can easily be seen, if a transition is not gradual, problems can more easily occur. Every dike segment transitions should be designed carefully.

4-4-4 Transition between slope and berm

Description

The transition between the slope and the berm is a transition between a slope and a horizontal. This type of transition can be found at the toe of the landward slope and at berm height. It can also be seen at driveways up the slope as described above.

Failure mode

The interface between the slope and the horizontal introduces problems because the flow down the slope attacks the transition heavily. The flow does not flow parallel but has an obliquely incident angle of attack, this results in larger and different load. The flow has to be 'bend' at this transition. The load

Figure 4-10: Initial situation transition slope to berm landward slope

Figure 4-11: Damage transition slope to berm landward slope

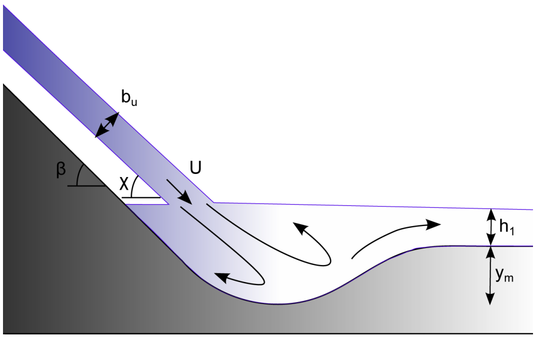

Figure 4-12: Jet principle at transition between slope and horizontal

is thus increased. If a slope is ongoing, the load would not hit the horizontal part but would directly flow into the water. The flow would than only flow parallel to the slope, while at this transition it hits the transitions under an angle. Damage occurs if the flow thickness and the velocity are such that the energy of each impact is to large for the transition to resist. The resilience of this transition depends on the 'smoothness' of transition and on the properties of the slope and berm material. The failure is due to an excess of velocity or force, the impacting jet causes this force. The flow would therefore need a load increase factor to account for such a transition between slope and horizontal.

Critical design aspects

This kind of transition can easily be spotted at each slope with a berm or an driveway. It is often stated that if the transition is gradual enough this isn't a problem, however how gradual this exactly has to be is unknown. During storms often erosion occurs at this spot first and can be seen as erosion of the cover layer.

Currently there are two ways of making a calculation on this critical point according to literature: The analogy with jet erosion and the cumulative effective hydraulic load, Verheij et al. [2012]. With the first approach a measure for failure might be the equilibrium depth. If this depth is larger than the top layer thickness than it is considered failure. The second approach is based on both erosion and fatigue, in which a damage number can be calculated. Limits are given as a certain value for the damage number D, for example failure: $D > 3500\ m^2/s^2$.

Figure 4-13: Transition between different types of revetment

Figure 4-14: Damage transition between different types of revetment

4-4-5 Transition between different type of revetments

Description

Transitions always introduce a weak/open spot along the slope. Therefore this is a weakening of the top layer. In addition to this the flow can be interrupted if the transition is between soft and hard, or it is just not gradual enough. Therefore the load can be increased as well, especially due to an increase in turbulence.

There are however transitions that cannot be placed in this, or in the category above, but which are present on the Afsluitdijk. For the different kind of transitions an inventory has been done on the Afsluitdijk. In table 4-3, the different transitions present on the Afsluitdijk are presented.

Table 4-3: Transitions at Afsluitdijk

Transitions
Pathways over the crest
Oblique driveways landward slope (paved/unpaved)
plates over the crest (Near Stevinlocks)
Transitions between different dike sections
Parking lots
Slope to horizontal
Highway A7
Cycling lane

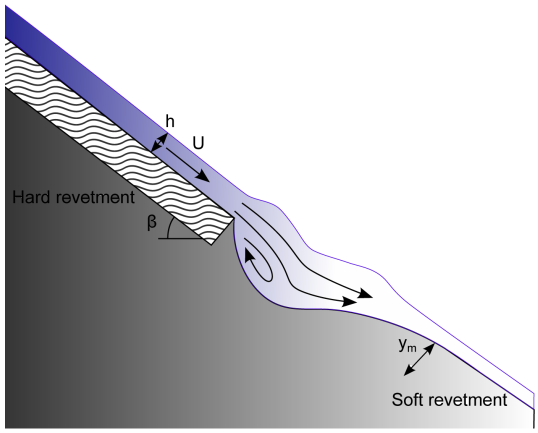

Figure 4-15: Transition between hard and soft revetment

Failure mode

Damage can occur in several ways. If the introduced turbulence is large enough so that it can lead to erosion. The damage starts with undermining, this is caused by the transition between a soft and hard revetment. The hard part is able to resist the flow while the soft soil is not. At the transition this difference becomes an erosion hole which leads to undermining. This erosion can undermine the revetment which causes it to fail. Another way of damaging is if the revetment is partly undermined, the incoming wave front is able to flow under the revetment which results in a pressure build up under the revetment. This can cause uplift of a revetment, pushing up a part or shearing of the revetment.

Critical design aspects

Transitions between different kind of revetment are easy to spot and can be seen clearly in development. Especially the transitions between hard and soft revetments should be avoided in order to prevent problems. When looking at present dikes, the revetments can be overgrown with vegetations. This means that the transitions are not visible, only when large amounts of overtopping occur the vegetation can be eroded after which the transition becomes visible again. This also happened during several tests with the wave overtopping simulator.

4-4-6 Transition between dike and Non-water-retaining objects

Description

The transition between the dike and NWO's introduces a transition between the object and the surrounding soil. A combination of the increasing load and the weakening of the slope makes this a critical points. The problem is that on large dikes there will always be NWO's and thus transitions are introduced. The variety in NWO's both in shape, size and penetration depth makes this a difficult category to cope with.

Failure mode

The damage can occur in a various ways, the damage under consideration in the thesis is damage as an effect of the concentration of the flow round the object. Uneven settlement of the object and the

Figure 4-16: Initial situation NWO staircase landward slope

Figure 4-17: Damage NWO staircase landward slope

Figure 4-18: Initial situation NWO tree landward slope

Figure 4-19: Damage NWO tree landward slope

surroundings can also cause damage. The erosion might get a better grip when uneven settlement occurs, due to the formation of cracks. The soil can also be disrupted by failure of the NWO it self. However the failure of the NWO is not taken into account. The interface between the NWO and the soil can give a preferred flow path which might cause micro instability or even piping. The strength therefore depends on a lot of different properties of the landward slope and berm and of the NWO it self.

Critical design aspects

The best way of solving this is to not allow any NWO's on the landward slope or berm. However, this is impossible for all dikes whether it are river, sea or lake dikes. In case of the Afsluitdijk this is only by the presence of the A7 alone, next to this there are a lot of NWO's on the Afsluitdijk. Due to this large variety of NWO's it is difficult to define the resilience part. It might be expressed with the help of fatigue, in which a certain critical velocity has to be defined. This critical velocity can depend on the type of object. As a countermeasure special water retaining objects can be constructed to protect the area around a NWO.

categorization of NWO's

To get a clear view on the NWO's that are present on the Afsluitdijk an inventory of all the NWO's has been made. The different NWO's are divided into several categories, which are chosen in such a way that the NWO's can be divided in the type of influence they have on the load or strength of the top layer. This is only possible because the variability and the amount of NWO's on the Afsluitdijk is manageable. There will be cases in which it is even more difficult to get a reasonable devision (only think about river dikes that runs through villages). At the Afsluitdijk the amount of NWO's is large, but often a single kind is repeated throughout the length of the dike (for instance the poles from the guard rail). These influences caused by NWO's can also be described as distinction between the type of transition, this is partly in accordance with Morris [2012] who only focused on structures:

- *Vegetation*
- *Structures*
  - *Buried*
  - *Buried but visible*
  - *External*
- *Cables and pipelines*
- *Poles*
  - *Poles with foundations*
    - *<15 cm*
    - *>15 cm*
  - *Poles without foundations*
    - *<15 cm*
    - *>15 cm*

The NWO's that belong to each category can be found, including photo's, in appendix C in table C-1. In this appendix a photo or screen shot of each NWO has been made in order to clarify each NWO. Also a more extensive description can be found in this appendix. A few examples of the NWO's can be seen in figure 4-20 to 4-23.

The described transition types show directly whether or not a structure penetrates the top layer of the landward slope. Different NWO's fit within this category, in this way not for every individual object an analysis has to be done. The subdivision is based on flow patterns, it can be under the structure, above, around or a combination of the three. Underneath protective revetments erosion can also occur but might not be visible until it is to late. The protection may suddenly fail during an extreme event. This can be solved by making sure that the filtration measures are sufficient. This allows water to flow underneath the revetment and pressure increase can be prevented, whilst the flushing out of sand particles is prevented by the filter. This is a dangerous way of failure, because initially it is not visible and therefore hard to observe and the failure then comes without warning.

Vegetation has an influence on both the load and strength of the top layer. The difference between vegetation and man made NWO's is that the roots of the vegetation mix with the top layer and man made structures do have abrupt transition between the top layer and the object. The structures can be divided into bridges and buildings. From testing it appeared that if objects are smaller than 15 cm the influence on surface erosion is very small Van der Meer et al. [2009]. When the objects become larger they do have an influence on the erosion process. There is a large variety of poles present on the dike, both with and without foundation. However, if the foundation becomes very large it might be better to think of it as structures instead of poles. Some objects cannot be placed in a specific category because these are unique in shape or interaction with the dike, for these objects a separate analysis has to be done.

As long as cables and pipelines are buried in the subsoil the flow and the strength of the top layer are not influenced. Therefore these are not interesting for the failure caused by erosion due to wave overtopping. This in contrary to the drainage channels of the highway A7, these have an exit point at the IJsselmeer side of the Afsluitdijk. Failure of the cables and pipelines is not considered in this thesis, because this would cause instantaneously failure of the landward slope top layer. But this is

Figure 4-20: Examples of buried but visible structures and poles

Figure 4-21: Examples of structures

Figure 4-22: Examples of transitions and others

Figure 4-23: Examples of external structures

worth mentioning. The choice has been made not to put the cables and pipelines within the category of buried structures, this is because the difference is that the buried structures are for example overgrown over the years or can be present in the landward slopes top layer and have a direct influence on the strength of this top layer, cables and pipelines have not. The lock complexes within the dike aren't seen as NWO's however, during the design the transition between lock and dike are critical points. Special transitional structures are needed to make sure that the transition does not fail. Currently at the location of this transition wider dike sections (or maybe small 'islands') are present which results in a smaller load on these transitions.

4-5 From input to load

In order to be able to make sure that the Afsluitdijk is strong enough to resist the loads caused by wave overtopping, these loads have to be determined. This section describes the procedure how to get from the input parameters to a load at the crest and landward slope. The dike has to be able to withstand this load.

4-5-1 Input

For the determination of the load on the crest and landward slope of the Afsluitdijk a script has been written in order to be able to quickly process changes. The different steps of this script, including the formulas will be described here. The output can be seen later in chapter 5. Also the references that resulted in the different steps will be mentioned in this section. The script can be found in Appendix E. For every calculation input parameters are required. These input parameters need to be gathered or determined on forehand. The script starts with the definition of the input parameters which are required for the next steps. The input required to determine the load is given in table 4-4. These

Table 4-4: Input variables

symbol	description	unit
B_b	berm width	m
g	gravitational acceleration	m/s^2
H_{crest}	crest height	NAP+m
H_{m0}	zero-th order spectral wave height	m
H_{berm}	berm height	NAP+m
H_w	still water level	NAP+m
T_{storm}	storm duration	h
$T_{m-1,0}$	mean spectral wave period	s
α	slope of the revetment	-
β	1. angle of attack	-
	2. landward slope angle	-
γ_f	reduction factor permeability and roughness	-
γ_r	reduction factor roughness	-
γ_v	reduction factor vertical wall	-

parameters are the basic parameters. There are some additional parameters that are required as input for the other formulas. These parameters need to be calculated, this can be done with the help of the input parameters as defined in table 4-4. These additional parameters are described below.

An important parameter is the freeboard which is defined as equation 4-2. This gives the difference between the crest and the water level, the lower this is the larger the wave overtopping will be. An even more important parameter is the relative freeboard which is defined as the freeboard divided by the significant wave height. As can be seen later on, a lot of the overtopping formulations are based on this parameter.

$$R_c = H_{crest} - H_w \tag{4-2}$$

The number of incoming waves during a storm:

$$N_w = \frac{T_{storm} * 3600}{T_{m-1,0}} \tag{4-3}$$

This is a fictive number, because the number of zero down crossings are much more than the number of waves resulting from equation 4-3. This is because these are only the waves with the period $T_{m-1,0}$. However, this is the number of waves that should be used as incoming waves for the calculations used here.

The surf similarity parameter (Iribarren number) ξ_0:

$$\xi_0 = \frac{\tan(\alpha)}{\sqrt{\left(\frac{H_{m0}}{g \cdot T_{m-1,0}^2/(2\cdot\pi)}\right)}} \tag{4-4}$$

Reduction factor for angle of attack of incoming waves γ_β:

$$\gamma_\beta = 1 - 0.0033 \cdot |\beta| \tag{4-5}$$

Reduction factor for the presence of the berm γ_B:

$$\gamma_B = 1 - \frac{B_B}{L_B}\left[0.5 + 0.5 \cdot \cos\left(\pi \frac{h_B}{x}\right)\right] \tag{4-6}$$

with:

$$h_b = H_w - H_{berm} \tag{4-7}$$

$$x = 2 \cdot H_{m0} \tag{4-8}$$

$$L_b = \frac{1}{\tan(\alpha)} \cdot H_{m0} \cdot 2 + B_b \tag{4-9}$$

4-5-2 Wave overtopping discharge

With the above parameters the run-up level that will be exceeded by 2% of the waves, $R_{u2\%}$, can be determined with equation 4-10. This would be the run-up level if the slope is long enough to reach this level. Depending on the value of ξ the left or right side of is valid. From measurements and testing it is derived that for values of $\xi < 1.8$ the left hand side is valid, for other values the right side is valid.

$$\frac{R_{u2\%}}{H_{m0}} = min\left(A \cdot \gamma_b \cdot \gamma_r \cdot \gamma_\beta, \gamma_r \cdot \gamma_\beta \cdot \left(B - \frac{C}{\sqrt{\xi_0}}\right)\right) \tag{4-10}$$

A, B and C are curve fitting parameters and have the values of 1.65, 4.0 and 1.5 respectively. The testing has been done for both shallow water as for deep water waves.

Intermezzo 1

An important aspect which should be realized calculating the above is that if the waves feel the bottom, so when it is shallow, instead of a Rayleigh distribution a Weibull distribution with a shape parameter of 3.6 (a Rayleigh distribution is a Weibull distribution with a shape parameter of 2) should be used for the distribution of the waves. This is if the transitional wave height H_{tr} is smaller then the wave height under consideration Battjes and Groenendijk [2000]. This depends on the depth and the slope of the foreshore. The transitional wave height can be calculated with equation 4-11.

$$H_{tr} = (c_1 + c_2 \cdot \tan(\alpha)) \cdot d \tag{4-11}$$

In which α is the slope of the foreshore and d is the depth. A linear variation of the foreshore slope is assumed in which c_1 and c_2 are empirically determined at 0.35 and 5.8, respectively. It means that for a steeper slope , less waves deviate from the Rayleigh distribution than with a milder slope. This can be explained with the fact that waves need time to adapt to the bottom configuration. The slope of the foreshore is an important parameter for the determination whether or not a Rayleigh distribution is valid. The larger waves are, the more these are influenced, large waves 'feel' the bottom earlier than the smaller waves. Therefore certain wave heights will not occur anymore. The foreshore near the Afsluitdijk is variable, along the length of the dike and also in time, this can clearly be seen in figures 3-5 and 3-6. The foreshore is not uniform and not linear, these two assumption are being made in order to reach equation 4-11. Depending on whether or not there is a shallow foreshore the waves do have a different distribution or otherwise said spectrum. Depending on the properties of the spectrum H_{m0} and $T_{m-1,0}$ the probability of overtopping can be calculated with $R_{u2\%}$. This level can be calculated using equation 4-10, according to van der Meer [2002] the formula for run-up is based on measurements, both in deep water and with a shallow foreshore.

Therefore the deviation of the spectrum, as described in Intermezzo 1, is already taken into account. In addition to this, the breaker parameter, which is also used in that formula is based on the spectral wave height and the spectral wave period, thus taking into account the spectral change due to the foreshore. All together the method described above can still be used even knowing that the foreshore is highly variable and shallow. This has been taken into account in the various formula's. However, it is important to realize that the foreshore is highly variable in time and space.

Understanding this and continuing, with this run-up level from equation 4-10 and relative freeboard from equation 4-2, the probability of overtopping P_{ov} of an incoming wave during a storm can be calculated. If the probability of overtopping and the number of incoming waves during a storm N_w are know the number of overtopping waves N_{ow} can be determined using equation 4-12.

$$N_{ow} = N_w \cdot P_{ov} = N_w \cdot \exp\left[-\left(\sqrt{-ln0.02}\frac{R_c}{R_{u2\%}}\right)^2\right] \tag{4-12}$$

This number of overtopping waves are the waves that overtop the dikes crest and therefore are able to cause damage to the crest or landward slope. These are the waves that need to be taken into account. The next thing that needs to be known is that when such a wave is overtopping the dike how much water is overtopping the dike. This parameter is the wave overtopping discharge q, to get to this number first the dimensionless wave overtopping Q should be determined using equation 4-13.

$$Q = \frac{0.067}{\sqrt{\tan(\alpha)}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.75 \cdot \frac{R_c}{\xi_{m-1,0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{4-13}$$

As described in section 2-1 this dimensionless wave overtopping discharge is depended on the angle of the outer slope α, the surf similarity parameter ξ as described in equation $4-4$, the reduction factor for the presence of a berm γ_B as described in equation 4-6, reduction factor for friction of the outer slope γ_f, reduction factor for the angle of incoming waves γ_β as determined in equation 4-5, a reduction factor for the presence of a vertical wall γ_v and the freeboard R_c as determined with equation 4-2. However, this is a dimensionless value, for the definition of load a dimension should be known. This can be calculated from the average dimensionless wave overtopping discharge using equation 4-14. Both the gravitational acceleration g and the significant wave height H_{m0} are used in order to calculate the average wave overtopping discharge q in $m^3/s/m$.

$$Q = \frac{q}{\sqrt{g \cdot H_{m0}^3}} \tag{4-14}$$

This wave overtopping discharge can be seen as a load and is defined as such at this moment. Certain amounts of wave overtopping are allowed, up till now this is limited to 10 $l/s/m$. Based on test the allowable average overtopping discharge is up to 30 $l/s/m$, these are described in Van der Meer et al. [2009] and Van der Meer et al. [2011]. When the values of overtopping discharge are larger the average wave overtopping discharge q is not a good measure any more.

Because the average overtopping is not the main cause of damage, but damage is caused by the larger overtopping volumes, above an wave overtopping discharge of 30 $l/s/m$ the focus should be on the larger volumes instead.

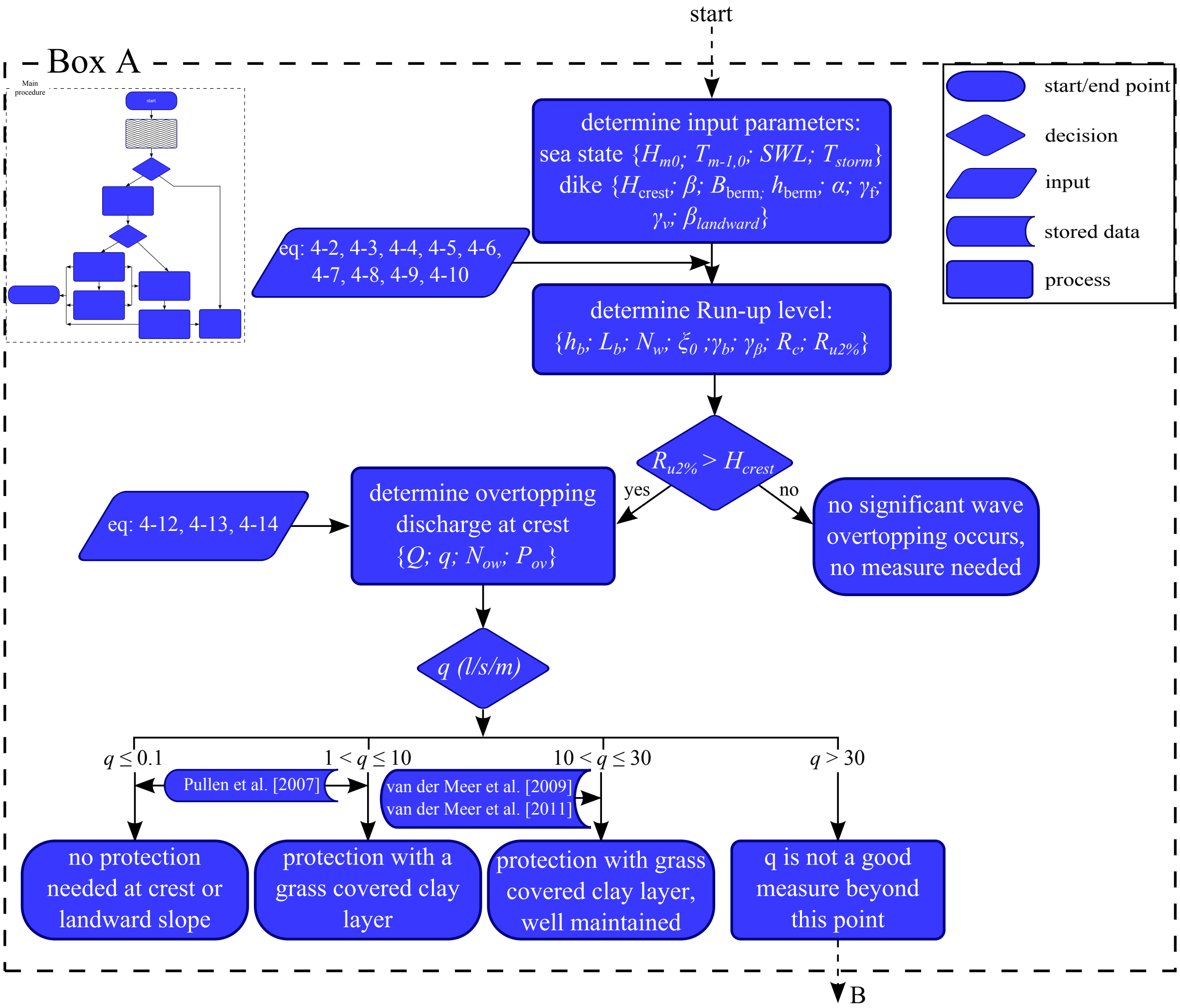

Figure 4-24: Box A: procedure to determine wave overtopping discharge q

The described procedure in this section is depicted in figure 4-24. The different symbols in the flowchart represent an action or input during the procedure. This meaning of each shape is shown as a legend in figure 4-24 in this chapter and in figure 5-1 in chapter 5. The position within the developed main design procedure can be found in the top left corner of the figure.

Figure 4-24 is a visual presentation of the procedure in order to get to the average wave overtopping discharge. If the average overtopping discharge of 30 $l/s/m$ is exceeded a different measure is needed for the load. This is indicated with the dotted arrow to **B** in the figure of the procedure. This arrow leads to the next section.

4-5-3 Flow velocity

The procedure presented in figure 4-24 shows that for larger discharges than an average overtopping discharge q of 30 $l/s/m$ another method for the load is required. The load on the landward slope is being caused by the overtopping wave. At the landward slope these loads are translated into a front flow velocity U (m/s) and a flow depth h (m). Local turbulence and increase in flow velocity is often the initiator of erosion.

What makes the load of an overtopping wave more difficult to handle than 'steady' overflow is the pulsating character of the wave overtopping flow. This gives a different kind of load. The load of steady state overflow is more constant, allowing weak parts to settle and giving it an increased strength. Due to the pulses the top layer is constantly loaded and unloaded, so that particles are not able to settle. This pulsating character causes also under and overpressure which have to be handled in a different way. The load should be expressed per wave.

For the load it is important to know how each wave results in a load at the crest or slope. The load should be expressed per wave. In order to do so as a first step the overtopping volume is determined. This is the overtopping volume per wave expressed in m^3/m. The overtopping volume per wave is based on the probability of overtopping which is defined as the second part of equation 4-12. The probability of a certain overtopping wave volume to occur for a random wave is given as a Weibull distribution with the parameters a and b. This Weibull distribution is given by equation 4-15.

$$P_v = 1 - exp\left[-\left(\frac{V}{a}\right)^b\right] \tag{4-15}$$

The parameters for this Weibull distribution are b, which is the shape factor of the Weibull distribution, it defines the extend of the tail and a is the dimensional scale factor which normalizes the distributions, defined as equation 4-16 given in Hughes et al. [2012].

$$a = \left(\frac{1}{\Gamma\left(1 + 1/b\right)}\right)\left(\frac{q \cdot T_m}{P_{ov}}\right) \tag{4-16}$$

Γ is the mathematical gamma function. q $(m^3/s/m)$ is again the average wave overtopping discharge according to equation 2-3 and 4-14, T_m (s) the mean wave period and P_{ov} the probability of wave overtopping. For the constant shape factor $b = 0.75$, which resulted from 14 sets of overtopping tests, the corresponding scale factor is given by.

$$a = 0.84 \cdot T_m \cdot q \cdot \frac{N_w}{N_{ow}} \tag{4-17}$$

N_w is the number of waves during a storm and N_{ow} the number of overtopping waves. These tests were conducted on mild seaward slopes and low to moderate overtopping conditions.

Often b is presented as a value of 0.75. However, it appears that for smooth impermeable slopes this factor depends on the relative freeboard R_c/H, this relation is given according to Hughes et al. [2012] in equation 4-18.

$$b = \left[exp\left(-0.6\frac{R_c}{H_{m0}}\right)\right]^{1.8} + 0.64 \tag{4-18}$$

This means that a lower relative freeboard gives a larger value for b and that the lowest value for b is 0.64 for a large relative freeboard. A larger value for b means that more waves will overtop the dike but with more or less the same volume and a smaller value means that a smaller amount of waves overtop the dike but with higher volumes per wave Zanuttigh et al. [2013].

This relation does not apply on ruble mount structures, the value of b was plotted against the relative discharge which is defined as:

$$\frac{q}{g \cdot H_{m0} \cdot T_{m-1,0}} \tag{4-19}$$

This resulted in another relation for smooth impermeable structures for the value of b:

$$b = 0.73 + 55\left(\frac{q}{g \cdot H_{m0} \cdot T_{m-1,0}}\right)^{0.8} \tag{4-20}$$

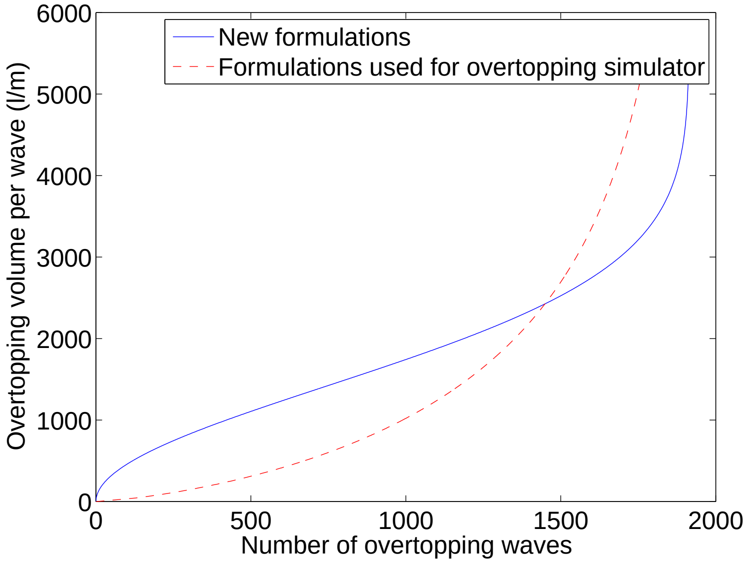

Figure 4-25: Comparison of wave overtopping volume distribution based on 1/10 000 conditions at the Afsluitdijk, old vs new method

So with an increasing relative discharge the value for b also increases thus resulting in more overtopping wave volumes with more or the less the same volume.

The dimensionless wave overtopping is again a function of the relative crest height. So the resulting wave overtopping volume per wave is also a function of the relative crest height. Another formulation of the value of b is given by Victor [2012] as a function of the slope angle α and the relative freeboard based on conditions representing heavier overtopping on steep seaward slopes.

$$b = \exp\left(-0.2\frac{R_c}{H_{m0}}\right) + (0.56 + 0.15 \cdot \cot\alpha) \tag{4-21}$$

Combining equation 4-21 and 4-16, Victor [2012] found a different formulation for the scale factor:

$$a = 1.13 \cdot \tanh(1.32 \cdot b)\left(\frac{q \cdot T_m}{P_{ov}}\right) \tag{4-22}$$

The latter gives the best fit for a released volume by the overtopping simulator, just after release on a horizontal or crest. From these measurements the presumption is made that the flow velocity increases at the slope. So equation 4-22 for a will be used in the thesis as well as equation 4-21 for b. The shape of the Weibull function and thus of the overtopping wave volumes is depended on the relative freeboard, the angle of the outer slope α, average wave overtopping discharge q, the mean wave period T_m and the probability of overtopping P_{ov}.

Intermezzo 2

Based on these kind of volume distributions also the overtopping simulator input files are composed van der Meer et al. [2006]. The probability distribution is converted to the number of waves in ascending or descending order, the relation is between the number of waves and the volume per wave as depicted in figure 4-25. These input files currently are not based on the shape and scale factor formulas used here but on equation 4-17 and corresponding shape factor b of 0.75.

This results in a different distribution of the overtopping wave volumes and overtopping flow velocities. It appears that smaller wave volumes for the method used here lead to the same average overtopping discharge. The integral of the distribution, which gives the total overtopping wave volume during the storm, is larger for the old method. Otherwise said, the same volume leads to a larger overtopping discharge.

Apply this to the overtopping simulator, for certain overtopping volumes it could be that a larger overtopping discharge should be the result instead of what was originally thought. A smaller volume per meter width is required to simulate the same discharge. With the same discharge the volume reduces for the two methods and the velocity on the crest and landward slope will change as well. However, the change in flow velocity due to other amounts of average wave overtopping discharge is not an issue here, because these are directly related to the overtopping volume via empirical relations. The failure of the slope or certain parts of the slope at certain velocities is still valid. But these velocities might occur from larger overtopping wave discharges then was assumed. Depending on the amount of overtopping the difference between the old distribution and the new becomes larger for larger amounts of wave overtopping. In addition to this for the overtopping simulator the lower parts of the graph are not used, so when using only the larger volumes, the differences becomes even larger. This needs further research and is case sensitive. It might be interesting to perform this calculation for the wave overtopping tests that have been done on the Afsluitdijk in order to get a insight in the strength of the dike according to the new formulations for the Weibull scale and shape factor.

As described at the beginning of this section, the front flow velocity and depth are the loading parameters that need to be known. The front flow velocity and depth of the overtopping wave are dependent on the overtopping volume per wave. However, a certain volume per wave has a certain probability of occurrence, which is depended on the relative freeboard or the relative wave overtopping discharge. The probability of a certain front flow velocity is coupled to the relative freeboard, therefore the load depends on the crest height, water level and the wave height.

During tests the flow depth h has been measured in two ways, by poles and by so called 'surfboards' that flow on top of the overtopping wave. The angle of rotation at the axis round which the surfboard rotates can be translated into an flow depth. The front flow velocity has been measured by high speed camera's in combination with a on forehand defined spatial grid drawn on the slope of the dike and by flow pedals positioned on the surfboards. This has resulted in a wave volume V (m^3) dependent front flow velocity u and depth described by equation 4-23 and 4-24, respectively. This is an empirical relation based on experiments on the Vechtdike (van der Meer et al. [2010]) as mentioned in equation 2-16 and 2-17.

$$u = 5.0 \cdot V^{0.34} \tag{4-23}$$

$$h = 0.133 \cdot V^{0.5} \tag{4-24}$$

However, after tests done in Tholen and in Belgium the front flow velocities and depths on the crest and on a horizontal were measured. These measurement led to a slightly different relation for the front flow velocity and depth, reported in Steendam et al. [2012a] and are given in equation 4-25 and 4-26.

$$u = 4.5 \cdot V^{0.3} \tag{4-25}$$

$$h = 0.1 \cdot V^{0.75} \tag{4-26}$$

By using the last two equations, the resulting front flow velocities are lower than with the older method of two equations above. The flow depth is slightly smaller up to a volume of 3.1 m^3/m, while for larger volumes it is larger for the older method. This can be seen in figures 4-26 and 4-27 for the front flow

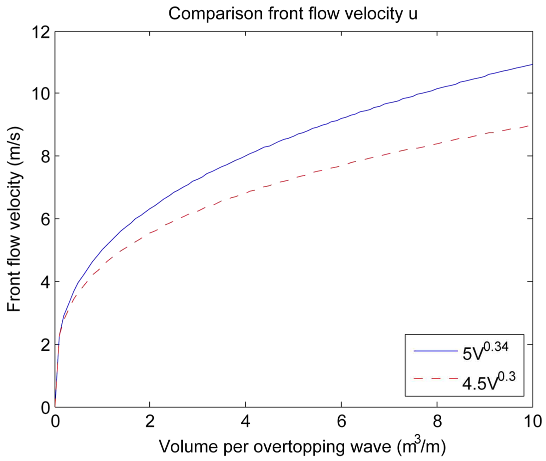

Figure 4-26: Comparison flow velocities

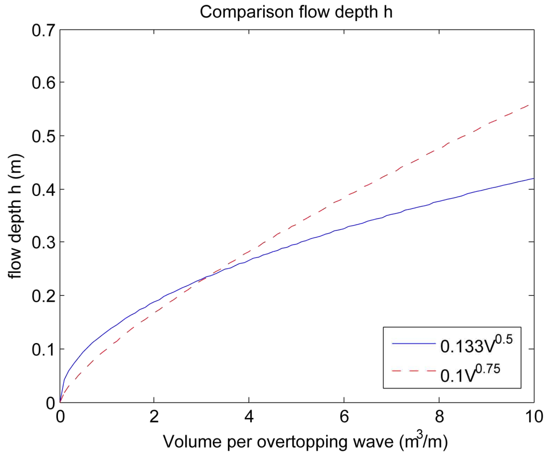

Figure 4-27: Comparison flow depths

velocity and the depth, respectively. These figures are plots of the relations between volume and front flow velocity and volume and flow depth. With the additional testing it appeared that the adaptation of the formulas was a better fit for the data gathered using wave overtopping tests, both for the velocity as for the flow depth. The difference with the old formulations can be seen clearly.

With these two equations the distribution of wave overtopping volumes per wave can be transformed to the distribution of overtopping front flow velocities for each wave at the crest together with the overtopping depths at the crest. This distribution of overtopping front flow velocity per wave is the load at the crest. This distribution gives an overtopping front flow velocity for each overtopping wave. By expressing this per wave the pulsating character of wave overtopping has been taken into account. This is the overtopping front flow velocity of distribution at the crest. This is the load for a smooth crest, without any discontinuities. If the slope is indeed smooth the required critical velocity can be calculated as described in section 4-6. Otherwise the next steps should be followed. In addition to this the development of the velocities down the slope should be taken into account as well. The flow velocity increases while the wave front is running down the landward slope. This leads to the first critical point, maximum velocity at the landward slope.

The theory on which the flow velocity and thickness development along the slope are based is the steady state theory. However, from measurements in Tholen and Belgium it appeared that the flow velocity did again decrease further down the slope, which is not according to steady state theory. From Steendam et al. [2012a] it appears that the front velocity of the overtopping wave increases when it flows down the landward slope of the dike. This can probably be explained by the fact that along the slope the overtopping duration increases. If the duration increases the velocity has to decrease again down the slope. On the upper part of the slope the velocity increase due to the down flow seems to be dominant over this duration increase. Depending on the overtopping wave volume the increase in velocity is approximately 20-35 %. These are the maximum velocities along the slope. This is considerably larger then the initial velocity at the crest. Therefore the front velocity of the overtopping wave should be increased as well with 20-35 %. For the flow thickness a similar result can be seen, the overtopping flow thickness becomes lower along the slope and depending on the volume then increases again or not. If the duration increases along the slope the flow velocity and the flow thickness don't have to be in phase anymore. These are all aspects that do not contribute to the validity of the steady state theory. However, with this steady state approximation the maximum flow velocity along the slope can be predicted, this is with a roughness factor f of 0.01. This factor is derived by comparing results of several test to calculations with different friction factors. The equations that can be used to calculate this steady state development along the slope are:

$$u = \frac{u_0 + \frac{k_1 \cdot h}{f} \cdot \tanh \frac{k_1 \cdot t}{2}}{1 + \frac{f \cdot u_0}{h \cdot k_1} \cdot \tanh \frac{k_1 \cdot t}{2}} \tag{4-27}$$

with

$$k_1 = \sqrt{\frac{2 \cdot f \cdot g \cdot \sin\beta}{h}} \tag{4-28}$$

$$t \cong -\frac{u_0}{g \cdot \sin\beta} + \sqrt{\frac{u^2}{g^2 \cdot \sin^2\beta} + \frac{2 \cdot s}{g \cdot \sin\beta}} \tag{4-29}$$

$$h = \frac{u_0 \cdot h_0}{u} \tag{4-30}$$

The velocity on the slope u depends on the initial conditions of the flow velocity u_0 and depth h_0 on the top of the slope, the angle β of the landward slope, the friction f and the position on the slope s. The unknown is the friction f, that has been optimized at 0.01 for grassed landward slopes in Steendam et al. [2012a]. However, on theoretical basis Schüttrumpf and Oumeraci [2005] recommend a value of 0.0058. The flow thickness depends on the velocity and the initial conditions. The maximum velocity in combination with the flow depth and duration should be known to determine the maximum load along the slope. This can be done by substituting the value of u in equation 4-1 with the maximum value u_{max} along the slope. Another way is to compute the amplification factor α_M as a function of the initial conditions and the slope properties as described above.

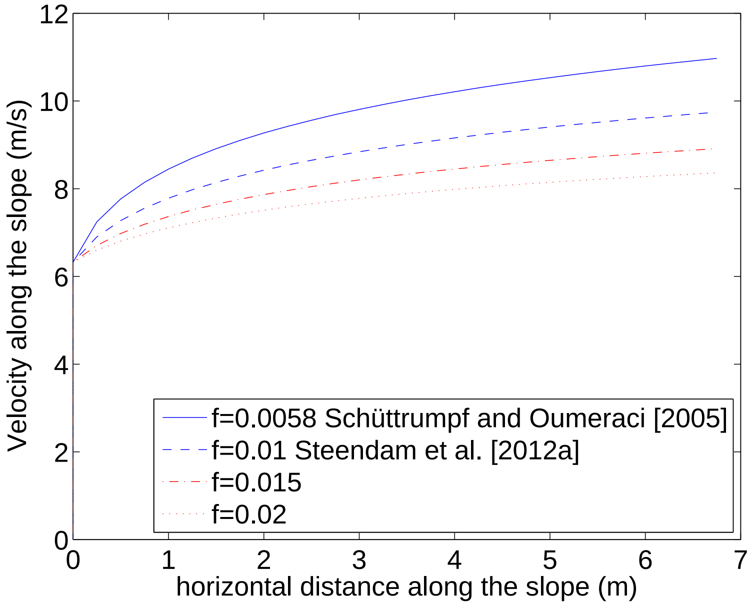

Figure 4-28: Maximum velocity along the landward slope

This method can be applied to the Afsluitdijk. The friction factor is the variable that has the largest uncertainty. The two values that are recommended give different results and do not seem to give a good fit for the larger overtopping volumes. The overestimation of the maximum flow velocity along the slope increases with a increasing overtopping discharge (or initial overtopping flow velocity on the crest of the dike). It seems the best choice based on the measurements in Tholen and Vechtdike for f=0.015. This gives a slightly lower value for the maximum velocity along the slope than the more

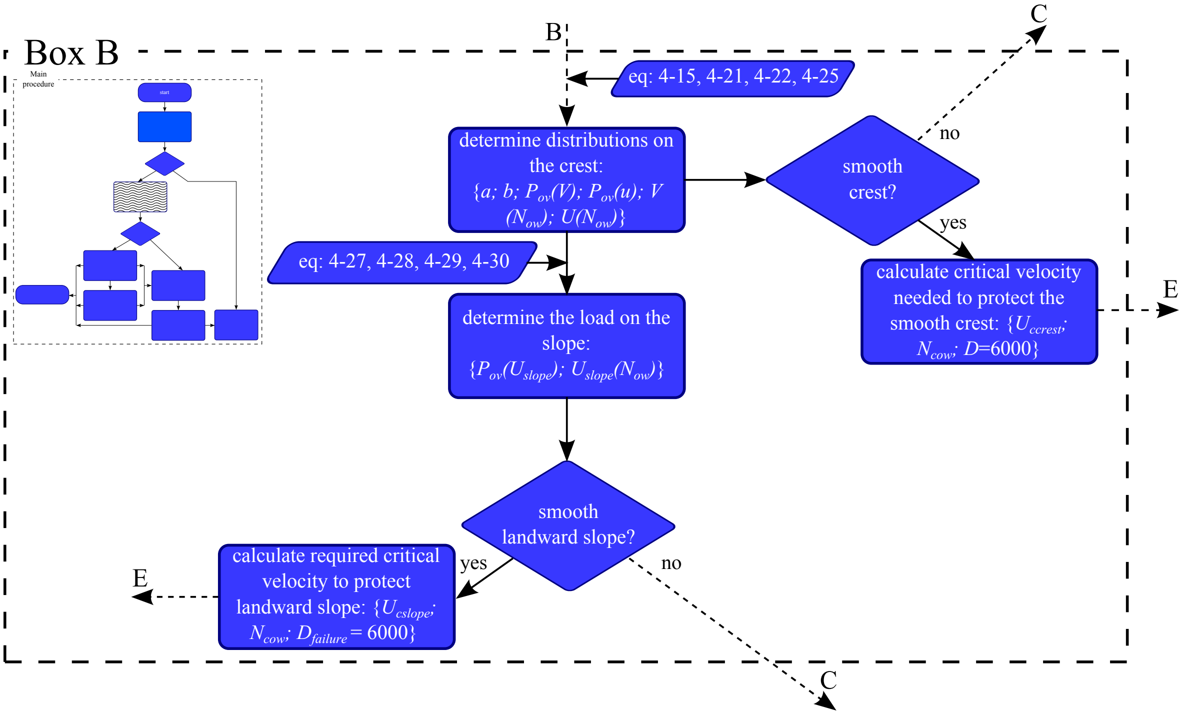

Figure 4-29: Process to determine flow velocity crest and slope

conventional f=0.01. The difference is approximately 1 m/s as maximum velocity along the slope. In accordance with the earlier stated 20-35 % the lowest value is more reasonable, that gives an increase of 34%. The resulting maximum flow velocity is 9.1 m/s with 6.3 m/s as example starting point for flow velocity at the crest as can be seen in figure 4-28.

The maximum value of each line is used as the overall maximum along the slope. This theory does not specify at which location along the slope the velocity is maximum. In addition to the above mentioned fact that the steady state assumption is not always valid, also the type of revetment is important for the influence on the flow on the landward slope. More information can be found in Pan et al. [2012] in which the influence of High Performance Turf Reinforcement Mats (HPTRM), Roller Compacted Concrete (RCC) and Articulated Concrete Block system (ACB) revetments on the discharge, flow depth and velocity has been investigated. HPTRM has the largest influence on the flow velocity and discharge, this is for surging overflow and combined overflow and wave overtopping, $0 < \frac{R_c}{H_{m0}} < -0.3$.

For the Afsluitdijk the choice has been made to calculate the velocity at the slope with the most conventional friction factor f of 0.01, because this value has been based on the largest amounts of tests. So the iteration described by equation 4-27 to 4-30 should be done for every overtopping wave in order to describe the distribution of maximum front flow velocity along the slope. This results in a distribution of front velocities of overtopping waves. If this distribution is known the choice is again whether or not there is a smooth slope. When there is a smooth slope this distribution should be used as the load. If not, additional steps should be taken. The process described in this section can be seen in figure 4-29. When the slope is smooth, this leads to **D**, which is described in section 4-6. If not the load can not be determined at this point and additional steps should be taken which is described in the next section.

4-5-4 Overtopping flow at critical points

The sections above describe how to get to a certain overtopping wave volume, resulting in a flow velocity and a flow depth out of the relative freeboard. However the flow depth and the flow velocity vary along the slope and these are influenced by the critical points. For each critical point an analysis has been made on what flow velocity, depth or other measure for load should be used. An attempt has been made to derive an amplification factor α_M for each critical point and for each categorization within these critical points.

Point of maximum velocity at the slope

The procedure in order to determine the load for the point of maximum velocity along the slope has already been described in the second part of section 4-5-3. This has been done because this is also a velocity without any interferences. The front velocity of the overtopping wave is only higher due to the fact that it increases while running down the slope.

Holes in the landward slope

The load is not so much increased if the hole is small enough. This is for example the case for bare spots. The strength of the top layer will be reduced but the load will not be notable increased. For a slope that is not perfectly smooth as defined in section 3-7 and only have small damages this can be solved by taken a allowable damage number $D_{failure}$ of 3500 m^2/s^2. However, when the hole becomes larger the load will be increased. The wear erosion prediction according to van Hoven et al. [2010] and other reports on the subject is given as equation 4-31, this is the so called erosion prediction model.

$$y_m = \frac{\sum_{i=1}^{n} (0.7\alpha U_m - U_c)^2 t_m}{E_{soil}} \tag{4-31}$$

The increase in velocity due to the increased turbulence, which sounds reasonable as a bear spot could increase the turbulence, is given in this equation as:

$$U_{max} = \alpha U_0 = (1.5 + 5r_0)U_0 \tag{4-32}$$

The factor r_0 gives the relative depth averaged turbulence intensity. The other parts in equation 4-31 are based on resilience of the slope. The lay out of the equation is based on the overload principle. This states thats:

$$\tau_0 - \tau_c \sim \alpha U_0^2 - U_c^2 \sim (U_0 - U_c)^2 \tag{4-33}$$

This is important because other theories are based on this shear stress and expressed as such. The above is actually more for erosion and formation of weak spots in the slope. So for bare spots and comparable damages. A erosion hole exists and after that, the erosion hole can erode downstream or upstream. The head cut erosion model can predict the speed at which the hole is growing in the upstream direction this is given by equation 4-34

$$dx/dt = C_{headcut}(A - A_c) \tag{4-34}$$

A is the hydraulic load and is given as

$$A = (qH)^{1/3} \tag{4-35}$$

H is the headcut height. The load is therefore depended on the depth of the hole and the wave overtopping discharge. This height can be calculated using 4-31. The growth of the hole in upstream direction seems to be not depended on the velocity but on the volume. Based on the above, the factor α_M might be based on the equation 4-36.

$$\alpha_M = 0.7 \cdot (1.5 + 5 \cdot r_0) \tag{4-36}$$

According to the evaluation report of the Afsluitdijk wave overtopping simulator tests, Verheij et al., r_0 will be between 0.2 and 0.4. This means that the factor α_M is between 1.75 and 2.45. This is a very large increase in the load that is caused by the overtopping wave. Because the method is still under investigation and a lot of uncertainties are present, the average value is used for the amplification factor $\alpha_M = 2.1$ for holes in the landward slope.

Transition between different dike segments

Depending on the kind of transition the load will be influenced or not. If there is a difference in height the flow is not only in cross sectional direction but may also flow perpendicular to the slope. This effect might be limited, the transition in height is often very gradual as currently is the case at the Afsluitdijk. When driving over the dike it is not very notable due to the gradual transition. The expected load increase will therefore be negligible at transitions in different heights for the current situation. As stated earlier a transition between different revetments on adjoining dike segments can occur. Currently no load increase or formulation has been made. This is probably because it is not very likely that such a transition in dike sections occurs. This is again confirmed by the fact that these kind of transitions are not tested up till now.

One kind of transition has been tested and this is at the Vechtdijk section 1, the Netherlands (van der Meer et al. [2010]). The transition consists of a driveway, the analogy with the transition between slope and berm is often used and seems reasonable, therefore for an increase in load see transitions between slope and berm. The transition can be paved or unpaved which means that at a paved transition the flow is additionally influenced by the presence of the pavement. For the influence on the load is referred to transition between different types of revetment. During testing it is concluded that not so much the load but especially the strength of the slope is influenced by the presence of the pavement (in this case so called ‘door groei stenen’).

Transition between slope and berm

The transition between the slope and the berm has been a subject of several researches. The load is increased due to the fact that the load is not parallel to the flow but has is a oblique incident flow. The resulting oblique incident flow can give larger forces. The analogy with jet erosion is mentioned in different researches. If a transition is gradual enough, then this mechanism does not occur. However, often the transition is not gradual and the load increases. According to one theory the load is expressed as Valk [2009].

$$\omega = (1.5 + 5 \cdot r_0) \tag{4-37}$$

$$\tau_0 = 0.016 \cdot \frac{1}{2} \cdot \rho_w \cdot \left(\frac{1}{1+\epsilon} \cdot U \right)^2 \tag{4-38}$$

$$\tau_0(d) = \tau_0 e^{-w \cdot d} \tag{4-39}$$

The shear stress as a function of the scour hole depth d and flow velocity U is given. The total load seems to be given as:

$$\omega^2 \cdot \tau_0(d) \tag{4-40}$$

At first sight this seems to be in accordance with the u^2 in the cumulative overload factor. A derivation to an amplification factor might be possible, but further research is needed.

Based on Hoffmans [2012] in Steendam et al. [2012a] an amplification factor has been derived. The scour depth is given as:

$$Z_{m,e} = f\left(U^{0.5}\right) \tag{4-41}$$

Because the amplification factor is related to u it can be given as:

$$\alpha_M = \frac{z_{2V}}{z_{2H}} \tag{4-42}$$

In which equilibrium scour depths related to 2D-H en 2D-V are given as:

$$z_{2V} = c_{2V} \cdot \sqrt{\frac{q \cdot U \cdot \sin\beta}{g}} \tag{4-43}$$

$$z_{2H} = c_{2H} \cdot \sqrt{\frac{q \cdot U}{g}} \tag{4-44}$$

This results in:

$$\alpha_M = \sqrt{\frac{c_{2V}}{c_{2H}}\sqrt{\sin\beta}} \tag{4-45}$$

c_{2v} and c_{2H} are both characteristic non-dimensional parameters for the soil strength.

$$c_{2v} = \frac{20}{\lambda} \tag{4-46}$$

λ is a dimensionless scour factor, which is depended on the d_{90}, the smaller it gets, the lower the value of lambda becomes and the larger the value of c_{2v} Hoffmans and Verheij [1997]. The values of a simple relation based on the d_{90} of the sand is a ratio of $\frac{c_{2V}}{c_{2H}}$, this is approximately 2 according to Steendam et al. [2012a]. This is valid for $0.1\ mm < d_{90} < 1\ mm$ and invalid for cohesive soils. Depending on the angle of the landward slope β the amplification factor varies from 1 to 1.5. because the angle of the slope is often written as $1V : xH$. with x the horizontal distance needed to drop 1 meter in height. the equation for the amplification factor α_M can be written as equation 4-47.

$$\alpha_M = \sqrt{2 \cdot \sqrt{\sin\left(\arctan\left(1/x\right)\right)}} \tag{4-47}$$

This results in the values for α_M as presented in table 4-5.

This formulation implies that for slopes with an angle less than 1:4 the increase in load is 1. So there is no increase, this could be the subject of additional research. If the angle of the slope is steeper than 1:4 then a transition slope could be designed which has a slope angle of less then 1:4 with both the horizontal and with the original slope. Probably the required length of this transitional slope is a function of the flow depth h. Because in this way an increase can probably be prevented it is worth investigating, the additional space required off course has to be available on the landward berm.

The question is whether this is valid or not if the revetment is not made from grass but from a different kind of revetment. Because the above formulation is depending on the soil parameters, this seems not to be the case. However, the load is still increasing due to the oblique angle of the flow. This is expressed in the factor formulated in equation 4-40. Also not taken into account here is the fact that the load reduces if there is water in the already eroded part of the hole. In Valk [2009] this has been taken into account. It would be a good to investigate see how this can be implemented in the amplification factor.

Table 4-5: Amplification factors transition slope to horizontal

slope angle	α_M
1:1	1.30
1:2	1.16
1:3	1.06
1:4	1.00

Transition between different type of revetments

This transition introduces an increase in turbulence and can give an increasing pressure under the downstream revetment. This is due to the passing wave front. Currently only the transitions between different types of revetment are only tested if these were present on a berm, as a driveway up the slope or as a maintenance road. No prediction formulation are currently known to predict the increase in load. Hoffmans [2012] has some formulations for the erosion downstream of a sill, however this is only for a transition from hard to soft en not from soft to hard. Also the formulation might not be valid because only large water depths are assumed. For scour protections often the factor α as used in equation 4-32 is used to calculate the maximum load. This seems a reasonable assumption. Another aspect when the transition is from soft to hard revetment is the pressure increase underneath such a revetment due to the wave front. This can cause uplift or undermining of the revetment. The above described has been summarized in the procedure depicted in figure 4-30.

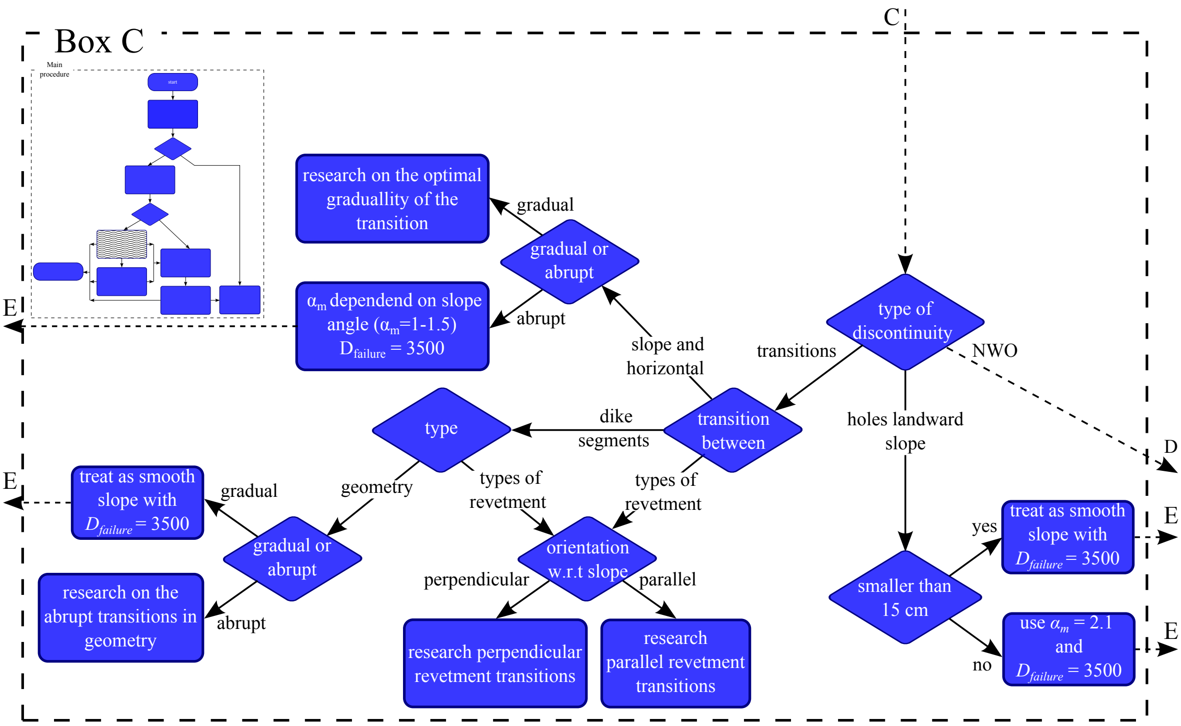

Figure 4-30: Procedure to determine load amplification based on discontinuities U_c

Non-water-retaining objects

The loads on NWO's can be described in two ways, the first is the increase in velocity and turbulence and the other is the force on the NWO it self. Due to the wave a certain force will be exerted on the NWO, which might cause failure of the NWO. The focus of the thesis is on failure by erosion or related failure mechanisms, therefore the failure of the NWO it self will not be taken into account. Only the load induced by the presence of the NWO's will be considered. NWO's have a lot of different shapes and forms, therefore the loads on the different NWO's differ a lot as well.

As a starter, objects such as trees and different poles often have a round shape. The analogy with scour around a bridge pier is a obvious one. Prototype tests have shown that small trees with a diameter less than 15 cm gives negligible erosion. However if the diameter increases, it gives an increased load. Because of the large varieties in NWO's the choice has been made to use the categorization as presented in section 4-4:

- Vegetation
- Structures
    - Buried
    - Buried but visible
    - External
- Cables and pipelines
- Poles
    - Poles with foundations
        - <15 cm
        - >15 cm
    - Poles without foundations
        - <15 cm
        - >15 cm

Because the NWO's appear in such a large variety the choice has been made to treat it as a separate group within the discontinuities. In Trung [2012] a 1/15 slope with a rounded obstacle and a square obstacle was tested with a wave overtopping simulator. Only limited amounts of wave overtopping were tested at the rounded obstacle, which wasn't enough to conclude anything about the damage around such objects. Nevertheless some interesting observations were made, in front of the large obstacle flow was blocked and damage occurred, while along the object no damage occurred. The latter is remarkable because the expectation is that the flow would increase along the object and cause damage. For the square object damages occurred after 10 hours for discharge from 60 to 100 $l/s/m$. The damage was on both sides of the obstacle and were extended slowly, but they were still limited. In addition to these observation a research about the influence of structural transitions was described in Pijpers [2013]. Here a numerical ComFLOW model was made in order to describe the load around such objects. What was seen in the tests as described above is that the flow velocity along a round structure did not increase that much. The main load increase was caused by the blockage of the flow in front of the rounded object. He describes two zones were initiation of erosion occurs: Zone A In front of the object on the upstream side, not caused by flow velocity but due to an increase due to impact on the structure and zone B next to the object, just after the corner of the square object, this is due to flow concentration. See figure 4-31. Also the presumption is made that zone B is not of an influence given an object with a round shape. This is based on tests and on the velocities shown by the model. However, for square objects an increase of the damage was found. Also an increase in damage has been found for objects, therefore the choice has been made here to choose an amplification factor in analogy with different literature.

Figure 4-31: Erosion zones A and B around objects Pijpers [2013]

This leads to one of the solutions in order to take into account the additional damage round objects. This is to use the amplification factor α_M to give an increase of the load due to the NWO's. Depending on the type of flow, turbulent or laminar the flow velocity alongside an object is twice as large as the flow velocity in front of the object or less than twice, respectively. Due to the fact that in the case of an overtopping wave the flow is considered to be turbulent the factor $\alpha_M < 2$. A relative simple method to derive this by continuity in which no energy loss is assumed Steendam et al. [2012a]:

$$U \cdot l = \alpha_M \cdot U \cdot (l - b) \tag{4-48}$$

in which l is the width of the test section and b is the width of trees or other obstacles. This gives:

$$\alpha_M = \frac{l}{l - b} \tag{4-49}$$

With a width of a tree of 0.5 m and a test sector of 4 m wide this results in a α_M of 1.15. On the basis of earlier research and the fact that the amplification factor also depends on the shape another relation is given by equation 4-50.

$$\alpha_M = 1.2 \cdot K_{shape} \tag{4-50}$$

Assuming the shape factors as presented below in table 4-6 according to Schiereck [2001] this leads to an resulting amplification factor α_M that varies between 1.0 to 1.5 based on the formulations above.

Table 4-6: Shape and amplification factors for various shapes (*it is recommended to use 1 it is not expected that α_M will be lower than 1)

shape	$\frac{length}{width}$ $[-]$	K_{shape} $[-]$	α_M $[-]$
cylinder	-	1.00	1.20
rectangular	1	1.20	1.44
	3	1.10	1.32
	5	1.00	1.20
elliptic	2	0.85	1.02*
	3	0.80	0.96*

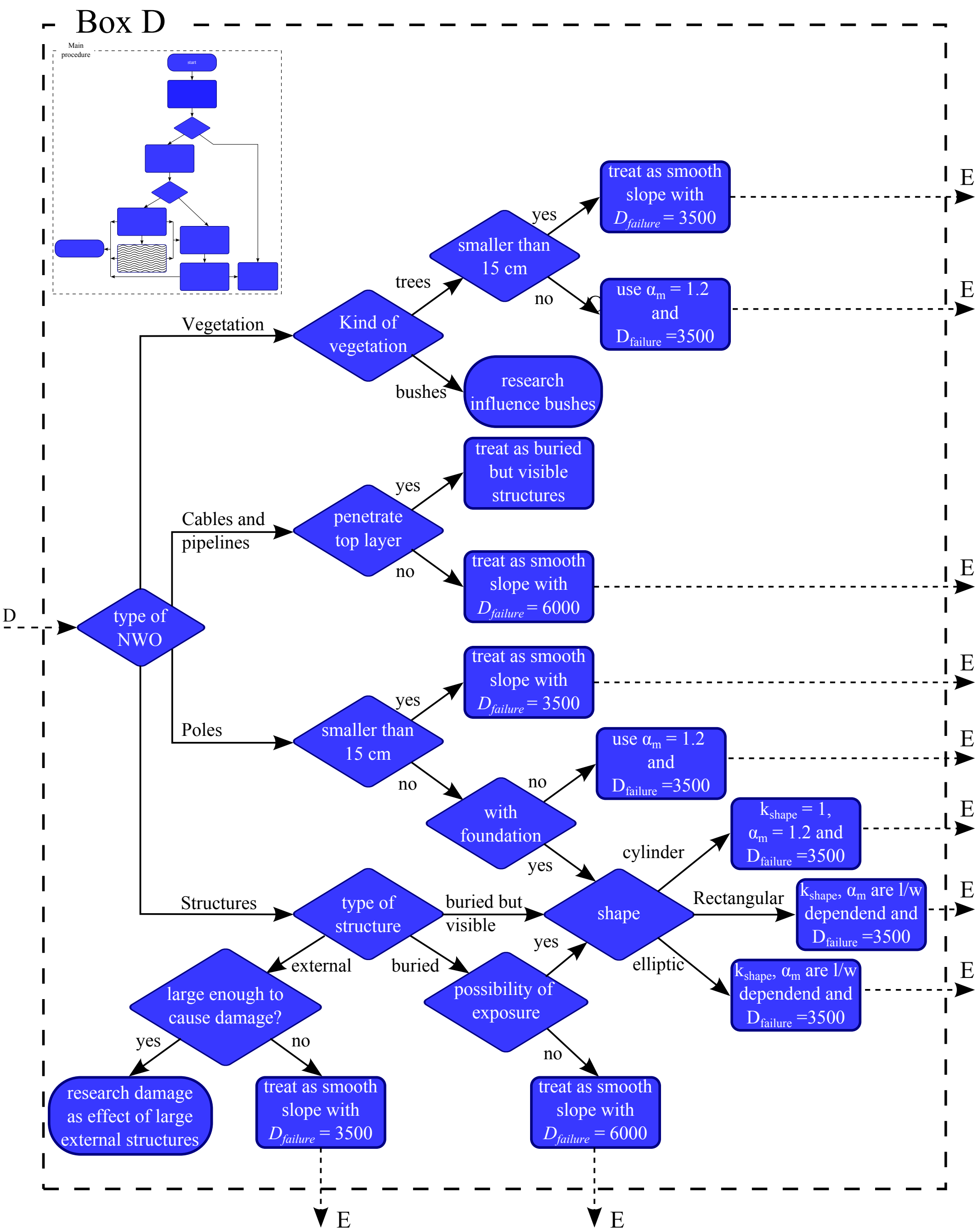

Figure 4-32: Procedure to determine required critical velocity U_c

Concluding remarks

As the formulations in this section show the wave overtopping volumes, flow velocities and depths as a function of the wave conditions and outer dike specifications can be determined quite well. This is especially on the dikes crest. The velocity development on the landward slope can be estimated with the crests overtopping values as initial conditions, this is however under the steady state assumption and gives a good representation of the maximum flow velocities. These are depended on the friction of the landward slope which has been estimated based on a few tests and literature recommends various values for this friction factor. Near obstacles and transitions the increase in velocities is currently difficult to calculate, the amplification factor can be estimated on the bases of theory. However, these are currently often first attempts to make a prediction on the velocity increase for the full scale testing with the overtopping simulator. The reliability of the calculated load increase at the different transitions and weak spots along the slope is therefore not very good at this moment. The question that now should be asked are the above mentioned methods reliable enough to really prove the load at the landward slope. This is the case for the load at the crest and if the friction factor is well chosen also for the maximum velocity along the slope. For the other critical points this is doubtful. The remark made at the beginning of the thesis on the fact that the loading conditions are expressed as flow velocities and depths per wave is important to keep in mind. The amounts of times that a certain velocity is exceeded is even more important than the maximum velocity.

4-6 Load to resilience

The load as determined in section 4-5 should be used in order to determine the required strength of the landward slope. The load is expressed as a the number of overtopping waves during a storm and a certain velocity distribution for these waves. Due to the fact that overtopping is a pulsating non steady state process the required strength, expressed as the critical velocity U_c can be calculated with the cumulative overload factor, or sometimes called the damage number D, the load and the strength can be coupled based on shear stress.

Due to the pulsating character the load on the landward slope is not time depended but expressed as the number of times a certain critical velocity (or actually shear stress) is exceeded, the principle of fatigue. Each wave that exceeds this velocity should be taken into account, the waves that do not exceed this velocity do not contribute to the damage.

Because the shear stress is used in the cumulative overload factor, the velocity should be squared. The limiting velocity or otherwise called the critical velocity U_c should be squared and be subtracted from each wave velocity squared to see what the 'contribution' of each wave is to the damage. This looks like the equation that is also mentioned in section 2-2:

$$D = \sum_{i=1}^{N_{cow}} \left(U^2 - U_c^2\right) \tag{4-51}$$

The parameter N_{cow} is the number of critical overtopping waves, defined as the number of overtopping waves that result in a larger velocity U than the critical velocity U_c. The load calculations result in a distribution for the number of waves that exceed a certain velocity. However, a certain critical velocity U_c is required to protect the landwards slope. To calculate this critical velocity the acceptable damage number D should be known. This damage number has been divided in four categories van der Meer et al. [2010]:

Initial damage	$D = 500\ m^2/s^2$
Damage at several locations	$D = 1000\ m^2/s^2$
Failure (for damaged slopes by mole holes)	$D = 3500\ m^2/s^2$
No Failure (for a smooth slope)	$D < 6000\ m^2/s^2$

The above values will be used in order to calculate the required critical velocity U_c. The damage number of 6000 m^2/s^2 will be used if the slope is smooth. If there are some kind of small 'negligible' damages the value of 3500 m^2/s^2 will be used. The values for initial damage and damage at several location can also be used, but this is not the kind of damage of interest for this thesis. Depending on

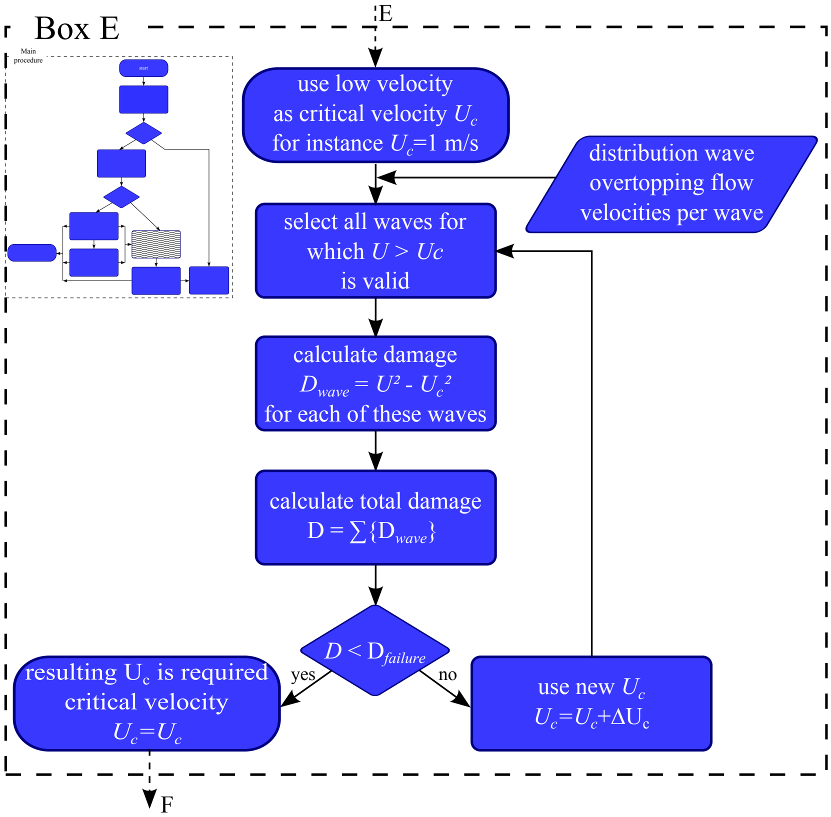

Figure 4-33: Procedure to determine required critical velocity U_c

the damage number the required critical velocity can be determined as well as the number of critical overtopping waves. This is an iterative process because the number of waves that contribute to the damage number is depended on the required critical velocity. Only the waves that result in a higher velocity than the critical velocity contribute to the damage. This is done by assuming a low critical velocity and calculating the damage number by adding each wave that contributes to the damage number.

So if the wave results in a higher velocity than U_c, it is used in the equation 4-51. When the damage number is larger than the value 3500 or 6000 m^2/s^2, to much waves have contributed to the damage and a higher critical velocity is assumed. This is done in very small steps until the damage number for failure has been reached. The number of waves that contribute to the critical velocity is known as well as the required critical velocity. With this number of waves the percentage critical overtopping waves can be calculated, which is here defined as equation 4-52.

$$P_{cow} = \frac{N_{cow}}{N_w} \tag{4-52}$$

This is the probability that a random incoming wave contributes to the damage at the landward slope. For the crest and for the slope this has been done. The process described here can be seen in figure 4-33.

The process starts by choosing a low critical velocity U_c, for step two the distribution for the resulting velocity U per overtopping wave is required (figure 5-19 for instance), select all the waves that result in a higher velocity than the chosen critical velocity. Step three is to calculate the damage for each of these waves with $U^2 - U_c^2$. Step four is to add the damages caused by each wave to get the overall damage number D. The last step is to see whether or not the overall damage number D is smaller than the used allowable damage number $D_{failure}$ (for instance $D_{failure}$ is 3500 m^2/s^2). If it is smaller than this is the required critical velocity for the landward slope. If it is larger a higher U_c should be used. This is done by adding a small ΔU_c to the U_c resulting in a new U_c. With this new U_c the cycle is repeated from step two on as long as it takes to reach $D < D_{failure}$. After this the required critical velocity U_c is known.

The procedure can be adapted in order to take into account the load increase due to objects or other discontinuities. This can be done with the already in equation 4-1, Steendam et al. [2012a] described adaptation of the cumulative overload factor. The formula is repeated here:

$$\sum_{i=1}^{N_{cow}} \left((\alpha_M U)^2 - U_c^2 \right) \tag{4-1 revisited}$$

What is important is that instead of taking the velocity U as front flow velocity in the procedure, the U should be replaced with $\alpha_M \cdot U$. The values recommend for α_M have been described in section 4-5-4.

The critical velocity which the landward slope should be able to handle is known for the different critical points after completing the procedure presented here. How can this required critical velocity be realized and what is the influence on the strength of the critical points are the next questions that should be answered.

4-7 Resilience

With the methods presented in section 4-6 a required critical velocity U_c has been found. This can be seen as the required resilience of the landward slope top layer. The question for the next part is how can this required resilience be determined. The critical velocity U_c can best be determined based on tests done in the past and testing in the field for grass. The measure to prove the resilience is most likely to be found in certain analogies, these are or can be: Jet erosion, spillways, scour protections, revetments, run-up and run-down. For the different critical points other aspects determine the resilience against erosion, already shortly described in the critical point section. For each critical point the resilience is described. Currently most of the testis on wave overtopping have been done on grassed landward slopes of dikes and not on other types of revetments. The resilience depends on the type of revetment applied to each dike segment and on the reduction or increase in strength at different critical points. One of the questions is *In what parameter can strength be expressed?* It often is expressed as a critical velocity or shear stress, however that is for grass. If the limits of grass will be exceeded another revetment should be applied. However, for other revetments almost no tests have been done. For testing this often is done in a certain discharge q which the slope is able to resist.

The overtopping waves can be discharged in two ways, or by minimum energy loss or by energy dissipation. These are contradictory concepts and each have advantages and disadvantages. Both solutions are also applied to spillway design. The latter option means reducing the load via energy dissipation. This is done in spillway design by applying some kind of stair-like revetment, for example articulated block systems. The load on the revetment itself increases but the load on the transition decreases. The first option means that the revetment does interfere in a minimal way with the water flow and let the water flow away as smoothly as possible, it is called minimum energy loss weir design. The spillways and the lakes currently have a somewhat similar problem as the Afsluitdijk, due to increasing discharges the water retaining capacity of reservoirs in insufficient. In addition to this the total design capacity of the reservoirs and spillways is insufficient to handle the amount of inflowing water. Currently there are three possible solution to increase the combined capacity of the reservoir and the spillway:

- Increase the capacity of the reservoir by heightening of the embankments
- Increase the capacity of the spillway
- Use the embankments as emergency (auxiliary) spillways

The last option can be compared with the overtopping resilient Afsluitdijk. The embankments are often earthen embankment dams. Therefore not designed to be able to withstand large amounts of water over flow. In order to reach this the same challenges arise as for the Afsluitdijk, the only difference is that often here it is overflow and not wave overtopping. This means that there is not a wave front passing with each wave, but a more continuous flow occurs. Reinforcement of the embankment is still required, at some locations this has already been done. The reinforcement of the embankment is done in a couple of ways:

- Vegetative barriers
- Geosynthetics
- ACB
- RCC
- Stepped spillway

The main difference between spillways and the application to dikes is that a spillway is only a small part of the embankment, while a dike has to be strengthened over the entire length. This leads to the next difference between wave overtopping and overflow, the position of a spillway can be selected in such a way that a minimal amount of objects are present at the location of the spillway. In case of the Afsluitdijk this cannot be done. The last important difference is that spillways often only need to resist overflow and not wave overtopping or a combination of overflow and overtopping. Also in spillway design a critical velocity is used for the different types of protective systems. The pulsating character of wave overtopping is therefore not taken into account. This character makes the load on the revetment harder to cope with due to different directions of forcing. Often the uplift is the force that makes the protective system fail. Due to the differences stated above the design of spillways and an overtopping resistant dike is hard to compare.

The problem in spillway design is different in a lot of ways, the main comparison that can be made is that an earthen embankment has to be able to withstand large amounts of water. However, the design practice is not useful for the design of overtopping resistant dikes.

4-7-1 The resilience for each critical point

U_c for the point of maximum velocity at the landward slope

Depending on the top layer on the landward slope the critical velocity should be determined in a different way. The current top layer consists of a layer of clay with on top of that a grass layer, for which a lot of research is done on the resilience. According to Hoffmans [2012] the critical velocity, and thus the resilience of grass can be determined by:

$$U_c = \alpha_{grass,u} r_0^{-1} \sqrt{\psi_c(\sigma_{grass,c}(0) - p_w)/\rho} \tag{4-53}$$

In which:

$$\alpha_{grass,u} = \alpha_0 \sqrt{1 + 3\alpha_{grass}} = 2.0 \tag{4-54}$$

For $\alpha_{grass} = 0.64$ and $\alpha_0 = 1.2$

$$\sigma_{grass,c}(0) = \frac{A_{root}}{A_1}\sigma_{root} \tag{4-55}$$

$$r_0 = 1.2\sqrt{gh_m(1-\eta_a)S_b}/U_m \tag{4-56}$$

η_a is the air concentration obtained from measurements, U_m is the depth averaged maximum velocity, h_m is the maximum flow depth and S_b is the sinus of the angle β of the slope and the horizontal. Equation 4-53 can also be used for clay, instead of $\alpha_{grass,u}$ and $\sigma_{grass,c}(0)$, α_0 and $C_{clay,c}$ are used, respectively. The latter is given as a function of the cohesion c:

$$C_{clay,c} = 0.6C_{f,m} = 0.02c \tag{4-57}$$

some indicative values for the allowable critical flow velocity U_c for different grass qualities and with and without water under pressure p_w are shown in table 4-7, based on Steendam et al. [2012a].

Table 4-7: Critical flow velocity grass

grass quality	$\sigma_{grass,c}(0)$ kN/m^2	U_c (with p_w=0) m/s	U_c (with p_w=-10) m/s
very poor	<3.0	<3.0	<6.2
poor	3.0-5.3	3.0-4.0	6.2-6.8
average	5.3-7.5	4.0-4.7	6.8-7.2
good	>7.5	>4.7	>7.2

When a different type of revetment appears to be necessary, the equation presented above might not be useful anymore. These are for cohesive soils. Other formulas applied for different revetment types, which normally can be found on the outer slope can for instance be interesting to consider. The overtopping wave front is the most interesting part because the highest velocities occur at the front, it induces a rapid pressure increase, and a more gradual drop. An analogy with waves that run-up or run-down might be interesting. However on the sea side of the dike this is not the most critical parameter and a wave front does not occur with run-down.

The cumulative erosion (maximal) on the landward slope (the slope it self) is given by van Hoven et al. [2010] as:

$$y_m = \frac{\sum_{i=1}^{n}(0.7\alpha U_m - U_c)^2 t_m}{E_{soil}} \tag{4-58}$$

In which

$$E_{soil} = 6.15\cdot 10^4 \frac{U_c^2}{\sqrt{g d_a}} \tag{4-59}$$

y_m	maximum erosion depth	$[m]$
t_m	overtopping duration	$[s]$
d_a	0.004	$[m]$
E_{soil}	erosion parameter	$[m/s]$
α	turbulence constant ($= 1,5 + 5r_0$)	[-]

The combination of the factor α is based on prototype test, but it could be an improvement to use the U_c as stated above.

Transition between different dike segments

The driveways up the slope is the only dike segment transition that has been tested with the overtopping simulator (van der Meer et al. [2010]), the failure was kind of the same as with the transition from slope to berm. Solutions might also be found in smooth transitions in order to prevent flow concentration. Currently there is no measure for when this transition is smooth or when it is not. Testing with asphalt also has been done at a dike (Van der Meer et al. [2008]), at the edge of the asphalt (thus at the edge of the test section) where no further reinforcements were made, some damage occurred to the asphalt layer which shows that the edges also in the cross sectional direction with respect to the dike are a weak spot. The testing was not meant to test this damage but on how asphalt performs during large amounts of wave overtopping. It did not fail for amounts up to 125 $l/s/m$.

Transition between slope and berm

The resilience for the transition can be found in the top layer and is currently expressed as the thickness of this layer. If the equilibrium depth of the scour hole reaches the core of the dike the top layer has failed. If the transition is gradual enough then it is most likely not a critical point anymore, however how gradual this has to be is still the question. This is also a self protecting phenomena, because the erosion hole will be (partly) filled with water, the jet will be dampened by this water in the hole. This effect reduces the load on the remaining part of the top layer. For the development of the erosion hole at the toe or at a driveway, three methods are mentioned in van Hoven et al. [2010]. The method Hoffmans, Stein and Valk. The method of Valk is not used because it is very comprehensive. Method of Hoffmans is given by:

$$y_{m,e} + h_t = U_{DL}\sqrt{sin(S)\frac{qU_m}{g}} \tag{4-60}$$

with

$$U_{DL} = \frac{23}{\left(U_c\left(\frac{\Delta}{vg}\right)^{1/3}\right)^{1/2}} \tag{4-61}$$

Stein is given by:

$$y_{m,e} = \frac{C_d^2 C_f \rho_w U_m^2 h_t}{\tau_c} \sin S \tag{4-62}$$

Options for increasing the strength at the location of the transition

- Increase the thickness of the clay layer on the horizontal part of the transition, this is also proposed in Kruse [2013]
- Gradual transition with a certain radius in order to reduce the hydraulic jump, for instance a radius of 9 m has been applied for the Noordwaard project de Vries and de Bruijn [2010].

Non-water-retaining objects

The resilience due to these objects does not necessarily have to be influenced negatively. With trees the assumption is made that the grass cover strength is not influenced by the tree. However, due to shadow effects in reality it often is influenced. In van Hoven et al. [2010] the NWO's are divided into different turbulence classes. From low to extreme high. Also the analogy with other erosion formula's has been made.

As shown it is difficult to determine the resilience per critical point. In what way the strength is influenced currently cannot be determined. Only for plane grass, with the right available data, the

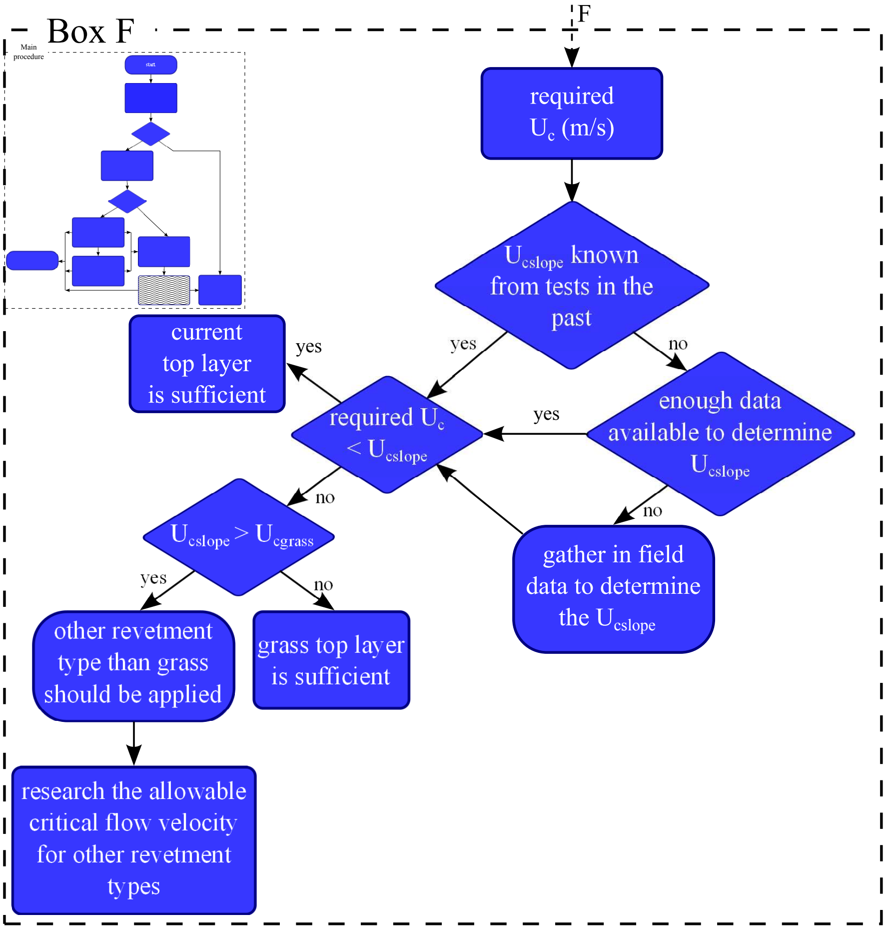

Figure 4-34: Procedure to translate critical velocity into required top layer

critical velocity of the slope can be determined. For other types of revetment, additional research is required as well as for the different critical points. Because the amplification factors of the loads are partly based on tests, a certain reduction of the strength is therefore all ready taken into account. Currently the critical velocity of the slope U_{cslope} for several dikes in the Netherlands is known based on tests. As well as some criteria for the allowable critical velocity for different species of grass. The commonly used method to determine the allowable critical flow velocity of a dike is to see whether there are some old test available from which this velocity can be extracted. Otherwise the data can be gathered from testing with a grass tension devise or be based on the number of roots within a certain area. If this appears to be higher than the required U_c caused by the overtopping wave this grass layer will be a good layer to protect the slope. If this U_{cslope} is lower, the grass quality can be improved or when this is not possible the grass layer isn't sufficient for the load. If this is not the case than another type of revetment is required and additional research has to be executed. This analyses is summarized in figure 4-34.

Concluding remarks

The wave overtopping resilience of the crest and landward slope is currently hard to determine with other methods than testing for other revetments than grassed landward slopes. Most likely the best analogy can be found in emergency spillways, for other revetments than grass. However, these are only limited applicable due to the steady state overflow, avoidance of NWO's and a limited stretch of embankment. It has become clear during the above study that the overtopping resilience of grass can only be proven up to a limited amount of wave overtopping ($<75\ l/s/m$). For asphalt and elastocoast the limits were larger, up to at least 125 $l/s/m$. If a certain amount of wave overtopping is exceeded the theoretical methods as presented in this chapter, on paper seem like good tools to calculate the load and the load increase due to objects. However, in practice these methods appear difficult to apply but give a good approximation nevertheless.

Chapter 5

Results

The results gathered with the executed methodology will be presented and described in this chapter. As stated in section 1-2-2 the goal of the thesis is to develop a design procedure to prove the resilience against wave overtopping of the Afsluitdijk. The results of the conducted methodology as described in chapter 4 consists of three parts:

1. The design procedure to prove wave overtopping resilience
2. Application of the design procedure to the case of the Afsluitdijk
3. Required investigations in order to complete the design procedure

In this chapter the available theoretical methods and the proposed procedures in order to prove the resilience of the crest and landward slope against large amounts of wave overtopping will be presented. The procedure tries to find the relation between the sea state and the strength of the landward slope. The second step is to apply this procedure to the case of the Afsluitdijk to see whether or not the procedure actually makes any sense, this theory only makes sense if it can be applied. This will be done right after the description of each sub-procedure or here called 'Box'. The procedure has some open endings, this means that additional information is required in order to be able to get enough information to fully complete the procedure. In other words, what should be investigated to obtain this additional required information, this can be found in chapter 6. These investigations are a part of the proposed design procedure.

5-1 Design procedure proving wave overtopping resilience

The resulting design procedure is a comprehensive scheme, therefore the choice has been made to first present a main scheme, which does not show all the details. This main scheme will be decomposed in the different boxes, or steps, it consist off. For each box a separate procedure has been composed in order to be able to show all the details.

The procedure as derived in chapter 4 and described in section 5-1-1 to section 5-1-7 is meant to be applied on the design of the overtopping resilient Afsluitdijk. All the formulas and methods are useless if these could not be applied to a real case. Because the derivation of the procedure is also based on the case of the Afsluitdijk this section and the methodology described in chapter 4 have been executed parallel to each other. The results therefore are partly sub results and partly final results. The presentation of the sub results has been done because this gives a better insight in the derived procedure. Also the touch and feel for the numbers involved is one of the goals of this research.

5-1-1 Main Design Procedure

One of the main results of the conducted methodology is the procedure presented in the flowchart depicted in figure 5-1. The different boxes shown in chapter 4 are put together in the procedure presented in this section. The Main Design Procedure consists of six different boxes (A to F), has a

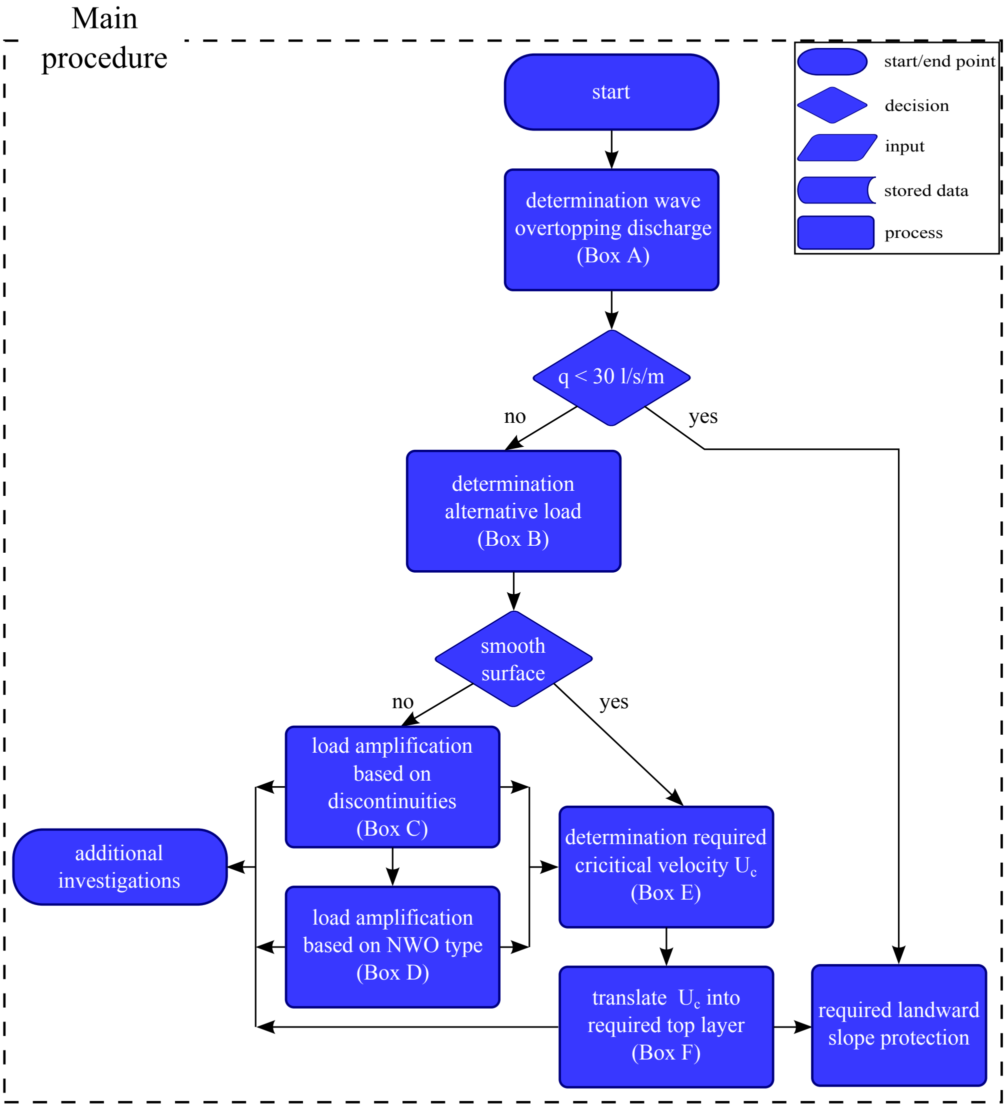

Figure 5-1: Main Design Procedure

starting point, two choices and two possible end points. The first step as described in Box A, section 5-1-2, is to determine the overtopping discharge q in $l/s/m$. This choice has been made because from an average overtopping discharge q of 30 $l/s/m$, the average wave overtopping discharge q appears not to be a good method in order to determine the required landward slope protection. When the calculations of Box A appear to result in a lower value of q, a required landward slope protection can be determined. If not Box B should be used.

The second box of the Main Design Procedure is Box B, section 5-1-3, in which the alternative load is determined, this load appeared to be the front flow velocity of the overtopping wave at the crest or at the landward slope, distributed over the overtopping waves. If the slope and crest are smooth, so no discontinuities are present, this is the load at the dike. However, if the slope is not that smooth additional steps have to be taken.

These additional steps are described in Box C, section 5-1-4. The load increase due to the presence of the different discontinuities is described in this box. This increase depends on the type of discontinuities that are present. For some types the load increase can be calculated with an amplification factor α_M, resulting in a final load (the increased velocity U). Others may lead to additional investigations as will be described in chapter 6, these investigations are open endings of the Design Procedure. For the Non Water Retaining Object(s) (NWO)'s a separate procedure has been created because there is a large variety within this category. Summarizing, Box C can lead to three steps: additional investigations, a load which can be used to determine required critical velocity U_c and to additional steps as described in Box D for the NWO's.

The steps required for the large variety of NWO's as described in Box D, section 5-1-5, are based on the categorization as described in appendix C. This box can lead to two different steps, the first is additional investigations to fill in the open endings the second is the increased load U per wave that can be used to determine the required critical velocity U_c as is described in Box E. For each category the attempt has been made to derive the amplification factor.

Box E, section 5-1-6, is used to calculate the required critical velocity U_c based on the cumulative overload factor, which is based on the number of times that a certain shear stress is exceeded. Based on the allowable damage $D_{failure}$ the required critical velocity can be determined with the use of the final loads resulting from Box B, Box C or Box D. This box, Box E, will always result in a required critical velocity which can be used to determine the required protection of the landward slopes top layer as can be found in Box F.

Box F, section 5-1-7, translates the required critical velocity U_c in a certain protection for the landward slope or crest. Box F can have two results as well, additional investigations or a resulting landward slope protection. With this the Main Design Procedure has been completed.

The details of the above described can be found in the sections below together with the application of the design procedure to the case of the Afsluitdijk. In each box that is described in the sections below, the main design procedure can be found in the top left corner of the box. The location of the box under consideration has been hatched in the main design procedure.

5-1-2 Box A: determination wave overtopping discharge

The first step of the main design procedure of figure 5-1 is the determination of the average wave overtopping discharge (Box A). This box of the design procedure consists of several parts it self. The detailed version of Box A is shown in figure 5-2. It takes the input parameters (for the sea state as well as for the lay-out of the dike) and with these determines the run-up 2% level in order to see whether or not significant amounts of wave overtopping will occur. These calculation can be done using equation 4-2 to 4-10 as described in section 4-5. If this $R_{u2\%}$ level does not exceed the crest height, no significant wave overtopping will occur and no measures are required to strengthen the landward slope of the dike. If the crest height is exceeded, the overtopping discharge has to be determined. This step has been implemented because if this level is not exceeded, no other calculations have to be done which reduces the amounts of work. First of all, the probability P_{ov} of an overtopping wave can be determined as well as the number of overtopping waves N_{ow}. This is an important number because only the overtopping waves can cause damage to the landward slope.

The next step is to determine the overtopping discharge, this is done by calculating the dimensionless average wave overtopping Q and accordingly calculate the average wave overtopping discharge q. The equations required to calculate these parameters are shown in section 4-5-2, equation 4-12, 4-13 and 4-14. If the average wave overtopping discharge q is known, certain choices can be made. If q is smaller than 0.1 $l/s/m$ no protection of the crest or landward slope is required. If q is in between 1 and 10 $l/s/m$ a protection with a grass covered clay layer will be sufficient. If q is between 10 and 30 $l/s/m$ a well maintained grass covered clay layer is required in order to protect the landward slope. Till this point the procedure is convenient. However, if q is larger than 30 $l/s/m$ the strength of the grass covered clay layer might not be sufficient any more. Some test showed damage above this value Van der Meer et al. [2009] and van der Meer et al. [2011]. Therefore a different measure for the load should be used. This leads to the next box: Box B.

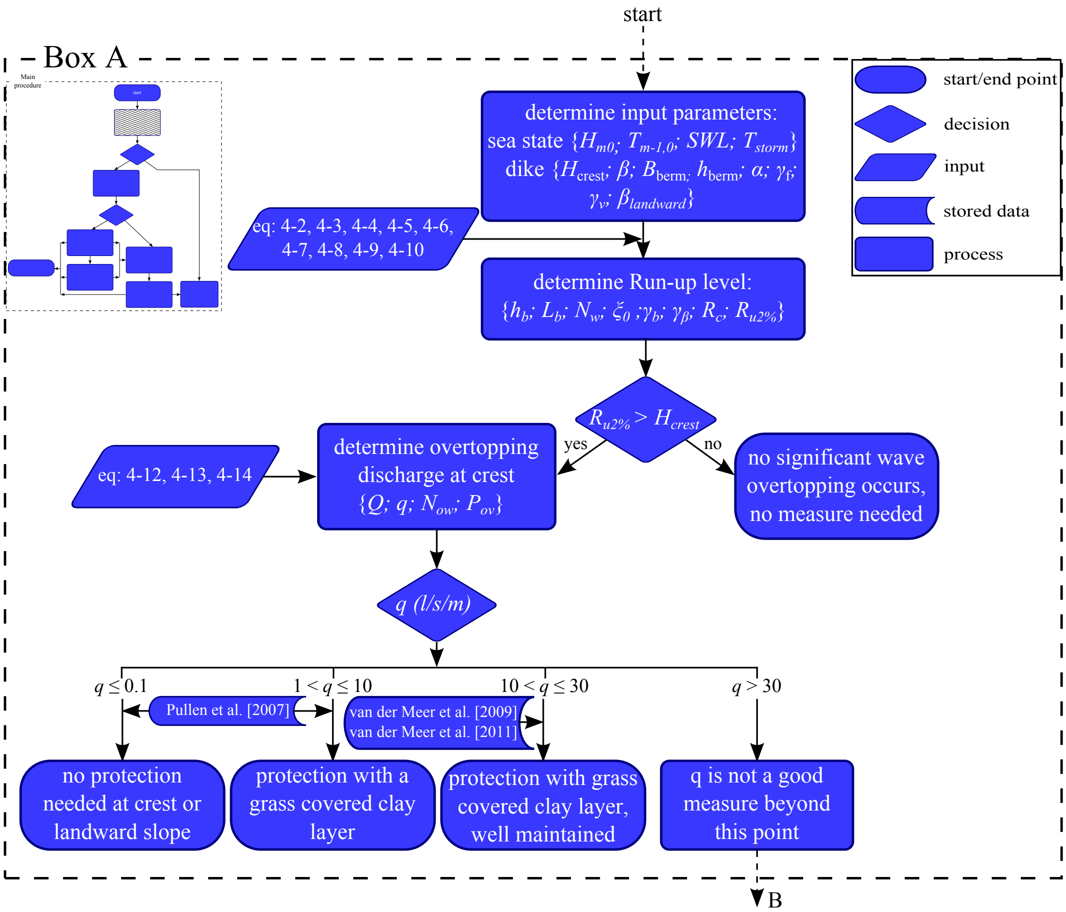

Figure 5-2: Box A: procedure for determination wave overtopping discharge

Application of Box A to Afsluitdijk

The presented procedure in Box A uses formulas and theoretical formulations, partly based on tests. To get a feeling for the numbers and figures that result from these formulations an applied calculation, for the Afsluitdijk, using the procedure has been made. The first step is to determine the input parameters. These can be divided into parameters that describe the sea state and the lay-out of the dike. The formulations used are depended on the sea state at the toe of the dike, this should be the location at which the input parameters are determined. In dike design often these parameters are known or are design variables. These design parameters can be influenced. Because it appeared that the seaside revetment of the dike was insufficient as well, this has become more important for the case of the Afsluitdijk. In addition to this also the length effect can be important here, currently different sections with different cross sections are present at the Afsluitdijk. For the input parameters the current lay-out of the Afsluitdijk is used in combination with the hydraulic boundary conditions. These have been determined in Deltares [2013] as described in section 4-1. The properties of the normative cross section of the Afsluitdijk can be found in appendix B, figure B-1 is used in combination with the governing conditions of 1/10 000 per year for dike section 8/9. The input is summarized in table 5-1.

Table 5-1: Input variables, given 1/10 000 year

symbol	description	value	unit
B_b	berm width	6.92	m
g	gravitational acceleration	9.81	m/s^2
H_{crest}	crest height	7.84	NAP+m
H_{m0}	zero-th order spectral wave height	3.92	m
H_{berm}	berm height	-0.25	NAP+m
H_w	still water level	5.28	NAP+m
T_{storm}	storm duration	6	h
T_m	mean wave period	6.3	s
α	slope of the revetment	1:3.5	-
β	1. angle of attack	18	-
	2. landward slope angle	1:2.7	-
γ_f	reduction factor permeability and roughness	1	-
γ_r	reduction factor roughness	1	-
γ_v	reduction factor vertical wall	1	-

The additional data required for further calculations is calculated according to equation 4-2 to 4-9 together with the input parameters shown in table 5-2. A quick look at the resulting parameters give a few results. With the value of the breaker parameter, or here called Iribarren number, ξ_0 the waves are breaking on the slope of the Afsluitdijk, the breaker type is a plunging breaker.

The reduction factor for roughness γ_r, berm γ_b and angle of incoming waves γ_β are all between 0.9 and 1, which means that there is hardly any reduction in the run-up level due to the geometry of the dike. This is due to the small incidence angle of the waves of 19 degree. The berm has little effect because the berm is relatively short with respect to the wave length and the water column on top of the berm is 5.53 m and therefore the waves don't 'feel' that much of the berm. This isn't strange taking into account that the Afsluitdijk was never designed to be able to handle the 1/10 000 year conditions used for these calculations. The resulting run-up level $R_{u2\%}$, calculated with equation 4-10, is far above the normative water level with a value of 6.63 m. With a freeboard R_c of 2.56 m the crest level is exceeded by the run-up level which means that wave overtopping occurs.

In order to get a clear view of all the input parameters required to perform calculations on wave overtopping, a visual presentation of the (calculated) input parameters presented in table 5-1 and table 5-2 can be seen in figure 5-3.

To continue with the procedure the wave overtopping calculations should be made. This has been done with equations 4-12, 4-13 and 4-14. To see what the maximum overtopping discharge q is a calculation based on equation 2-1 has been done, which gives a maximum average wave overtopping discharge and does not take into account all the available parameters. Therefore the results from equation 4-13 are more reliable and should be used here.

Because the freeboard divided by the run-up is a small, the probability of wave overtopping is large. Over half of the incoming waves during a storm are likely to overtop the Afsluitdijk, P_{ov} is 0.56. This means that approximately 1914 waves will overtop the dike during a storm with a duration of six hours. The average dimensionless wave overtopping Q has a value of 0.0066, this leads to an average overtopping discharge of 0.161 $m^3/s/m$ or 161 $l/s/m$. This is a value way over the maximum of 10

Table 5-2: Calculated additional input variables

symbol	description	value	unit
h_b	water height on berm	5.53	m
L_0	'deep water wave length'	61.8	m
L_b	virtual berm length	34.6	m
N_w	number of incoming waves	3429	-
R_c	freeboard	2.56	m
$R_{u2\%}$	run-up level exceeded by 2% of incoming waves	6.63	m
γ_β	reduction factor angle of incoming waves	0.94	-
γ_b	reduction factor berm	0.96	-
ξ_0	Iribarren number	1.14	-

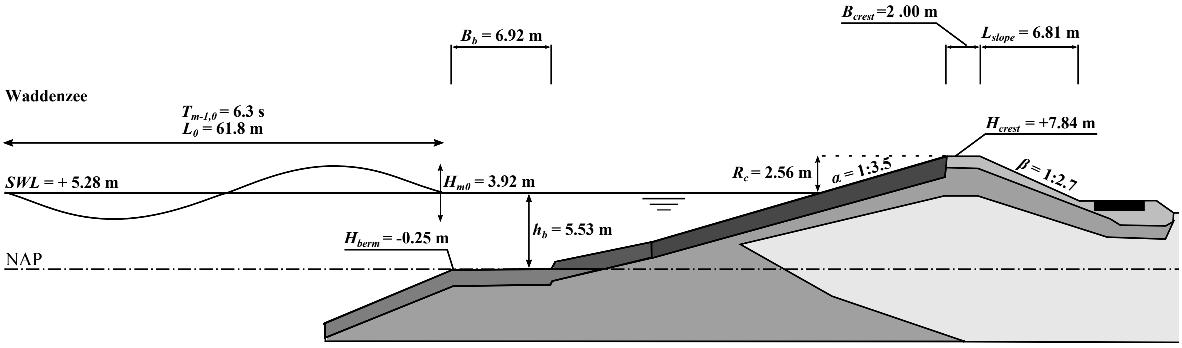

Figure 5-3: Visualization of the input data

$l/s/m$ currently used Pullen et al. [2007]. Also the limits which are based on tests are exceeded, grass on clay would be able to handle 30 $l/s/m$ Van der Meer et al. [2009]. Therefore the overtopping discharge is not a good measure for the load caused by wave overtopping. The overtopping discharge is within the expected range of 150-200 l/s/m of Rijkswaterstaat (RWS). The maximum wave overtopping discharge q_{max} is not, with 799 $l/s/m$, but as indicated this is based on less detailed data and therefore will not be used. These results are summarized in table 5-3.

Table 5-3: Overtopping discharge, given 1/10 000 year

symbol	description	value	unit
P_{ov}	probability of wave overtopping	0.56	−
N_{ow}	number of overtopping waves	1914	−
Q	dimensionless wave overtopping discharge	0.0066	−
q	average wave overtopping discharge	161	$l/s/m$
q_{max}	maximum average wave overtopping discharge	799	$l/s/m$

5-1-3 Box B: determination alternative load

According to figure 5-2 the average overtopping discharge q as a measure for the load appears to be insufficient if it reaches a value higher than 30 $l/s/m$ (van der Meer et al. [2011]), therefore a different measure needs to be found. This is what the second box, Box B: determination alternative load, in the Main Design Procedure as presented in figure 5-1 is all about. The alternative load appears to be the front flow velocity U of the overtopping wave. The overtopping volume per wave should be known first. This can be determined using equations 4-15, 4-21 and 4-22. The Weibull shape parameter b and scale factor a follow from the last two equations. These determine the shape and the scale of the distribution and therefore the distribution of the volume V per overtopping wave. The cumulative probability and the probability density function can be plotted as a function of the volume. Instead

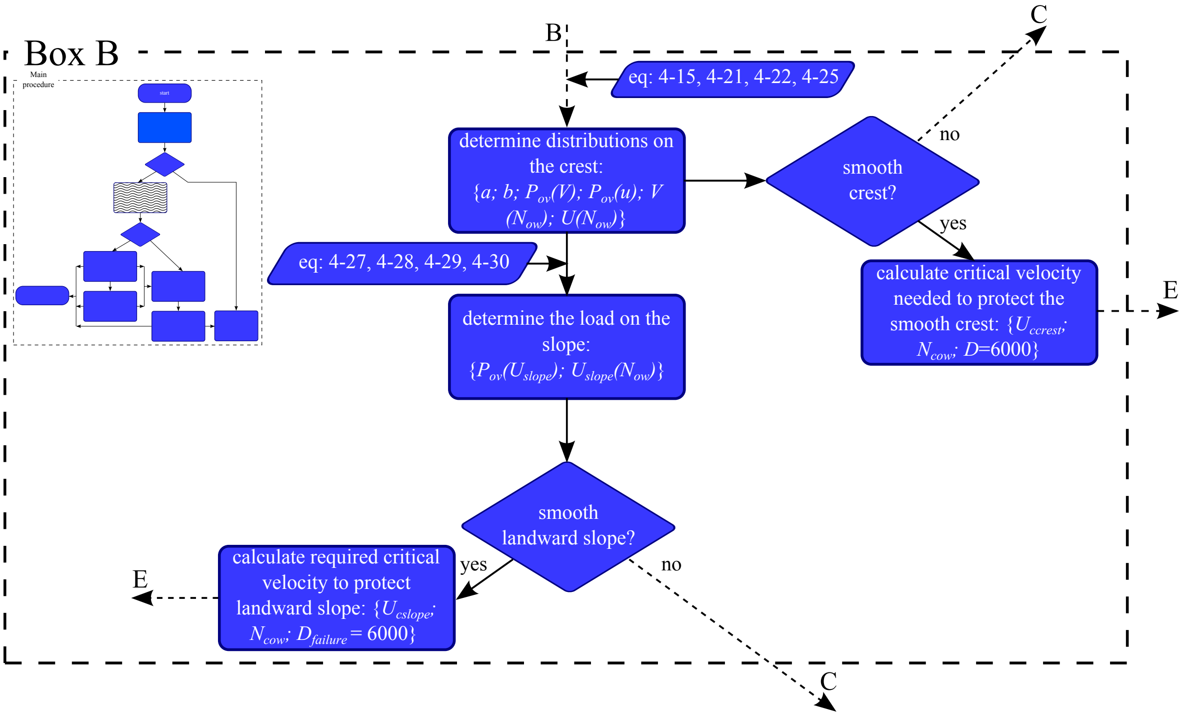

Figure 5-4: Box B: procedure for determination alternative load at crest and slope

of the volume the velocity needs to be known, therefore the volume V has to be transformed in a front flow velocity U, this can be done using the empirical relation as describe by equation 4-25. This results in a probability of a certain overtopping front flow velocity to occur. However, a probability is not a load as such. To transform the distributions of the probability of overtopping volume and front flow velocity to a load as a function of the number of overtopping waves. The distributions should be multiplied with the number of waves. The result is a resulting velocity per overtopping wave. This is thus a load per wave at the crest.

When the overtopping waves run-down the landward slope the velocity of the wave increases. This velocity increase can be calculated using equation 4-27 to 4-30 presented in section 4-5-3. This results in a front flow velocity per wave at the landward slope of the dike. The front flow velocity is the maximum front flow velocity per wave at the landward slope. The location along the slope at which this maximum front flow velocity occurs is not specified.

If the crest and landward slope are smooth, so without any discontinuities, this velocity per wave distribution is the load at the crest and landward slope. With this load the resulting required critical velocity U_c can be determined using Box E of the Main Design Procedure as described in section 5-1-6. However, if discontinuities are present at the crest or landward slope additional steps are required, this leads to Box C as described in section 5-1-4.

Application of Box B to Afsluitdijk

The average overtopping discharge q for the Afsluitdijk is 161 l/s/m, which is beyond the limit of 30 $l/s/m$. Therefore another measure for the load has to be found. The probability of a random wave to overtop the dike is more than 50% as described in section 5-1-2. In addition to this probability of overtopping, also the probability distribution of the overtopping volume per wave $P_{ov}(V)$ can be determined. First of all the value of the scale factor a is depended on the shape factor b, the overtopping discharge q as determined in section 5-1-2 (161 $l/s/m$) and on the probability of overtopping P_{ov} of 0.56. The value of the shape factor b is determined by the relative free board (R_c/H_{mo}) and the angle of the seaward slope α of 1:3.5. This leads to a value for b of 1.96 and value for a of 2.03 m^3/m. These results are presented in table 5-4. The distribution of the probability of a certain overtopping volume

Table 5-4: Parameters Weibull distribution probability of overtopping volume per waver, given 1/10 000

symbol	description	value	unit
R_c/H_{m0}	relative freeboard	0.65	-
a	scale parameter	2.03	m^3/m
b	shape factor	1.96	$-$

per wave to occur $P_{ov}(V)$, given 1/10 000 year conditions, at the crest, can be found in figure 5-5. The 90 % value is smaller than 3.1 m^3/m. The maximum values are above 5 m^3/m.

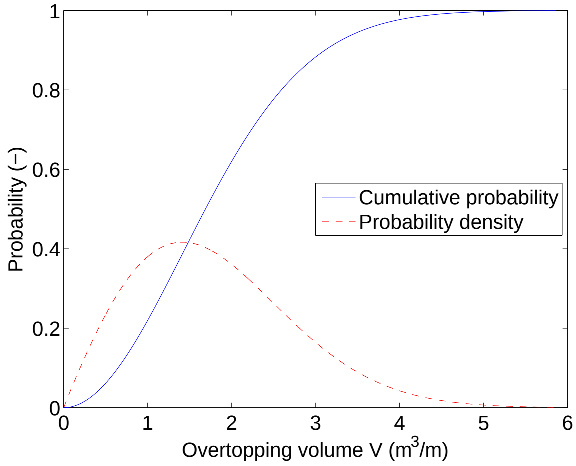

Figure 5-5: Distribution probability of overtopping volume per wave, at the crest, given 1/10 000 year conditions

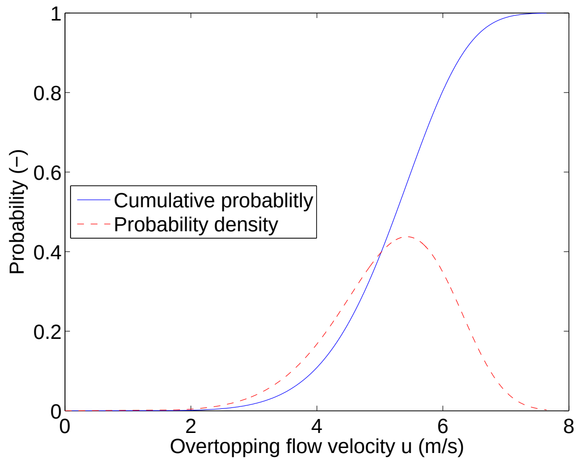

Figure 5-6: Probability of overtopping flow velocity per wave, at the crest, given 1/10 000 year conditions

Instead of the distribution of the volume per overtopping wave the distribution of the resulting front flow velocity per overtopping wave is the parameter that needs to be known. The distributions for the front flow velocities per wave $P_{ov}(U)$ are based on the distribution of the volume. These can be recalculated using the empirical relation. The distribution of the probability of a certain overtopping velocity to occur is shown in figure 5-6. The 90% value is a front flow velocity of approximately 6.3 m/s. The maximum front flow velocities are larger than 7 m/s.

A distribution of the probability of a certain volume or velocity per wave at the crest is known. This can also be used to determine a distribution per wave. What needs to be known is the velocity per overtopping wave. If the distributions are multiplied with the number of overtopping waves the velocity per overtopping wave is known. This results in a certain amount of waves that result in a certain velocity at the crest, this is depicted in figure 5-7. These waves are the number of waves that overtop the Afsluitdijk during 1/10 000 year conditions (only overtopping waves result in a velocity at the crest). If the crest is smooth, so no discontinuities are present at the crest, this would be the resulting load at the crest. With this velocity distribution the required critical velocity can be determined using Box E of the Main Design Procedure. Figure 5-8 shows the flow depths that go with the overtopping flow velocities. These flow depths in combination with the flow velocities at the crest will be used for the calculations of the overtopping flow velocities on the landward slope.

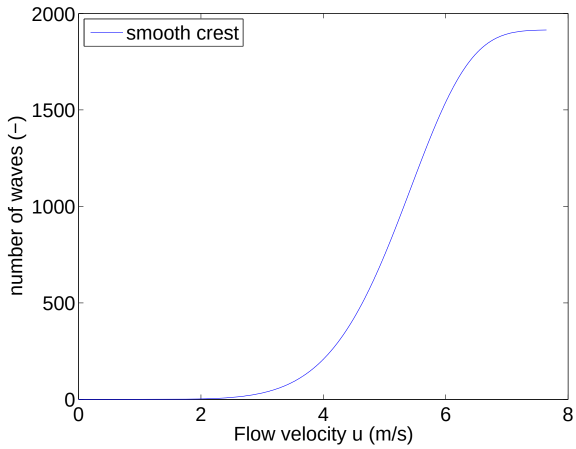

Figure 5-7: Crest wave overtopping velocities per wave, given 1/10 000 year conditions

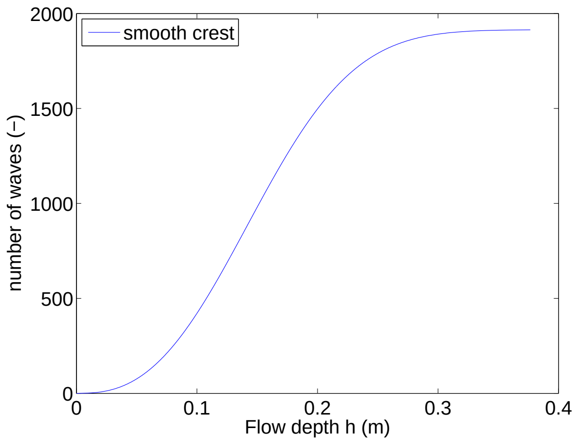

Figure 5-8: Wave overtopping flow depth per wave at crest, given 1/10 000 year conditions

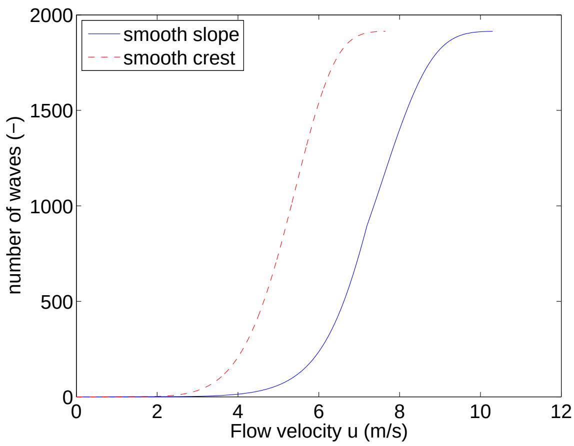

Figure 5-9: Wave overtopping velocities per wave for smooth slope, given 1/10 000 year conditions

The velocities at the landward slope of the Afsluitdijk are even more important, because more discontinuities are present at the landward slope and from testing it appeared that the velocity increases as the overtopping wave runs down the slope. This velocity increase can be calculating using the friction factor f and knowing the landward slopes angle β of 1:2.7. For the Afsluitdijk the choice has been made to calculate the velocity at the slope with the most conventional friction factor of 0.01, this has been done because this is the most conventional value at this moment and has the best results with the overtopping simulator tests. This results in figure 5-9, what can be seen is that the velocities along the slope are much higher than the velocities at the crest. This means that a higher critical velocity is required at the landward slope. This is the load at the Afsluitdijk for a smooth landward slope. The maximum front flow velocities are higher than 10 m/s and most of the waves result in a velocity between 4 and 8 m/s. However, the landward slope of the Afsluitdijk is not smooth and therefore a certain load amplification should be determined. For the case of the Afsluitdijk the procedure of Box B is not sufficient to lead to Box E. The next step is to increase the load with an amplification factor based on the different discontinuities that are present at the Afsluitdijk. This is the next step which is described in Box C, section 5-1-4.

5-1-4 Box C: load amplification based on categorization of discontinuities

Only in an ideal situation the dike would have a smooth slope without any discontinuities, in reality there are always small damages and also other discontinuities. In order to obtain information on the influence of these discontinuities, the type of discontinuity has to be determined. These can be transitions, holes in the landward slope or NWO's. This last category has such a wide variety it self that a separate procedure has been made in order to derive the amplification factor. This procedure can be found in section 5-1-5. For the holes in the landward slope, whether or not the flow is influenced

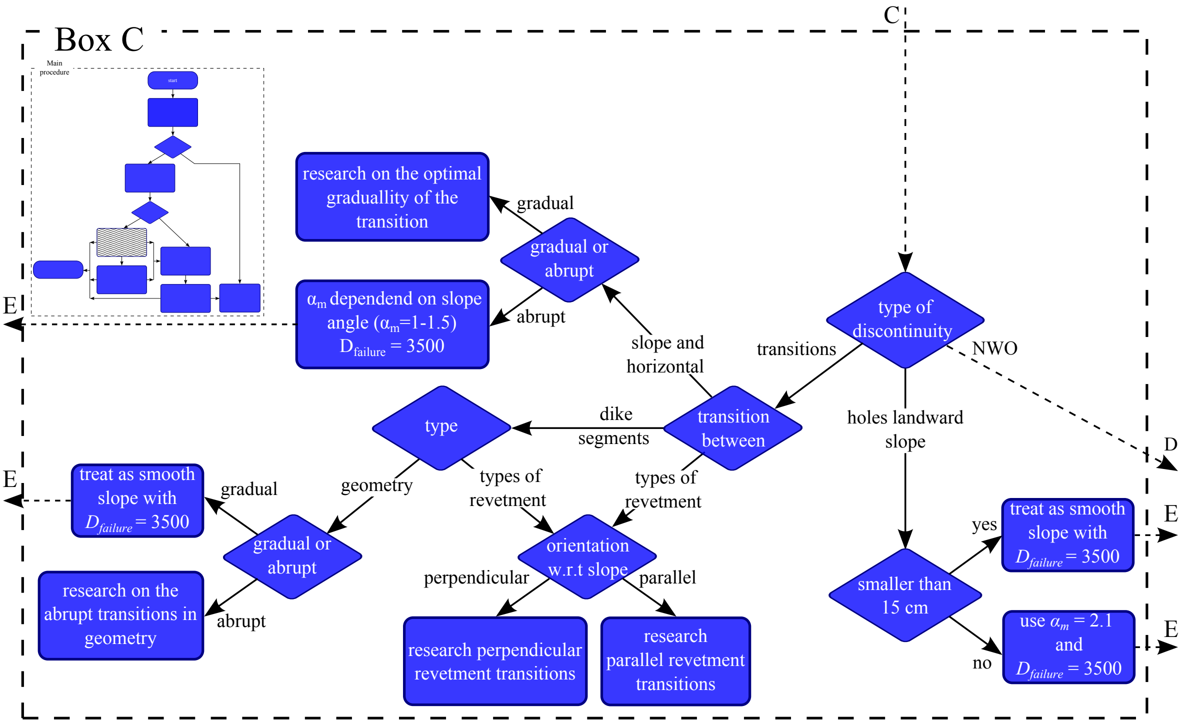

Figure 5-10: Box C: Procedure to determine load amplification based on discontinuities

depends on the size of the hole. If it is 15 cm or smaller there is a negligible influence on the flow velocity of the overtopping wave and the procedure of Box E in section 5-1-6 with a allowable amount of failure $D_{failure}$ of 3500 m^2/s^2 should be used. If the holes are larger than 15 cm there is an influence on the flow. The amplification factor α_M as derived with equation 4-36 is 2.1. This amplification factor is based on grass.

The last category are the transitions, within these there are a few different transitions. These are the transition between slope and horizontal, transition between different dike segments and the transitions between different types of revetments. If the transition between slope and horizontal is gradual there is a negligible influence on the flow. On the other hand if it is abrupt there is an influence on the flow. The amplification factor for the increase of the flow velocity can be calculated using equation 4-47. Depending on the angle of the landward slope α_M varies between 1.1 and 1.5. For some angles the amplification factor α_M can be found in table 4-5. With this amplification factor the steps described in Box E, section 5-1-6, can be used to determine the required critical velocity with a $D_{failure}$ of 3500 m^2/s^2. The exact limits for which the transition is gradual are not known. Therefore additional research should be done on this subject to be able to make a more accurate statement about how gradual a certain transition has to be in order to have a negligible influence on the flow.

The transition between different types of revetment can be found in two separate orientations, perpendicular or parallel to the slope. The largest influence is expected when the transition is perpendicular to the slope. The latter has been tested while the first has not been tested. This testing has only been done in combination with a transition from slope to horizontal. The erosion started at these kind of transition in most cases. Especially the combination with the transition to the horizontal makes this a critical transition. For the parallel transition an amplification factor is not available at this moment. Therefore more research should be done on the transitions between different types of revetment parallel to the slope.

The transitions between different dike sections can consist of two components, a difference in the geometry of the dike or a difference in the revetment type. The latter leads to the choice presented earlier, so perpendicular or parallel to the slope. The parallel transition is the options which applies to the transition between dike sections. If the geometry of the dike becomes different the flow can be concentrated around this transition. It depends on how gradual this transition is performed. As long as the distance over which the transition has been done is long enough the influence on the flow will be negligible. However, if it is abrupt than the flow can be influenced or concentrated. This has not been investigated up till now, so additional research is necessary here as well. If it is gradual the $D_{failure}$ of 3500 m^2/s^2 can be used in the procedure described in Box E.

Application of Box C to Afsluitdijk

Because the derivation of the design procedure is based on the Afsluitdijk and is now applied to the Afsluitdijk as well gives that almost all discontinuities as described in this section are present at the Afsluitdijk. Therefore all the paths in the procedure of Box C have to be considered. This will result in one of the discontinuities to be governing. The resulting governing discontinuity is only valid for the case of the Afsluitdijk and does not apply to other dikes. The same discontinuity can be governing for other dikes as well, but this is not necessarily the case. This is because the discontinuity that is governing here might not be present at other dikes or the situation round the governing or other discontinuities is not the same as for the Afsluitdijk.

The first discontinuities under consideration are the transitions, to start with the transition between slope and horizontal. The landward slope of the Afsluitdijk has an angle of 1:2.7 which makes it per definition an abrupt transition (>1:4). By using equation 4-47 the amplification factor α_M becomes 1.09. This is a load increase with approximately 10% due to the effect of the slope. The velocities are slightly higher than at the landward slope, the maximum velocities are higher than 11 m/s. The resulting velocity profile can be seen in figure 5-11. The remark must be made that the velocity used here is the maximum velocity U_{max} along the slope. However, in reality the flow decelerates towards the end of the slope, this has been found during testing. This has not been taken into account, because the measured deceleration was not the same for all slopes or velocities, which makes it difficult to take into account. Therefore this is most likely a conservative velocity to use, in reality the values might be lower. The amplified velocity as shown in figure 5-11 can be used in Box E to determine the required critical velocity

The next step is to look at the transition between different dike sections. There are two types of transitions, geometrical transition, so the height or width of the dike can change, or transitions between

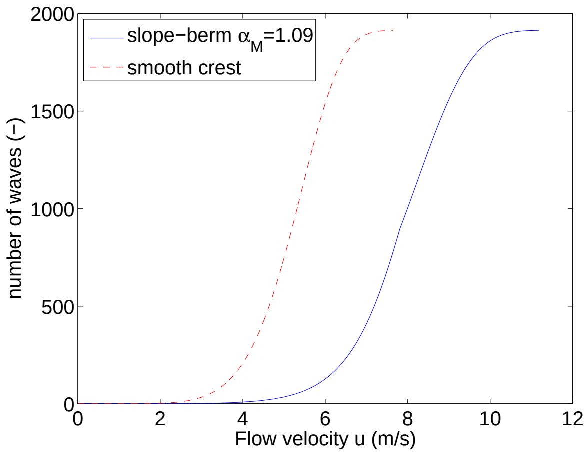

Figure 5-11: Distribution of amplified front flow velocity at transition between slope and berm, given 1/10 000 year conditions

different type of revetments. The first type occurs at various locations along the length of the Afsluitdijk. Also when the driveways up the slope are taken into account. The transitions in height along the dike do most likely not cause an increase in the load because these often have a long stretch (>100 m) and the height and width differences are not very large. Off course this requires more research. However, the expectation is that there will be a negligible influence of this type of transitions. For the Afsluitdijk the advise is to use a $D_{failure}$ of 3500 m^2/s^2 combined with figure 5-9 for the transition in geometry of the different dike sections. Currently there is no transition between different types of revetments along the length of the Afsluitdijk, parallel to the slope. For the driveways up the slope the advise is to use the front flow velocities as depicted in figure 5-11 combined with a $D_{failure}$ of 3500 m^2/s^2. This already takes into account the transition between slope and horizontal. The expectation is that the angle of the driveway does not have an additional effect on the flow velocity.

For the paved driveways the situation is different, there is a combination of a change in geometry and a change in revetment type perpendicular to the slope. However, the expectation from testing is that the transition between the slope of the dike and the slope of the driveway is still the governing failure mode. This is based on tests executed with the wave overtopping simulator.

Off course there will be some initial damages caused by animals, objects or other causes. The question is if the expectation is that these will be larger than 15 cm and will influence the flow. By allowing only a damage number $D_{failure}$ of 3500 m^2/s^2 the smaller damages, of 15 cm and smaller, are covered and the load as defined in figure 5-9 can be used. Only when the maintenance is very good the statement can be made that holes larger than 15 cm will be repaired with a time interval that is small enough to prevent larger holes and therefore the influence on the flow velocity is negligible.

The Afsluitdijk is an important water defense in the water safety system of the Netherlands, therefore the expectation is that the maintenance will be good enough to guarantee that no large damage will be present long enough to endanger the safety of the dike.

Because this is the expectation the $D_{failure}$ of 3500 m^2/s^2 combined with the load as depicted in figure 5-9 should be used instead of an amplification factor. For the points treated above the required critical velocity U_c can be determined with Box E.

The last transition type are the NWO's. These are present at the Afsluitdijk and therefore Box D should be used in order to determine the influence of the different NWO's on the front flow velocity.

5-1-5 Box D: load amplification for NWO's

The NWO's appeared to be present at such a large variety themselves that a separate procedure has been composed in order to get a load amplification factor for this category. This procedure is based on the categorization of the NWO's as presented in section 4-4-6. For each category an analysis has been made to see what additional steps need to be taken to get a value for the amplification factor α_M. The procedure can be found in figure 5-12.

First of all the category within the NWO's to which the NWO under consideration belongs has to be determined. The choice is between vegetation, cables and pipelines, poles or structures. For vegetation the influence depends on the kind of vegetation whether it are trees or bushes. For bushes, no testing has been done or no theoretical method is available at this moment. Therefore research to the influence of bushes on the flow velocity should be done. On the other hand there can be trees which do have an influence on the flow if the diameter is larger than 15 cm. The amplification of the flow velocity is expressed by the factor α_M of 1.2. Due to the shadow effect round trees also the allowable damage $D_{failure}$ of 3500 m^2/s^2 should be used. These two adaptations can be used for the determination of the required critical flow velocity U_c with Box E. For trees smaller than 15 cm there is no influence on the flow. However, due to the shadow effect the allowable damage $D_{failure}$ of 3500 m^2/s^2 should be used in the procedure of Box E.

For the category cables and pipelines the question is whether or not these penetrate the top layer. If this is not the case there is no influence on the flow. Therefore a $D_{failure}$ of 6000 m^2/s^2 can be used in Box E. If they do penetrate the top layer then these should be treaded as buried but visible structures as described later in this section.

The third category has the same criterion as trees. If the poles have a diameter smaller than 15 cm the influence on the flow velocity is negligible and only the allowable damage has to be adapted to 3500 m^2/s^2 and can be used in Box E. If the poles are larger than 15 cm in diameter there will be an influence on the flow. This influence also depends on whether or not the pole has a foundation or has just been driven into the ground. If these do not have a foundation the load should be increased with a factor α_M of 1.2 and $D_{failure}$ of 3500 m^2/s^2 should be used in Box E. If the poles do have a foundation these should be treated as buried but visible structures as will be described below.

The structures category consists of three different types of structures, these are external structures, buried structures or buried but visible structures. Most likely external structures do not have an influence on the flow velocity because these are flushed away.

If the external structures are large enough to cause damage to the top layer these do have an influence. Currently there is no criterion for this subject. This is a subject for further research.

When these are not large enough to expect damage, the slope should be treated as a smooth slope with an allowable damage number $D_{failure}$ of 3500 m^2/s^2 and this should be used in Box E. Next to external there are also buried structures which in principle are not exposed to the flow of the overtopping wave. However, some of the buried structures are not completely below the top layer, so if a part of the top layer erodes the buried structure will become exposed at the surface. Therefore the question is if the buried structure has a probability of exposure. If this is not the case it can be treated as a smooth slope with a allowable damage number $D_{failure}$ of 6000 m^2/s^2 in Box E. If it has a possibility to be exposed the buried structure should be treated as a buried but visible structure. The buried but visible structure has an influence on the flow it depends on the shape of the structure. The shape can be elliptic, rectangular and cylinder. The amplification factor, dependent on the shape and the ratio of length and the width of the structure, can be determined using equation 4-50 in combination with the shape factors presented in table 4-6.

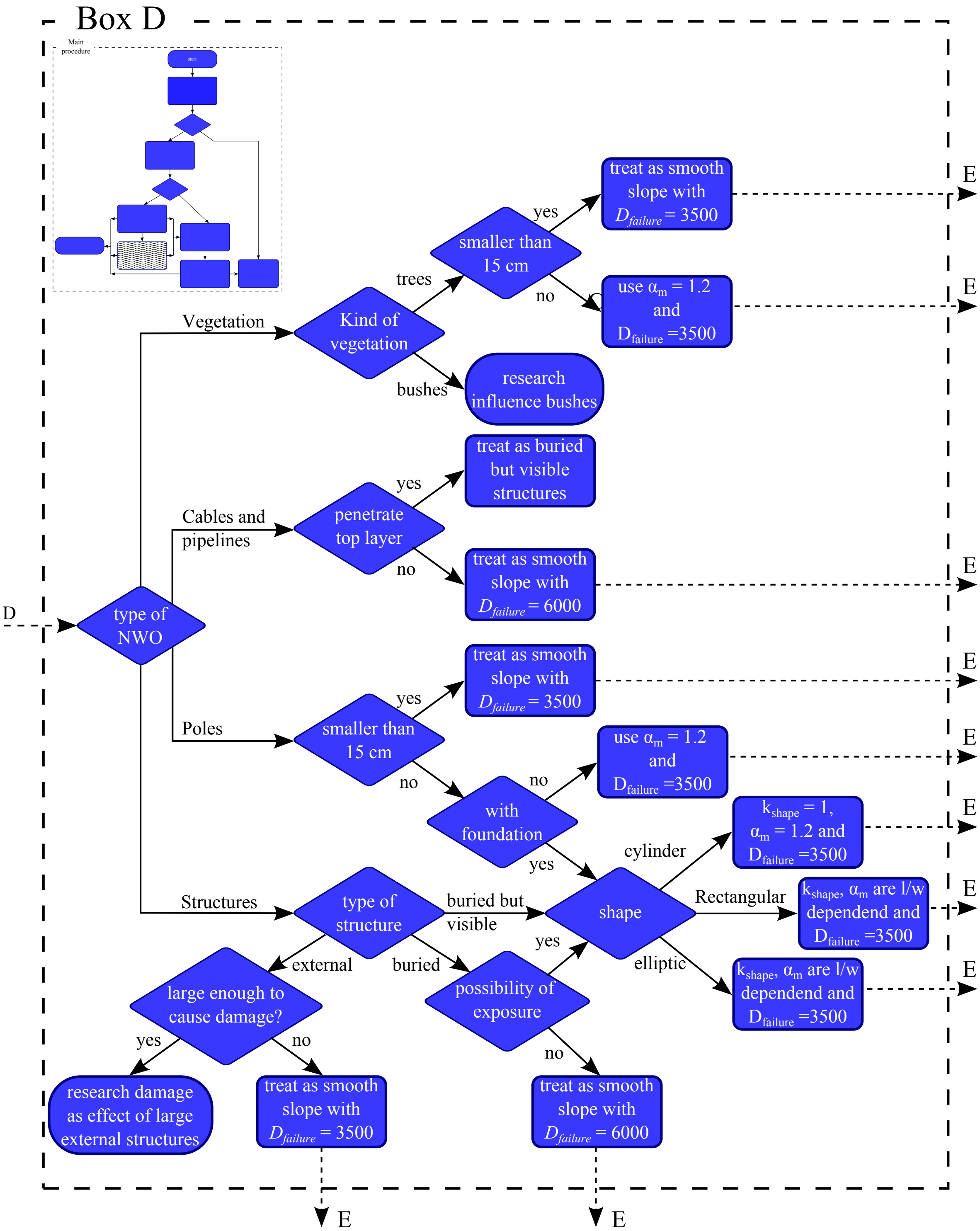

Figure 5-12: Box D: Procedure to determine required critical velocity U_c

Application of Box D to Afsluitdijk

For Box D the same situation applies as for Box C, the categorization is based on the Afsluitdijk and is applied to the Afsluitdijk as well. Therefore all the categories presented in Box D are present at the Afsluitdijk. Each category within this Box will be treated here separately.

Vegetation is present at the Afsluitdijk but mostly on the parts where the bunkers are positioned, near the Den Oever lock complex and the Kornwerderzand lock complex. Near Den Oever the vegetation is not located at crest or the landward slope of the dike. Only in the front of the dike and on the IJsselmeer side of the A7. At Kornwerderzand some trees/bushes are present near the crest of the dike. However, this is a special dike section, the whole dike is wider at that section, therefore the load will probably be less than is calculated here. The amount of overtopping will be reduced by the foreland at that location. The choice has been made to use the load as depicted in figure 5-7, because the trees/bushes are standing on the crest. Currently the influence of bushes on the flow is unknown and therefore no amplification factor can be given. Assuming that trees with a diameter larger than 15 cm are present at this location the amplification can be determined with equation 4-50 and a shape factor K_{shape} of 1, resulting in an amplification factor α_M of 1.2. The distribution of the amplified front flow velocity U resulting from the overtopping waves as depicted in figure 5-13. The maximum velocities for trees on the crest are a little bit lower than the maximum velocity on the slope, but higher than the velocities on the crest. The maximum velocities are higher than 9 m/s.

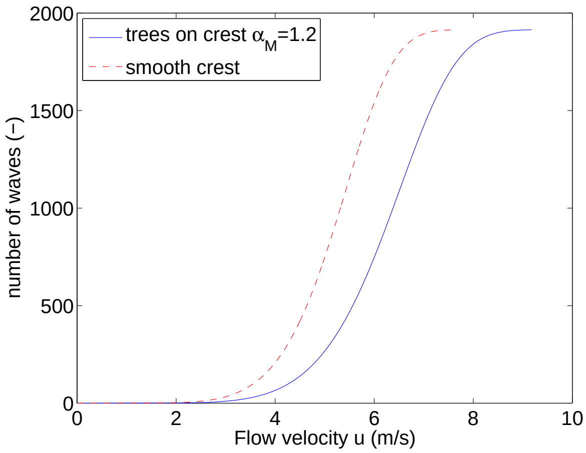

Figure 5-13: Front flow velocity amplified with trees on crest, given 1/10 000 year conditions

Cables and pipelines that penetrate the top layer have not been found during the inventory of the NWO's and failure of cables and pipelines are not under consideration. Therefore no load increase or a reduction of the damage factor $D_{failure}$ is required.

Poles are present in a large variety, there are poles smaller than 15 cm in diameter, for instance the reflector poles along the highway A7 and poles larger than 15 cm in diameter like directional signs and land posts. The poles smaller than 15 cm have a negligible influence on the flow therefore the $D_{failure}$ of 3500 m^2/s^2 together with the front flow velocities as depicted in figure 5-9 should be used here. For poles large than 15 cm there are two categories, the ones with a foundation and the ones without a foundation. Without a foundation a $D_{failure}$ of 3500 m^2/s^2 in combination with equation 4-50 and a shape factor of 1.2 should be used. Because on the landward slope the load before amplification is the largest (the highest front flow velocities) as depicted in figure 5-9 will be used as input before amplification. The resulting amplified front flow velocities are shown in figure 5-14. This can be used in the procedure as described by Box E. The maximum velocities are higher than 12 m/s. Which is higher than the earlier derived amplified velocities for poles, trees and maximum velocity at the landward slope. Instead of poles without a foundation, also poles with a foundation are present. For

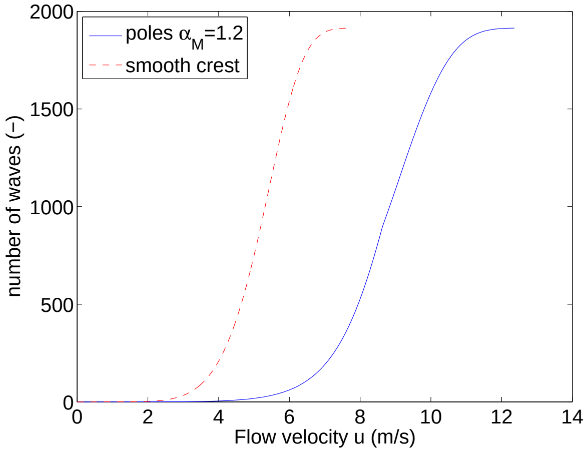

Figure 5-14: Front flow velocity amplified for poles without foundation, given 1/10 000 year conditions

instance the beacon or the portals over the highway A7. These will be treated as structures that are buried but visible and will be treated later on in this paragraph.

The Afsluitdijk also has a large variety of structures, as described in the procedure in Box D. There are three different categories within the structures category. First the external structures will be considered. These can be the benches and tables, the highway A7 or the agricultural tanks. The objects like benches and tables and tanks will flush away with the flow and are assumed not to be large enough to cause damage. For these kind of objects the flow velocity at the landward slope as shown in figure 5-9 in combination with damage number $D_{failure}$ of 3500 m^2/s^2 should be used for the input of Box E.

The highway A7, cycling path and parking lots are a different story. These will most likely not flow away all at once, but will be undermined and have impact erosion at the front of the transition between these structures and the soil. Currently there is no method in order to determine the effects of flow on such structures. This is a subject that should be investigated.

Next to the external structures there are also completely buried structures, whether or not these are present at the Afsluitdijk is not clear. However, these have to be taken into account. If there is no risk of exposure before the top layer has been eroded away completely this category can be treated as a smooth slope without any discontinuities and have a load as depicted in figure 5-9. However, if there is a risk of exposure then these must be treated as buried but visible structures as described next.

The buried but visible structures are the structures with the largest influence on the load. Depending on the shape of the structure the influence is determined. All shapes can be found on the Afsluitdijk and therefore will be treated here. For the circular shapes the amplification of the load can be found in figure 5-14. For poles and circular structures this is the same. For rectangular and elliptic shapes the flow velocity is depended on the ratio between length and width. The variety at the Afsluitdijk is so large that it is actually most interesting to see what the governing object does with the flow.

The objects that have the largest influence are rectangular objects with a ratio between length/width of 1. The amplification factor α_M becomes 1.44. Examples of such objects are the portals and bridge pillars. The resulting velocities are shown in figure 5-15. Most of the waves result in velocities between 7.5 and 12.5 m/s. The maximum velocities are higher than 14 m/s. These are very high flow velocities. As long as these objects would be on the landward slope of the Afsluitdijk these are the front flow velocities. Because the maximum velocity along the landward slope has been used as front flow velocity to amplify. This means that the velocities shown here would occur if the structure is located at the same location as where the maximum velocity along the landward slope occurs. This is the worst

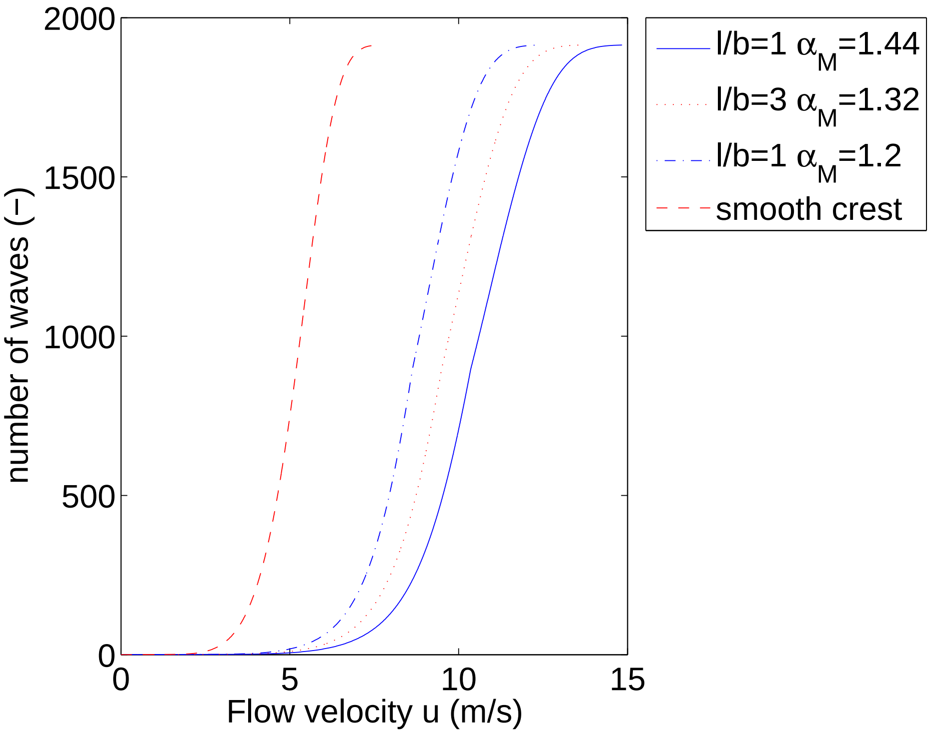

Figure 5-15: Front flow velocity amplified for rectangular structures on the slope, given 1/10 000 year conditions

combination that can occur. However, currently at the Afsluitdijk these objects are not located at this location and this velocity is likely to be an overestimation. This is the maximum amplification at the location were the velocity is already the highest.

The other objects that can be found on the Afsluitdijk have other ratios between length and width. For instance a ratio of 3 can be used. This results in an amplification factor α_M of 1.32. Another option is a ratio between the length and the width of the structure of 5, which results in an amplification factor α_M of 1.2, which is the same as the amplification factor for trees or poles. The resulting front flow velocities per overtopping wave can be found in figure 5-15. The resulting front flow velocities are slightly lower than for a ratio of 1. The highest velocities are higher than 13 m/s for $l/w = 3$ and higher than 12 m/s for $l/w = 5$. The larger the ratio between the length and the width of the structure is the lower the front flow velocities are, this can be seen clearly in figure 5-15.

For the elliptical shaped structures, the amplification factors α_M are very close to 1, also depending on the l/w ratio. The amplification factor for l/w of 2 and 3 are 1.02 and 0.96, respectively. Because it is not the expectation that the flow velocity will reduce because of the presence of an object, and these amplification factors are so close to 1, and the uncertainties are large, the choice has been made to take the amplification factor α_M for structures with an elliptical shape at 1. Therefore for elliptical shaped structures the front flow velocity as depicted in figure 5-9 in combination with a $D_{failure}$ of 3500 m^2/s^2 as input for Box E. With this all the categories of discontinuities are treated and amplified to a front flow velocity.

Another important observation can be done based on the velocity distributions for the different discontinuities. The shape of the distribution changes slightly when it is amplified. The large the amplification factor, the larger the change in shape. The distribution becomes less steep for larger amplification factors. This can be seen the most clearly in figure 5-15. This means that the contribution of each wave ($U > U_c$) increases with an increasing amplification factor α_M. The difference between U and

U_c is larger. This partly explains why the number of critical overtopping waves decreases for larger amplification factors.

5-1-6 Box E: determination required critical velocity U_c

The procedure of Box E: determination required critical velocity U_c, is solely about converting the load into a required critical strength. The load, influenced by discontinuities or not, should be recalculated to a critical velocity U_c which can be used to determine what kind of protective measures are required.

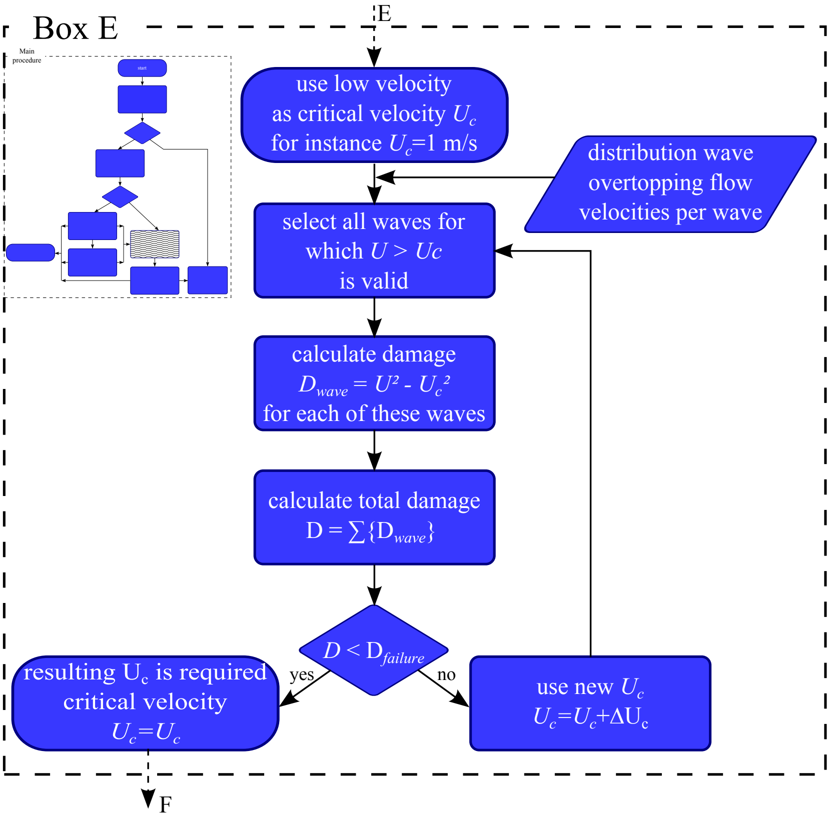

Figure 5-16: Box E: Procedure to determine required critical velocity U_c

The procedure as shown in figure 5-16 starts with the loads determined with box B, C or D in the form of a front flow velocity per wave. A low critical velocity U_c is chosen for instance 1 m/s. The second step is to select only the waves for which the statement $U > U_c$ is true. If these waves are selected the number of waves that contribute to the damage is known for an U_c of 1 m/s. For each of these waves the contribution to the damage can be calculated and added for the total damage using equation 4-51.

If the allowable amount of failure $D_{failure}$ is smaller than the amount of failure calculated with the above described procedure, the assumed critical velocity is the required critical velocity. However, when this is not the case a new critical velocity should be assumed by adding a small step ΔU_c to the earlier assumed critical velocity. The procedure should be repeated starting with selecting the waves for which $U > U_c$. This loop has to be repeated as long as the critical velocity has not been found. As long as $D < D_{failure}$ is not true the required critical velocity U_c has not been found. If that criteria is true the required critical velocity has been found, the next step is using Box F in order to find the required slope protection.

The process as described in this box in figure 5-16 is an iterative process which takes a lot of time given the large amount of waves that need to be taken into account and the number of iterations that should be done in order to get to the right critical velocity. The time is depending on precision that is required for the required critical velocity or the ΔU_c. The choice has been made to choose a precision of 0.1 m/s for the required critical velocity U_c.

Application of Box E to Afsluitdijk

The procedure as described here will be used in order to determine the required critical velocity U_c for all the points as described in Box A, Box B, Box C and Box D. These critical velocities can be determined and compared. First the smooth crest and slope will be described, secondly the loads amplified by the critical points from Box C and as last the resulting amplified load as found by application of Box D. At the end of this section these critical velocities are compared and the governing point can be determined for the case of the Afsluitdijk.

Smooth crest and slope

The results of the procedure depicted in figure 5-16 for a smooth crest and slope ($D_{failure}$ = 6000 m^2/s^2) are a required critical velocity U_c of 5.5 and 8.0 m/s, respectively. The percentage of waves that contribute to this damage are 22.2% and 15.1% for crest and slope, respectively. The waves that contribute can been seen in figure 5-17 and 5-18. These numbers are the solving point for U_c belonging to the load, the distribution of overtopping velocities, and the damage number of 6000 m^2/s^2. Due to the velocity increase the percentage of waves that contribute to the damage decreases. This is because of the fact that each wave has a larger contribution to the damage. So with less waves more damage can be done. The difference is approximately 7% between the waves contributing at the crest or at the landward slope.

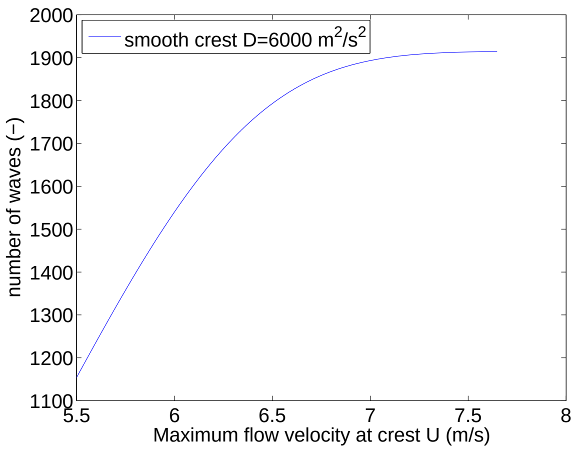

Figure 5-17: Number of waves contributing to damage number D=6000 m^2/s^2 at crest, given 1/10 000 year

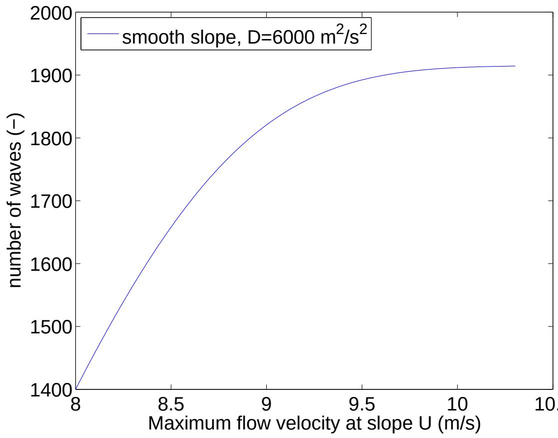

Figure 5-18: Number of waves contributing to damage number D =6000 m^2/s^2 at slope, given 1/10 000 year

Small damages

The same has also been done for slopes with some small holes and damages along the slope, so with an allowable damage number $D_{failure}$ of 3500 m^2/s^2. The results are not that different, with only a small increase in the required critical velocity. For the crest a U_c of 5.8 m/s is needed, with a contribution of 15,1% of the overtopping waves and for the slope 8.3 m/s with 10.2% of the overtopping waves contributing. The contributing waves and the resulting velocities can be seen in figure 5-19 for the crest and 5-20 for the slope.

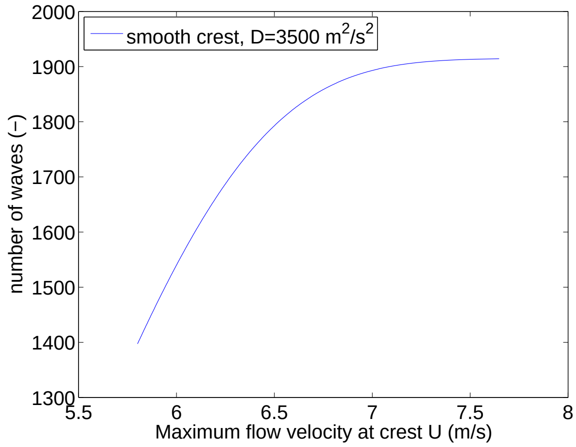

Figure 5-19: Number of waves contributing to damage number D=3500 m^2/s^2 at crest, given 1/10 000 year

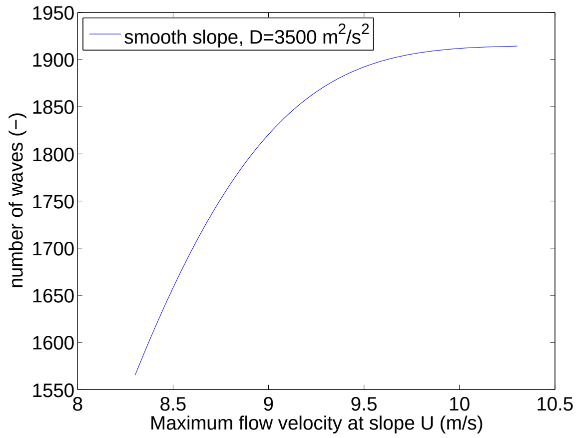

Figure 5-20: Number of waves contributing to damage number D =3500 m^2/s^2 at slope, given 1/10 000 year

The difference in number of waves that should be taken into account is significantly different for both the slope and the crest. The reason for this is that the cumulative overload method is based on the velocity squared. So if the allowable damage number is decreased by half, the amount of waves that contribute to the damage is lowered as well. The reason that only 0.3 m/s increase in required critical velocity already results in such a large reduction in the percentage of waves that contributes, is that the curve of the number of waves against the front flow velocities is steep. This means that a little increase results in a lot less waves. This is also the reason for the slower decrease of the number of waves contributing to the damage for the slope. Already a lower percentage contributed to the load, which means that the curve was less steep. Therefore the decrease in the percentage of waves contributing to the damage for the slope is lower than for the crest.

Amplified by discontinuities

The load, the amplified velocities as shown in section 5-1-4 and 5-1-5, give different results than the above presented critical velocities U_c and number of waves contributing to the damage N_{cow} for a smooth slope. These amplified velocities are used as input for the procedure of Box E. Because all of the critical points for which the increased velocities have been determined are discontinuities around which the probability on small damages is large, the allowable damage number $D_{failure}$ for all these points has been set to 3500 m^2/s^2 for this exact reason.

The first discontinuity under consideration is the transition between the slope and the berm. The input can be seen in figure 5-11. The required critical velocity is 9.1 m/s and the number of waves that contribute to the damage is 9.0 %. These velocities and waves can be seen in figure 5-21.

Another discontinuity are the trees at the crest of the dike of which the amplified velocities can be seen in figure 5-13. The resulting required critical velocity for this discontinuity is 7.1 m/s and the percentage of waves contributing to the damage is 12.6 %. The critical velocity is lower than for the transition between slope and berm or the landward slope with small damages. This is because the trees are positioned on the crest at which the velocities are lower. The waves and velocities contributing to the damage are depicted in 5-22.

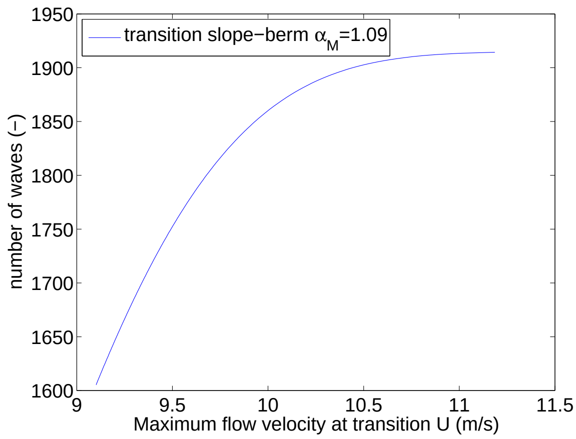

Figure 5-21: Number of waves contributing to damage number D =3500 m^2/s^2 at transition slope-berm, given 1/10 000 year

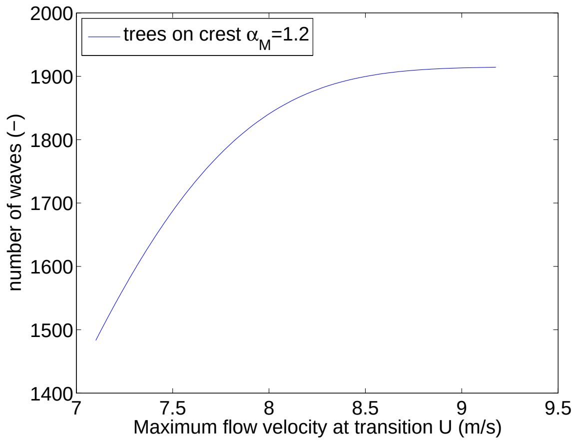

Figure 5-22: Number of waves contributing to damage number D =3500 m^2/s^2 at crest for trees, given 1/10 000 year

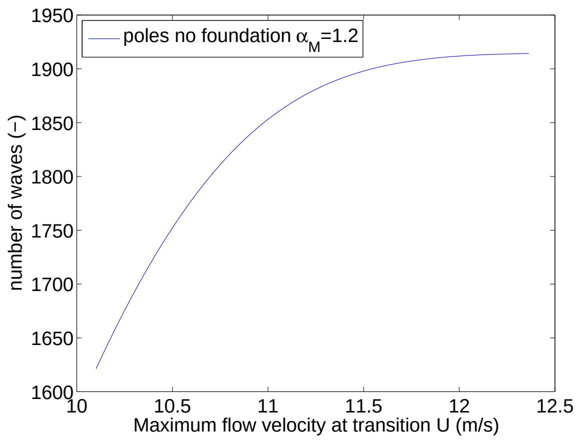

Figure 5-23: Number of waves contributing to damage number D =3500 m^2/s^2 at slope for poles without foundation, given 1/10 000 year

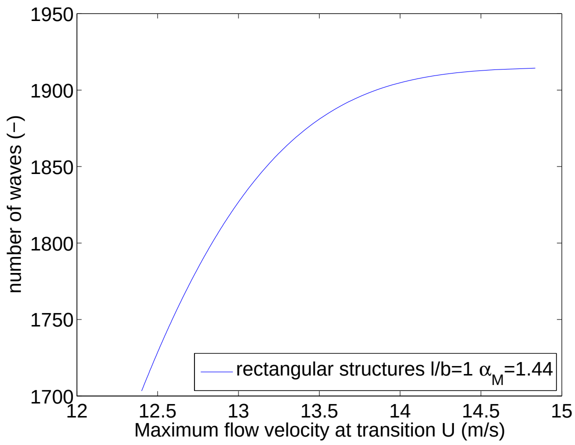

Figure 5-24: Number of waves contributing to damage number D =3500 m^2/s^2 at slope for rectangular structures with l/w=1, given 1/10 000 year

The third type of discontinuity are poles without a foundation from which the amplified velocities can be found in figure 5-14. The resulting required critical velocity is 10.1 m/s and the percentage of waves contributing to the damage is 8.6 %. This is higher than all of the above velocities. This is because the amplification factor is the largest as well as the input velocity. The waves with the velocities contributing to the damage can be found in figure 5-23.

The different rectangular structures result also in different velocities. As a starter the rectangular structures which have the same length and width, so a ratio between length and width of 1 have an amplified velocity per wave as depicted in figure 5-15. The resulting required critical velocity is 12.4 m/s and only 6.2 percent of the incoming waves contribute to the damage. The waves with their velocity can be seen in figure 5-24. For rectangular structures with a length/width ratio of 3, so the length is 3 times as large as the width, the amplified velocity profile can be found in figure 5-15. The resulting required critical velocity is 11.3 m/s and the percentage of waves contributing to the damage

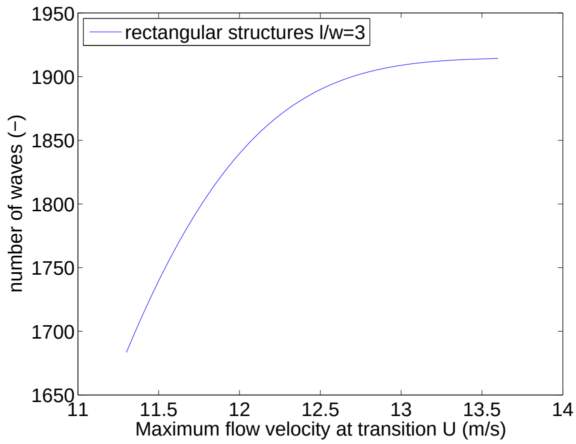

Figure 5-25: Number of waves contributing to damage number D =3500 m^2/s^2 at slope for rectangular structures with l/w=3, given 1/10 000 year

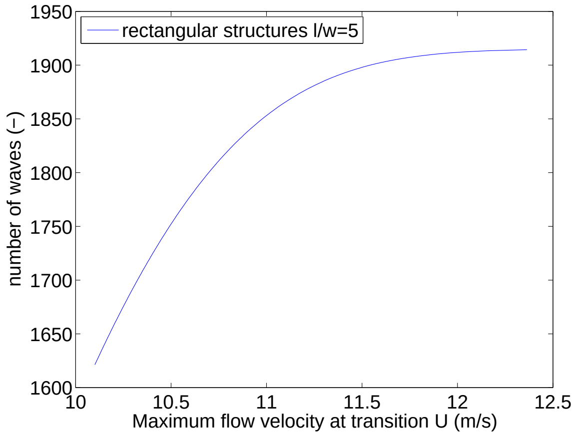

Figure 5-26: Number of waves contributing to damage number D =3500 m^2/s^2 at slope for rectangular structures with l/w=5, given 1/10 000 year

is 6.8 %. The waves contributing to the damage are shown in figure 5-25 with the resulting velocities. Rectangular structures with a length/width ratio of 5 have an amplified velocity per wave as shown in figure 5-15, the resulting required critical velocity is 10.1 m/s and the percentage of waves contributing to the damage is 8.6 %. These waves and the velocities are shown in figure 5-26.

The results for structures with the shape of a cylinder are the same as for poles without a foundation. These are presented in figure 5-23, the required critical velocity is 10.1 m/s and 8.6 % of the waves are contributing to the damage.

There are also a lot of discontinuities that appeared to not have an amplified load, the amplification factor α_M for these discontinuities appeared to be 1. For all these discontinuities the velocities as depicted in figure 5-9 are used to determine the required critical velocities. The resulting required critical velocity is also the same with 8.3 m/s and a percentage of waves that contribute to the damage of 10.2%, these waves are shown in figure 5-20. The discontinuities to which this applies are holes in the landward slope <15cm, gradual transitions between dike segments, small external structures and elliptical shaped structures.

Summarizing the results

A summary of the results of the in this section determined required critical velocities U_c, number of critical overtopping waves N_{cow} and the percentages critical overtopping waves contributing to the damage P_{cow} can be found in table 5-5.

The required critical velocity U_c can be plotted against the percentage of critical overtopping waves P_{cow} that contribute to the damage. This can be seen in figure 5-27. What is shown is that there seems to be a trend in the relation between both the required critical velocity and the percentage of critical overtopping waves. The exact trend is not known, the amount of data points is to low to quantify this. However, what can be seen is that the relation appears to have an exponential character. This can be expected because the required critical velocity has been determined with the number of waves. The number of waves is used as the amount of summations that has to be done in order to calculate the total damage number. It is important to realize that this is for the damage number of 3500 m^2/s^2. If the allowed damage number is larger the percentage of waves that is used in order to reach this number with the same U_c will be larger. This trend is only valid for the distribution of overtopping wave volumes as determined for the Afsluitdijk. Therefore this trend is not applicable for other situations. It is interesting to see that the allowable critical velocity results in an non linear

Table 5-5: Critical flow velocities of the different locations/discontinuities

location/type	$D_{failure}$ $[m^2/s^2]$	N_{cow} [-]	P_{cow} [%]	U_c $[m/s]$
crest	6000	761	22.2	5.5
crest small damages	3500	518	15.1	5.8
landward slope	6000	515	15.0	8.0
landward slope small damages	3500	350	10.2	8.3
Holes landward slope $<$ 15 cm	3500	350	10.2	8.3
transition landward slope and berm	3500	309	9.0	9.1
gradual transition dike segment geometry	3500	350	10.2	8.3
trees on crest $>$ 15 cm	3500	432	12.6	7.1
trees on crest $<$ 15 cm	3500	518	15.1	5.8
poles without foundation	3500	294	8.6	10.1
small external structures	3500	350	10.2	8.3
buried structures no risk of exposure	6000	514	15.0	8.0
cylinder shaped structures	3500	294	8.6	10.1
elliptic shaped structures	3500	350	10.2	8.3
rectangular shaped structures $l/w = 5$	3500	294	8.6	10.1
rectangular shaped structures $l/w = 3$	3500	232	6.8	11.3
rectangular shaped structures $l/w = 1$	3500	212	6.2	12.4

number of waves that contribute to the total damage. This is caused by the overtopping flow velocities being squared as well as the critical velocities. The higher the velocities are the larger the difference in contribution to the damage number for each wave is. Also the influence of the change in shape of the front flow velocity distribution is a factor that contributes to this. The contribution of each wave is larger for larger amplification factors (less steep distribution) and therefore less waves are required to result in the same damage number. And therefore also the percentage of critical overtopping waves. This is what can be seen in figure 5-27.

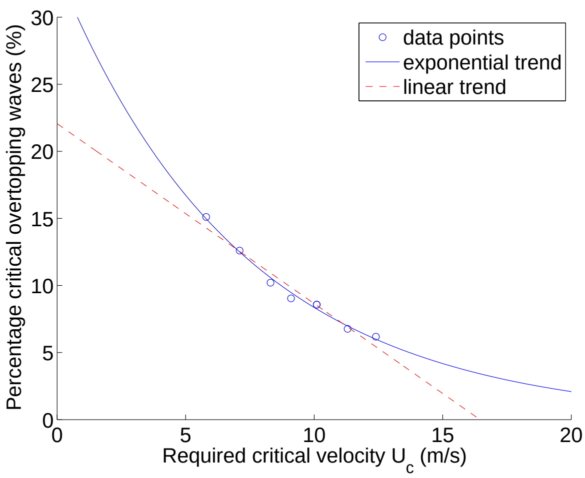

Figure 5-27: Percentage of waves contributing to damage number D =3500 m^2/s^2 for different required critical flow velocities, given 1/10 000 year

The above procedure has also been executed with a storm duration other than six hours. A three hour storm duration has been considered. This leads to the results as presented in table 5-6.

Table 5-6: Critical flow velocities of the different locations/discontinuities, storm duration 3 hours

location/type	$D_{failure}$ $[m^2/s^2]$	N_{cow} [-]	P_{cow} [%]	U_c $[m/s]$
crest	6000	621	36.2	4.9
crest small damages	3500	464	27.0	5.3
landward slope	6000	446	26.0	7.4
landward slope small damages	3500	318	18.5	7.8
Holes landward slope $<$ 15 cm	3500	318	18.5	7.8
transition landward slope and berm	3500	281	16.4	8.6
gradual transition dike segment geometry	3500	318	18.5	7.8
trees on crest $>$ 15 cm	3500	346	20.2	6.7
trees on crest $<$ 15 cm	3500	464	27.0	5.3
poles without foundation	3500	233	13.6	9.7
small external structures	3500	318	18.5	7.8
buried structures no risk of exposure	6000	446	26.0	7.4
cylinder shaped structures	3500	233	13.6	9.7
elliptic shaped structures	3500	318	18.5	7.8
rectangular shaped structures $l/w = 5$	3500	233	13.6	9.7
rectangular shaped structures $l/w = 3$	3500	206	12.0	10.8
rectangular shaped structures $l/w = 1$	3500	184	10.7	11.9

It is interesting to see that if the storm duration reduces, also the required critical velocity reduces. The reduction is approximately 0.5 m/s for most of the points under consideration for a reduction of the storm duration from 6 to 3 hours. For some 0.6 or 0.4 m/s. The percentage of waves that contribute to the damage P_{cow} increases. This is expected, because the number of incoming waves during a storm are less than with a storm duration of 6 hours. The number of waves that contribute to the damage are decreased as well. At first glance this seems strange, if the relation between the number of waves and the velocity per wave would be a linear relation it would be strange. With the distribution derived here it is not. The larger the percentage of waves is, the larger the relative contribution of the larger waves is. This means that the damage caused by each overtopping wave is larger than for a smaller percentage of waves contributing to the damage. This in combination with the fact that the distribution simply consists of less waves, makes the influence of each wave large as well. The velocity difference between each wave is higher. The reduction in critical velocity is also expected, because there are less waves which will result in a larger velocity.

To see whether or not a certain trend is visible this calculation has been done for a storm duration of 1, 2, 3, 4, 5 and 6 hours and for each critical point that results in a different required critical velocity. The resulting trends can be seen in figure 5-28.

The amplified flow velocities at the slope, so the rectangular structures with $l/w = 1$, rectangular structures with $l/w = 3$, transition between slope and berm, poles without foundation and even the slope velocity that is not amplified, all show similar trends for the influence of the storm duration for a damage number of 3500 m^2/s^2. If the damage number is increased to 6000 m^2/s^2 the trend differs. The increase for short storm duration is larger for the damage number higher than D=3500 m^2/s^2. For longer storm durations this trend is less different. The trend for the slope and crest also shows similarities. So it seems to be the case that the location or amplification of the flow velocity does not have an influence on the influence of the storm duration. The damage number does have an influence on the trend. The increase in required critical velocity is larger for a larger damage number, therefore the influence of the storm duration is larger for a larger damage number. Depending on the required critical start velocity the trend can be given by the power equation:

$$U_c = a * T_{storm}^b + c \tag{5-1}$$

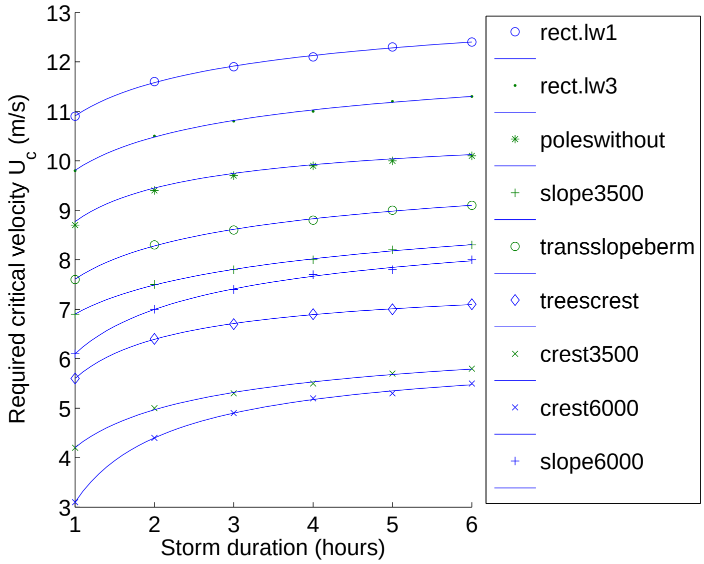

Figure 5-28: The influence of the storm duration T_{storm} on the required critical velocity U_c

With a, b and c are fitting constants, in this case a and b are constant for each velocity on the slope and crest for a damage number of 3500 m^2/s^2 and c differs for each point. The shape of the trend lines might differ for other distributions of overtopping waves, so with a different relative freeboard. Because of the lack of data the values of a and b cannot be given with any certainty. However, it is important to realize that the trend is similar for the same damage number for each point on the crest or slope and the damage number itself has an influence on the trend.

The required critical velocity is more sensitive for shorter storm durations than for longer. The larger the damage number, the steeper the trend line will be for shorter storm durations. This means that the required critical velocity is more sensitive for the storm duration for larger damage numbers.

With the required critical velocities as determined in this section the required top layer can be determined using the procedure of Box F.

5-1-7 Box F: translate critical velocity into required top layer

The results from Box E as described in section 5-1-6 have to be translated into certain top layer protection measures. This means that the required critical velocity is the parameter which the landward slope top layer should be able to withstand. Depending on the velocity different options are available to apply as revetment.

This procedure as depicted in Box F in figure 5-29 starts with the required critical velocity U_c from Box E. The next step is to find out if the allowable critical velocity of the slope U_{cslope} has already been calculated or has been tested with the wave overtopping simulator. If this is the case the resistance of

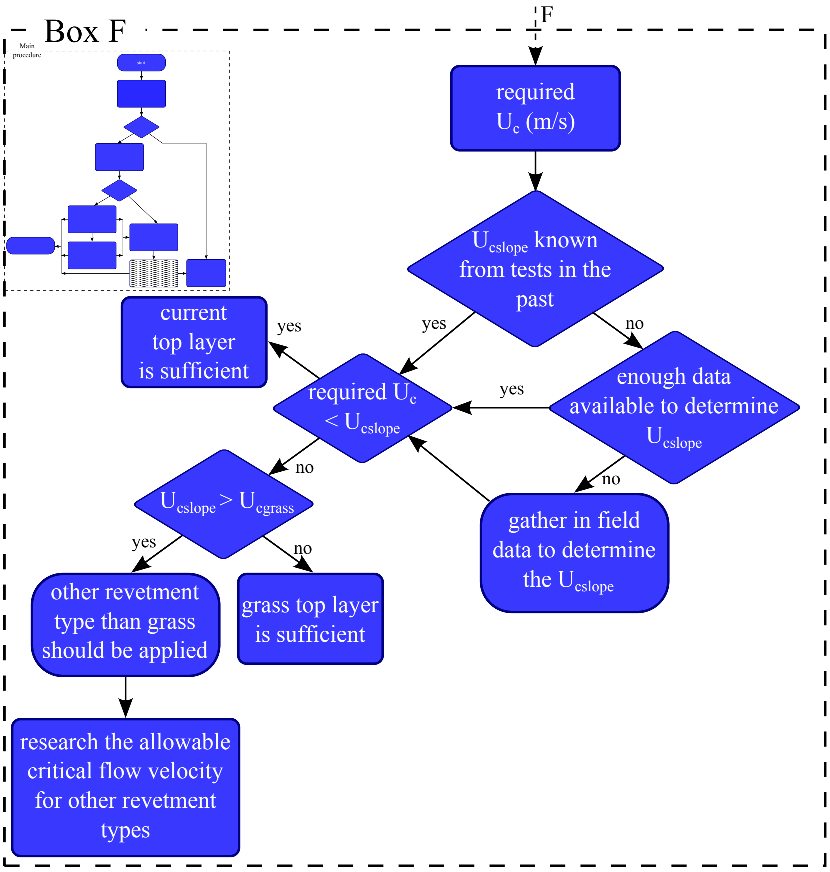

Figure 5-29: Box F: translate critical velocity into required top layer

the current slope is known. If this is not the case, the data in order to calculate the resistance of the grass slope with equation 4-53 should be gathered. This data can be gathered with the grass tension devise or by estimating the number of roots within the grass layer. Another way of acquiring this critical velocity is by simply test it with the wave overtopping simulator.

If the allowable critical velocity of the landward slope is known it should be checked if the required critical velocity is smaller than the allowable critical velocity. If this is the case, the current top layer has a sufficient level of protection. If this is not the case, it should be checked whether or not it is possible to improve the critical velocity of the grass layer, if this is the case the grass top layer can be made sufficient. If this is not the case another revetment type than grass should be applied. However, due to the lack of knowledge on other types of revetment for pulsating wave overtopping, research should be done on other revetments than grass.

Application of Box F to Afsluitdijk

When box F as presented in 5-29 is used to see what the required top layer of the Afsluitdijk could be the devision between the different critical points has to be made. To start with the required critical velocity resulting for a crest with small damages, U_c is 5.8 m/s. When looking at the second step in the procedure of Box F, it is to see if tests are done. Form the literature study as presented in chapter 2 the Afsluitdijk has been tested and the resulting critical velocity of the slope is larger than 6.3 m/s. This is larger than the value of 5.8 m/s that is required, this would mean that on the crest the current grass layer would be sufficient to protect the slope.

If the velocity along the slope is taken into account, the required critical velocity is 8.3 m/s. This is far beyond the allowable critical velocity for the slope of 6.3 m/s. Therefore the current grass layer is insufficient for the load by wave overtopping. For the velocities around objects this would also be insufficient. The next question is if the quality of the grass layer can be improved so that the allowable critical velocity U_{cslope} can be increased to a value higher than 6.3 m/s. The quality of the grass on the Afsluitdijk was already given the verdict ‘good’. This means that the grass quality probably cannot be increased by better maintenance. *Therefore a different revetment type should be used for the slope of the Afsluitdijk and further research should be conducted.*

In table 5-7 this has can be seen for all the different critical points.

Table 5-7: Grass or other revetment type for different critical points

location/type	U_c $[m/s]$	grass	other
crest	5.5	x	
crest small damages	5.8	x	
landward slope	8.0		x
landward slope small damages	8.3		x
Holes landward slope $<$ 15 cm	8.3		x
transition landward slope and berm	9.1		x
gradual transition dike segment geometry	8.3		x
trees on crest $>$ 15 cm	7.1		x
trees on crest $<$ 15 cm	5.8	x	
poles without foundation	10.1		x
small external structures	8.3		x
buried structures no risk of exposure	8.0		x
cylinder shaped structures	10.1		x
elliptic shaped structures	8.3		x
rectangular shaped structures $l/w = 5$	10.1		x
rectangular shaped structures $l/w = 3$	11.3		x
rectangular shaped structures $l/w = 1$	12.4		x

What these other revetments will be is an open ending up to this moment. There are several options when looking at literature, so would an artificially reinforced grass mat, the use of geotextile, concrete block revetments, asphalt or elastocoast be a possible solution. However, RWS wants the dike to have a natural appearance so a grass revetment would be preferred. If another protective layer is chosen probably a layer on top of the protective layer will make sure that the Afsluitdijk has a natural appearance.

It seems interesting to see for which points grass is the best solution and just apply grass only on these points and a different revetment on the other points. The expectation is that the transition between the grass revetment and the hard revetment will increase the required critical velocity more than only 0.5 m/s and such a transition is not suitable for the case of the Afsluitdijk as well. A uniform revetment type along the dike would be preferred.

Chapter 6

Required investigations

Another part of the design procedure and therefore of the results of this thesis are the required investigations that should be executed in order to be able to complete the Main Design Procedure as presented in chapter 5, section 5-1-1 to section 5-1-6. In this chapter these investigations will be described. The investigations are only required if the procedure leads to one of these investigations otherwise the procedure can be completed without these investigations. This required investigations will be described for each box that the required investigations appeared from. The subjects of the investigations per box are:

Box C
- Development of wave overtopping flow on the landward berm
- Gradual transition between slope and horizontal
- Abrupt transition between geometry of dike section
- Parallel revetment transitions
- Perpendicular revetment transitions

Box D
- Influence of bushes on the flow
- Damage as an effect of large external structures

Box F
- Resilience of revetments other than grass

These subjects will be described in the sections below. A description of the subjects of the required investigation will be given as well as some ideas on how to execute these investigations. For the case of the Afsluitdijk all these investigations will have to be executed in order to complete the procedure. For other dikes these may not be required.

6-1 Resulting from Box C

Development of the flow on the berm of the Afsluitdijk and the IJsselmeer slope

The properties of the overtopping flow on the crest and the landward slope of the dike can be estimated quite well. However, the development of the flow velocities and depths on the berm and the IJsselmeer side slope cannot. Currently there is no data available on the development of the flow on the berm. Because most of the Non Water Retaining Object(s) (NWO)'s are present at the berm of the Afsluitdijk the required critical velocities might be overestimated with the current methods. The expectation is that the velocities will reduce on the horizontal berm. This is due to the horizontal orientation of the berm and because of the expected lateral spreading of the overtopping wave. The testing can be done in two ways, the first way is to test on the Afsluitdijk it self with the wave overtopping simulator. Due to the presence of the highway A7 this might be a problem, because that has to be shut down and probably will be damaged. Another way of testing this can be by model testing or by building a full scale dike section in an open available area, like 'De ijkdijk'[1]. A real life dike should be build in order to see what the flow velocities are. The advantage of this is that the current dike is not damaged and there are no scaling errors which can occur during model testing. A computational model might be interesting as well. However, before the results can be used such a model should be calibrated and validated. For the crest and landward slope this is possible because a lot of data is available. For the berm there is not very much data jet.

Gradual transition between slope and horizontal

If the transition between slope and horizontal is gradual enough the influence on the overtopping flow is negligible is a statement that can be found in various reports and also in this thesis. According to the method used in order to derive the amplification factor for the transition between the slope and horizontal part of the landward side of the dike the influence on the flow is negligible if the slope has an angle of 1:4 or smaller. If this is the case is still questionable. So first of all research should be done to determine whether or not this criterion is true. Another question is, if the angle for which the slope has no influence is known, how can that be used. This seems very obvious, just create the landward slope of the dike with an angle lower than this angle. Due to the fact that on present dikes, and also for the Afsluitdijk, the length that can be used to perform such a slope is limited this might not be a good solution. The following question would be, how can a steeper slope be created with a rounding of the transition so that the transition still would have a negligible influence on the flow? This can be done in two ways, the first one is to execute the transition with a certain radius $R_{transition}$ and the second one is to make a certain transitional slope for which the angle with the original slope (ϕ_{up}) and with the original berm (ϕ_{down}) is smaller than the allowable slope angle as shown in figure 6-1.

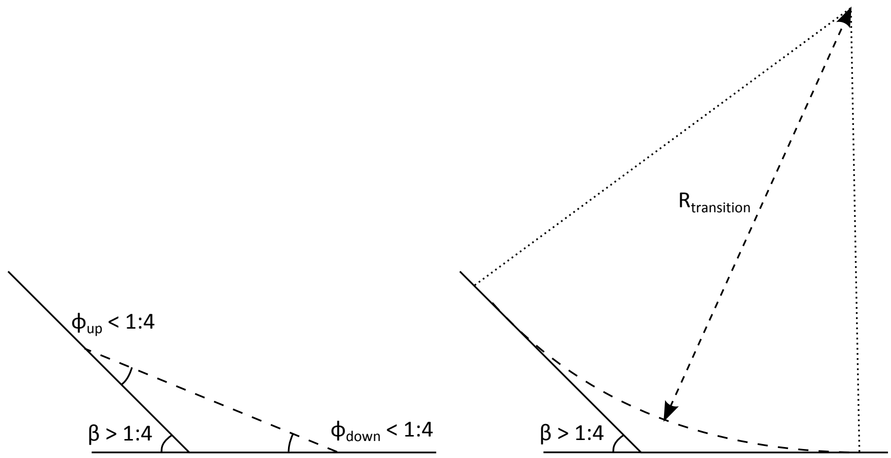

Figure 6-1: Gradual transitions

The second question would then be, how long should such a transitional slope or the radius be? These are all question that should be answered before a statement about the graduality of the transition can

[1] http://www.ijkdijk.nl/nl/

be made. Research should be done to answer all these questions. Again model testing can be done on smaller scale or full scale. On the Afsluitdijk it self several test section can be build and tested with the wave overtopping simulator to see what the damage development is during large volume wave overtopping events. Also here it might be interesting to use computational models, in order to simulate the flow.

Abrupt transition between geometry of dike sections

Along almost every dike there are transitions in geometry between different dike sections. This means that the cross section changes in width or in height. If this transition is performed over a long stretch, it can be stated that it is a gradual transition. How long this stretch has to be is unknown at this moment. The expectation is that the flow over the dike is not influenced very much by these type of transitions and that it has to be abrupt to have an influence. From model testing with different lengths of stretches the influence of the flow can be seen very clearly. Model testing probably is sufficient. For the case of the Afsluitdijk the stretches have a length of approximately 100 m. The expectation is that the flow is not very much influenced by this type of transition. The research to this subject would not have to be prioritized.

Parallel revetment transitions

This type of transition between revetments parallel to the slope is not found often. Therefore the research to such a revetment type transition would not have priority. However, during testing with the wave overtopping simulator on an asphalt layer along the edges of the test section small damages occurred even though the test section was width limited by wooden plates. It is uncertain what happens if this wooden plates would not have been positioned there. The expectation is that scour around the edges would occur, due to the transition between a hard and a soft revetment. Even if the transition would be between two hard revetments damage is also expected at these location due to water infiltration in between such a transition. Pressure can build up (locally) under the revetment causing damage or uplift. The effects are not well tested or described up to this moment. If such a transition is present research should be done on the effects of such a transition. Probably with a well executed transition the damage can be limited or prevented. This is for instance by applying a good filter layer under the revetment. These kind of solutions should also be tested. Probably model testing will suffice.

Perpendicular revetment transitions

Transitions between different revetment types perpendicular to the slope occur a lot more than the above described parallel revetments. The expectation is that these kind of transitions do have more influence on the overtopping flow. Also from testing with the wave overtopping simulator, actually when testing wave run-up, the run-down accidentally bumped into a asphalt revetment layer, which was instantaneously lifted up a couple of centimeters as told by J.W. van der Meer and described in appendix A-2. The cause of this was likely to be initially erosion of the soil in front of the asphalt layer, after which the wave front collided into the erosion hole and pressure build up could occur. Also a lot of paving was present during testing with the wave overtopping simulator and almost always initiation of erosion occurred at these points after which the pavements where undermined. It is important to realize that these pavements were not designed to handle overtopping water. This means that there were no filter layers present which can prevent pressure build up or washing out of the sand. If these measure are taken the damage might be prevented or limited. When applying these kind of transitions this should be tested extensively. This can be done with model testing or full scale testing in order to prevent scaling errors. When research is being done it is important to look at the different locations where the transitions can be present. The transition can be on the same location as the transition from slope to berm, it can be on the slope or on the berm. The location of the transition can be of great influence. Furthermore the transition can be from soft to hard, from hard to soft or between two different hard revetment types. These are all aspects that should be taken into account during the investigations.

6-2 Resulting from Box D

Influence of bushes on the flow

The presence of vegetation does have an influence on the flow and on the strength of the revetment. A variety in the presence of different length of roots has a positive effect on the strength of a vegetative top layer. If the vegetation becomes to large it will cause an increase in the flow velocity (see section 5-1-5 on trees). However, when the vegetation becomes smaller and looks more like bushes. The influence on the flow is unknown. It will most likely cause an increase of the turbulence within the flow. Turbulence most of the time is the initiator of erosion and it can cause an increase of the erosion as well. On the other hand, protective measures round bushes are difficult to take due to the presence of roots in the soil. When a hard revetment type is required, the advise is not to allow any bushes on the crest, landward slope or berm. When a soft revetment complies, the advise is to prevent the presence of trees and bushes as well. However, when for certain reason these will still be present, the influence of bushes should be tested. Not only the influence on the flow but also the influence on the strength of the soft top layer. The effects of roots and vegetation are difficult to scale, therefore full scale testing would be advised in this case. Another important aspect to realize is that when a dike is near other vegetation, that weeds and small bushes can easily grow on the dike so even when initially no bushes or vegetation is present on the dike, a lack of maintenance can cause it to be present over time.

Damage as an effect of large external structures

External structures will most likely be flushed of the dike by the overtopping wave if the volume is large enough. If structures are large enough they could cause a certain amount of damage to the dike or when really heavy would not be flushed away and may have to be treated as buried but visible structures. However, for what sizes and what sort of weights damage would occur to the dike is unknown. An estimate can be made based on common sense and maybe engineering judgment. The amount and sincerity of the damage is also dependent on the type of revetment. For grass damage will probably occur sooner than for a stone revetment. This can be tested quite well, by making an inventory of external objects that can be expected during the lifetime of the dike. Based on engineering judgment and common sense the devision can be made in objects that would cause no harm and objects that might harm the dike. For objects within the last category simple testing can be done by pushing these objects down a test slope and see what happens to the revetment. Also the effect of non protective paving should be treated as an external structure. These can be cycling paths, roads, sidewalks etc. These are not designed to withstand large amounts of overtopping water. No protective measures are taken to prevent erosion along the edges or prevent pressure build up or the formation of gullies under these kind of structures. This should be tested when large amounts of water are expected to overtop the dike. For instance on the Afsluitdijk the A7 seems to be a protective layer, but was never designed to withstand large amounts of water. It is however present at the berm where the flow velocities most likely already have been decreasing. If the highway during extreme 1/10 000 year conditions is being flushed away, that is not a problem and can be repaired. However, if the berm will be eroded in a short time period and the bolder clay dam would become endangered it becomes a problem, because the residual strength of the Afsluitdijk has not been taken into account here. The asphalt layer of the A7 should have a protective function it self. The behavior of these kind of external structures is unknown at this moment, there is a large risk of damage and failure.

6-3 Resulting from Box F

Strength of revetment

Of all the investigations that should be performed in order to complete the design procedure as presented in chapter 5 the research on the resilience or strength of the crest, landward slope and berm top layer is the most important investigation. On the load there has been a lot of different research, a lot of measurement on the flow velocities in model and full scale testing. Both for water overflow and intermittent wave overtopping. The resilience against overtopping is not as well known as the load caused by it. The resilience against water overflow, which is a steady state process, is relatively well known and several protective systems have been developed to protect earthen dams against overflow. These are however emergency spillways with a limited width and discontinuities can be prevented. This in combination with the fact that wave overtopping is not a steady state process but a pulsating process makes the criteria for these kind of protective systems unsuitable. Up to certain amounts of wave overtopping water the resilience of the top layer can be determined quite well together with the required top layer. These limits are: an average wave overtopping discharge q of up to 30 $l/s/m$ and depending on the top layer grass quality the resilience can be up to a critical velocity U_c of 6.3 m/s (for the Afsluitdijk). If these flow velocities are exceeded another type of protective top layer should be applied. Asphalt and elastocoast have been tested and no damage occurred up to 125 $l/s/m$. So if the velocities appear to be higher than the grass on the dike is able to handle, another protective top layer is required. Currently the knowledge on the resilience of these layers is limited. Therefore if such a measure has to be chosen it should be tested extensively. The advantage of non vegetative materials is that it can be scaled better than vegetative top layers. This means that model testing will become an interesting option. For instance with the Delta Flume or other wave flumes. For these artificial top layers also testing with the wave overtopping simulator in Colorado is an option. This simulator is able to simulate extreme volume wave overtopping events. The disadvantage is that it is a stationary simulator, this in contrary to the overtopping simulator in the Netherlands and Vietnam. But these cannot simulate extremely large wave overtopping volumes. For testing different revetments the simulator in Colorado can be interesting, because the controlled environment is only a disadvantage for vegetative top layers. For artificial layers, it might be an advantages because the trays can be switched quickly and testing can be executed in a short time span for a lot of different revetment types. The focus of these test should be on the critical points as mentioned in this report. Another important point of focus should be the pulsating character of wave overtopping. This appears to be a destructive characterization of wave overtopping. Also the effect of over pressures due to the wave front might be an interesting aspect to look at.

6-4 Probabilistic calculation

A probabilistic calculation can be an interesting way to see which parameters do have a large influence on the resilience against wave overtopping. An attempt to perform a probabilistic calculation has been done and can be seen in appendix D. The choice has been made not to present the whole calculation here but to add it to the appendix section and show some limited results here. From the procedure as described in section 5-1 it appeared that there is not a formula possible to couple the load and the strength. Therefore the probabilistic analysis only concerns the overtopping discharge q which eventually leads to the load caused by wave overtopping. This probabilistic part can be seen as an addition to the above procedure in order get an overview of which required investigations is the most efficient to execute. This is only a first attempt to see whether or not this will work with the current knowledge and formulations.

The calculation has been done with a Monte Carlo approach, which is explained in the appendix as well. From this approach is appeared that the overtopping discharge with a probability of 10 % would be advised to use, the value is 240 $l/s/m$. The sensitivity of the overtopping resilience for the sea state and geometry of the dike could not be given because a formulation between this load and the resilience of the dike could not be given. If the knowledge would be sufficient to do so, it would be advised to use a probabilistic analysis to analyze the influence of all these parameters. The research that should be executed could also be valued to the importance it has on the outcome of the calculations. The details of the probabilistic calculations can be found in appendix D.

Part III

Reflection on application and derivation of Design Procedure

Chapter 7

Discussion

In this chapter the used methodology will be evaluated and discussed. Some of the assumptions that have been done in order to get to the procedure or to complete the calculations can be criticized, which can be found here. The subjects that are under discussion are: the amplification factors that are used to determine the velocity increase at discontinuities, the allowable damage numbers $D_{failure}$ that are used in order to determine the required critical velocity U_c, the 1/10 000 year conditions that have been used as boundary conditions for the calculations, the concept of the overtopping resilient dike and the ability to apply the proposed design procedure to other dikes. These are all subjects that have certain limitations and therefore should be discussed.

7-1 Safety norm of 1/10 000 per year

As already partly explained in section 3-7 the Afsluitdijk has to be able to withstand conditions that belong to an annual probability of exceedanc of 1/10 000 per year. This probability of exceedanc is the smallest that is currently allowed in the Dutch water safety system. As explained in section 3-7 the Minister of Infrastructure and Environment writes that she is in favor of a risk based approach to be able to give a value for the consequences of a breakthrough of a dike. For the Afsluitdijk this is a difficult discussion. The Afsluitdijk is currently treated as a primary sea deference. But should it be treated like that?

The Afsluitdijk connects two dike ring areas and protects several other dike ring areas against high waters. However, not directly because the hinterland of the Afsluitdijk is primarily water which is surrounded by other dikes. An argumentation could be that a breakthrough of the Afsluitdijk is not as fatal as it looks at first glance. The dike will not disappears completely during one storm or even a couple of storms. Even if a breakthrough occurs, the outside water level will be dampened by the body of the dike that is still present along the stretch of the Afsluitdijk, this dampening will also occur for the incoming waves which will break on the remaining body of the dike. So the dikes that are behind the Afsluitdijk will not suddenly be exposed to the water levels and wave conditions of the Waddenzee. The advantage of the Afsluitdijk is that on the 'landward' side there is water instead of land that floods, so the damage only occurs to the dike it self, or by a secondary breakthrough. This in combination with the fact that the Afsluitdijk is a very wide dike body and certainly will have a certain residual strength, which is currently not taken into account, makes it a point of discussion whether or not the 1/10 000 year condition is the right criteria for the Afsluitdijk. In addition to this, by application of the risk based approach, that most likely will be introduced to the Dutch water safety system, the probability of exceedanc will most likely be lowered due to the relatively low consequences of (partial) failure of the Afsluitdijk.

It would therefore be advisable to see what happens to the conditions (water level, wave height and period) if the probability of occurrence will be lowered and what this would mean for the overtopping discharge q, volume V and velocities on the crest, slope and berm. If the influence of a lower but still acceptable probability of failure is large enough to reduce these amounts to values that the current Afsluitdijk is able to withstand it can be worthwhile for Rijkswaterstaat (RWS) to reconsider the probability of exceedanc or to wait on the outcome of the risk based approach to see what the actual risk of the Afsluitdijk is before the decision is made to strengthen the dike according to the concept of an overtopping resilient dike. On the other hand if the influence of the probability of occurrence is not significant for the overtopping discharge, volume or velocities, the decision should be made to strengthen the dike, as it has been done at this moment.

A final remark should be made concerning the choice of RWS to maintain the current probability of occurrence. The procedures for testing and reinforcing the current water safety system of the Netherlands takes a lot of time (order of decades). If the current procedures to strengthen the Afsluitdijk will be put on hold in order to wait for the risk based approach, that most likely will be introduced, it would take an even longer time than it already took for RWS to realize the strengthening and a lot of procedures would be executed over again. For RWS it is difficult to anticipate on the decision that are made by politicians in the future (short or long term). Therefore RWS has to work with the current safety standards which are also based on political decisions.

7-2 Amplification factors

For the increase of the velocity for several discontinuities an amplification factor α_M is given, these are based on testing with the wave overtopping simulator or are derived based on theory. Although these amplification factors result in a velocity that can be used to calculate the required critical velocity, the quality of these amplification factors is limited. The amplification factors give an approximation of the velocity increase and a bandwidth for the amplification due to a discontinuity with certain properties. The amplification factors will result in the right critical point being normative. However, the exact value might differ from reality.

The amplification factors currently can be seen as a framework for the different discontinuities and give boundaries for the different categories of discontinuities. These are currently the best guesses for an amplification factor. However, one has to be aware of the fact that to be entirely sure of what the amplification of the load (or flow velocity) is, model testing will result in more reliable values.

7-3 Damage number

To calculate the required critical velocity U_c an allowable damage number $D_{failure}$ has been assumed for the different discontinuities. Only two values for $D_{failure}$ are used in this thesis: 6000 m^2/s^2 and 3500 m^2/s^2. For a smooth slope and for a slope with small damages, respectively. In reality a smooth slope is rare, practically every slope has some small damages. Taking also into account that the influence of the decrease of the damage number to 3500 m^2/s^2 only has a small effect on the required critical velocity U_c it is advised to use a damage number of 3500 m^2/s^2 for all slopes, just to be on the safe side at all times. For the research in this report it is required to use the damage number of 6000 m^2/s^2 to be able to see what the influence is of the damage number on the critical velocity. In the design procedure also a damage number of 6000 m^2/s^2 is advised for some smooth slopes, if the user is convinced that the slope is smooth without any small damages, it can be used but is not advised.

Another aspect of the damage number is that the criteria is based on several tests executed with the overtopping simulator. The damage number has been determined using the calculated resistance of the top layer and the measured velocities. The damage number is only based on tests executed on grass. It is uncertain whether or not this also accounts for other revetment types. Maybe even the basis on which the damage number is calculated, an excess of shear stress is not a valid assumption for hard revetments like asphalt or concrete. Maybe the excess of work, so velocity to the power three, is a better measure here. The opinions of experts are divided between whether to use the excess of work or the excess of shear stress as a criterion for the damage number as such. Therefore before using the damage number as a criterion for hard revetments, this should be verified using the wave overtopping simulator. On the other hand this damage number is a criterion for the amount of damage and if the critical velocity becomes larger (which happens with other revetment types than soft soil with a grass top layer) the criterion might be the same as for grass, small damages are small damages for both grass or hard revetment types. In short: the damage number should be used with care.

7-4 Application to other dikes

The proposed design procedure as presented in chapter 5 is based on and applied to the Afsluitdijk. One of the main question is whether or not this procedure can be applied to other dikes. In principle this can be answered positively. The discontinuities as presented here are not only based on the Afsluitdijk, but have been determined based on an extensive literature study. In this study several dikes have been taken into account together with executed tests and expert opinions. The categorization of the Non Water Retaining Object(s) (NWO)'s is based on the case of the Afsluitdijk. On other dikes it is possible that there are certain objects that do not fit within one of the categories as presented here. These object have to be categorized themselves, or when an unique object is present a different amplification factor should be determined. Taking this into account the procedure can be applied to other dikes than the Afsluitdijk but always with care.

7-5 Concept of overtopping resilient dike

The concept of an overtopping resilient dike is the method of strengthening that has been chosen by RWS as a feasible and payable solution. However, during the execution of the thesis a retesting of the Afsluitdijk by Witteveen+Bos (W+B) showed that the whole outer revetment is insufficient to withstand the 1/10 000 year conditions as well. This means that the whole outer revetment should be replaced with a new revetment.

From the research executed in the thesis it appeared that a lot of additional research is required to make sure that the Afsluitdijk is resilient against large amounts of wave overtopping. The uncertainties in both load and strength of the dike are large. The velocities are so high that the grass top layer is insufficient for protection of the landward slope. This means that another protective layer should be found and the whole crest and landward slope should be covered with this new layer. What this new layer should be is currently unknown. Especially the pulsating character of the overtopping wave and the large amount of discontinuities that are present along the Afsluitdijk make it a difficult task to transform the Afsluitdijk into an overtopping resilient dike. Therefore it is possibly cheaper and more efficient, with the knowledge that the whole outer revetment should be replaced, to see how the amount of overtopping can be reduced to velocities that the crest, landward slope and berm of the Afsluitdijk are able to withstand. Based on testing this resistance is already up to a critical velocity of 6.3 m/s. Or as the Dutch would say: 'Voorkomen is beter than genezen' (prevention is a better solution than a cure). The overtopping resilient dike would probable be a better solution if the outer revetment does not have to replaced.

Chapter 8

Conclusions and recommendations

The conclusions and recommendations will be used to evaluate the results as presented in chapter 5 and 6 and to give recommendations on further research and how to approach such research. The questions stated in section 1-2-2 will be answered as well as the solution to the stated problem.

8-1 Conclusions

This section is a summary of the conclusions that have already been drawn in the report and answers the research questions as stated in chapter 1. To refresh the memory of the reader the goal of the thesis is repeated first:

Develop a predictive procedure in order to prove the landward slope resilience of the Afsluitdijk against large amounts of wave overtopping water

8-1-1 Preliminary conclusions

Before the conclusion is drawn whether or not the research objective has been reached, some preliminary conclusions will be drawn first. The first part is based on the conducted literature review.

The current design methods are aimed at determination of the crest height in order to prevent significant amounts of wave overtopping. These limited amounts are based on a certain strength. However, the current state of knowledge has not been incorporated jet to this design method and larger amounts of wave overtopping can be allowed.

Testing with the wave overtopping simulator has already shown that the Afsluitdijk is able to resist an average overtopping discharge of 30 $l/s/m$. Even amounts of 75 $l/s/m$ did not completely erode the top layer, but large damage did occur.

The wave overtopping simulator in the Netherlands as well as the one in Vietnam have a limited capacity of what can be discharged on a test section. The stationary wave overtopping simulator in Colorado is able to simulate larger overtopping volumes, but is unsuitable for testing of vegetative top layers due to the controlled environment the vegetation is grown in, it might be interesting to use this simulator to test other revetment types.

8-1-2 Subquestions

The first sub-question that has been formulated in chapter 1 is:

what are the main physical processes when it comes to the loads induced by large amounts of wave overtopping?

The load caused by wave overtopping is expressed as the average wave overtopping discharge q (l/s/m). Due to the non steady state character of wave overtopping this is only a good measure for the load up to 30 $l/s/m$. Above an average wave overtopping discharge q of 30 $l/s/m$ it is better to express the load as an overtopping volume V (m^3/m) per wave. This should be a distribution of an overtopping volume per wave.

A new method to determine this distribution has been applied in this thesis. This is a Weibull distribution with shape factor b (-) and scale factor a (m^3/m), the shape factor b is dependent on the relative freeboard R_c/H_{m0} and the outer slope angle α. The scale factor a is dependent on the shape factor b, the overtopping discharge q, the mean spectral wave period $T_{m-1,0}$ and the probability of overtopping P_{ov}. This method has been found in literature and differs from the old method which had a fixed value for b and another relation for a. This results in a smaller overtopping volume per wave giving a larger average overtopping discharge q. It is recommended to use this new method, because it takes into account more variables and results in a better fit for the overtopping volumes.

The wave overtopping discharge for the Afsluitdijk is approximately 160 $l/s/m$, this is caused by the small reduction of the wave run-up on the outer slope (between 0.9 and 1). This is because the berm is ineffective (water depth on top of the berm is large, waves do not 'feel' the bottom), the small incident wave angle and the low roughness of the outer slope. This in combination with a low relative freeboard results in large amounts of wave overtopping.

This distribution of overtopping volumes per wave can be converted to an overtopping front flow velocity U (m/s), with empirical relations based on tests done with the wave overtopping simulator. This results in a distribution of a front flow velocity per overtopping wave (N_{ow}). This velocity distribution is dependent on the sea state and on the geometry of the dike. This is the load at the crest.

The load at the slope can be determined based on the velocities and flow depths per wave on the crest as described above. These can be transformed with the angle of the landward slope, the slope length and the friction factor f. Recommended for f is a value of 0.01. This results in a distribution of the maximum front flow velocity U_{max} per wave on the slope.

The load of an overtopping wave is more difficult to handle than steady state overflow due to the pulsating character of wave overtopping. The pulsations result in a constant loading and unloading of the top layer, the top layer is therefore not able to settle against the load.

The load at the crest and landward slope can be determined quite well. Especially on the crest. The velocity development on the landward slope can be estimated with the crest overtopping values as initial conditions, this is however under steady state assumption and gives a good representation of the maximum flow velocity. The value of the friction factor f that is used is not very reliable at this moment in time.

What are (new) limit states that need to be taken into account when wave overtopping is allowed?

The limit states that are important when wave overtopping occurs are wave overtopping it self, with a larger allowable average wave overtopping discharge q than is currently allowed. The limit states caused by wave overtopping are infiltration and surface erosion. Infiltration can cause shearing of the landward slope, shearing of the landward slope top layer or pushing off the landward slope top layer. Surface erosion will lead to erosion of the landward slope top layer. If one of these occurs, the landward slope is considered to have failed. The failure of the landward slope top layer is now seen as the new limit state.

What are the critical points of the landward slope and berm?

The critical points on the crest, landward slope and berm are the point of maximum velocity on the slope and the discontinuities. The discontinuities can be divided in holes in the landward slope, transitions and Non Water Retaining Object(s) (NWO)'s. The transitions are the transition between the landward slope and berm, the transition between different types of revetment and the transition between different dike segments. For the NWO's a categorization has been made, based on the influence each point has on the load and the resilience:

- Vegetation
- Structures
  - Buried structures
  - Buried but visible structures
  - External structures
- Cables and pipelines
- Poles
  - Poles with foundation
    - $<$ 15 cm
    - $>$ 15 cm
  - Poles without foundation
    - $<$ 15 cm
    - $>$ 15 cm

These critical points are the critical points specifically for the Afsluitdijk. For other dikes other objects could be found, which might not fit within one of the categories. For these objects, a different amplification factor should be derived.

Are these critical points able to withstand large amounts of overtopping and how can that be proven?

The influence of each critical point on the load and resistance has been analyzed. The influence on the flow has been expressed with an amplification factor for the overtopping front flow velocity. These amplification factors α_M for the different critical points vary from 1 for poles without a foundation that are smaller than 15 cm to 1.5 for rectangular structures with a ratio between length and width of 1. The larger this ratio between length and width becomes, the lower the effect of the object on the overtopping flow. Holes $>$ 15 cm result in an amplification factor of 2.1.

Because the Afsluitdijk is an important water defense in the Netherlands, the maintenance can be assumed good enough for these large damage not to be present during design storms.

The maximum velocity on the landward slope can be calculated using a friction factor f. Depending on the location of the critical point, the velocity distribution on the crest or the distribution of maximum velocities along the slope is amplified with the amplification factor. To see whether or not the critical points are able to withstand the load caused by wave overtopping the distribution of overtopping waves has to be translated into a certain critical velocity U_c. Whether or not the top layer is able to handle these amplified flow velocities depends on the resulting required critical velocities.

The shape of the distribution of the overtopping front flow velocities per wave changes when the flow is amplified. The larger the amplification factor is, the less steep the distribution becomes. This means that the contribution of each wave increases with an increasing amplification factor. The difference between U and U_c becomes larger for less steep distributions. The number of critical overtopping waves therefore decreases due to the amplification.

The methods in order to determine the load at the crest and landward slope are good tools in theory. However, when applied to a real case as done here for the Afsluitdijk, it requires at least a sense of engineering judgment to be able to determine the right velocity distributions. The values for the crest and slope with small damages are the most reliable. When the amplification factor for the discontinuities is used, the location on the dike should be taken into account, this makes it difficult to estimate the right values, for example because the location of the maximum velocity along the slope is unknown. The estimations of the amplification factors are often only first attempts and therefore the reliability is not optimal.

What is the relation between the load and resistance parameters?

The coupling of the load and the resistance is based on the cumulative overload factor D, which is defined as the excess of the critical shear stress of the top layer per overtopping wave $\left(D = \sum_{i=1}^{N_{cow}} \left((\alpha_M U)^2 - U_c^2\right)\right)$. This is based on the principle of fatigue and takes into account the pulsating character of overtopping. The main difference with overflow is that the duration of the flow is not very important, only the number of times the critical flow velocity is exceeded.

It appeared that a decrease of the damage number from 6000 to 3500 m^2/s^2 only has a small effect on the required critical velocity. This in combination with the fact that small damages are hard to prevent, results in the advise to assume an allowable damage number $D_{failure}$ of 3500 m^2/s^2.

If the storm duration shortens, the required critical velocity decreases, a clear trend has been shown. It appeared that the larger the damage number, the larger the influence of the storm duration T_{storm} on the required critical velocity U_c will be. The sensitivity of the required critical velocity is the largest for short storm durations. For a value of the damage number a clear trend in the increase of the critical velocity for longer storm durations has been found.

The shorter the storm duration, the less overtopping waves will result in high front flow velocities. However, if the storm duration decreases, the number of waves contributing to the damage also decreases. This is because the percentage of waves contributing to the damage number increases for shorter storm duration. The relative contribution of each wave will also increase which means that less waves have to contribute to result in the same damage number. So for a shorter storm duration less waves will contribute to the damage.

For a value of the damage number a relation can be found between the required critical velocity and the percentage of critical overtopping waves that contributes to the damages P_{cow}. The shorter the storm duration becomes the larger this percentage is. This is expected because the same damage number has to be reached and the number of incoming waves is lower. For a larger percentage of critical overtopping waves the relative contribution of each wave becomes larger. This is due to the non linearity of the distribution of the front flow velocity per overtopping wave. The lower the required critical velocity is, the larger the percentage of waves that contribute to the damage.

When the required critical velocity has been determined, only a limited amount of waves contribute to the damage, these are only the waves which result in a higher front flow velocity than the required critical velocity.

Using the cumulative overload method means that the maximum occurring flow velocity is not equal to the required critical velocity U_c. The required critical velocity is lower.

What are the main physical processes when it comes to the resistance against large amounts of wave overtopping?

The pulsating character of wave overtopping is the main aspect looking at the resilience. For overflow certain protective top layers are available. The difference between overflow and wave overtopping is the pulsating character of the overtopping flow. The passing of the wave front results in a changing of the load in direction and in size. This means that the top layer is not able to settle against this load. This loading and unloading has analogies with the principle of fatigue. This can also be seen in the cumulative overload factor.

The resilience of the dike has to be able to withstand large overtopping flow velocities. Depending on the revetment types other properties determine the critical flow velocity. The resilience part of the process is the critical part in the design. A required critical velocity that the protective top layer has to be able to withstand can be determined quite well. However, the translation of such a critical velocity in a protective top layer is difficult if this required critical velocity exceeds the limits of grass. The critical velocity of grass can be the determined, with a theoretical method or based on tests done with the wave overtopping simulator. For other revetment types than grass it is not possible to determine the critical velocity for wave overtopping with the current methods other than testing with the wave overtopping simulator or in wave flumes.

For the Afsluitdijk the velocities that occur on the crest of the dike result in a required critical velocity of 5.8 m/s for a crest with some small damages. The current grass layer has an allowable critical velocity of 6.3 m/s, this means that if no other discontinuities would be present on the Afsluitdijk the current grass layer would be sufficient to protect the crest. The maximum velocity along the landward slope results in a required critical velocity of 8.3 m/s, the current grass layer is thus insufficient to protect the landward slope against large amounts of wave overtopping. As soon as the flow velocities on the crest and landward slope are amplified the resulting required velocities are higher than 6.3 m/s and the grass layer in insufficient to protect the Afsluitdijk. The maximum required critical velocity that is present along the dike is at a rectangular structure with a ratio between length and width of 1 on the landward slope, U_c that is required is 12.4 m/s which is very large. The current Afsluitdijk is therefore not able to withstand the amount of wave overtopping caused by the 1/10 000 per year conditions. The current available theoretical procedures are not suitable for the determination of the landward slope protection layers for the required critical velocities, this leads to the next research question.

For which parameters additional information is required and with what methods can these be determined?

As appears from the above conclusions, especially the resilience of the protective layers against pulsating wave overtopping requires extensive investigations. The other points on which additional information is required are:

- The development of the flow velocities on the landward berm.
- The graduality of the transition between the slope and horizontal
- Transitions between different type of revetment
- The influence of bushes on the flow
- Damage as an effect of large external structures

8-1-3 Final conclusion

The main research question is:

Can a predictive procedure be developed in order to prove the landward slop resilience of the Afsluitdijk against large amounts of wave overtopping water?

In this thesis an attempt has been made to develop a predictive design procedure in order to prove the resilience of the landward slope. The procedure that has been developed is able to prove the required resilience of the landward slope of the Afsluitdijk and shows that the current Afsluitdijk is unable to cope with the amounts of overtopping that occur. However, the procedure is limited by the current state of knowledge. The required critical velocities that occur as an effect of wave overtopping cannot be translated into a required protective top layer when the limits of grass are exceeded. This one of the key issues for the investigations that should be done. If the open endings of the procedure can be completed with the execution of the required investigations as presented in chapter 6, a complete predictive design procedure has been developed. With this the research objective has been reached. With the current state of knowledge the design procedure cannot be completed without performing the required additional research if the required critical velocities exceed the limits of grass revetments.

8-2 Recommendations

The recommendations in this section are focused on further research and the use of the results. In this way the results of the thesis can be used optimal and further research might be easier to execute. The recommendations are separated in general recommendations, recommendations on the procedure and recommendations for the Afsluitdijk.

8-2-1 general

The thesis focuses on surface erosion, however there are more problems that are caused due to wave overtopping, especially the influence of infiltration should be a part of further investigation. Infiltration can cause protective layers to shear or being pushed of, which will immediately reduce the strength of the top layer, therefore it can be a dangerous mechanism. Infiltration is partly caused by overtopping and partly by the water level on the Waddenzee side of the dike. The combined influence on the infiltration and with that the raise of the phreatic water line within the dike should be investigated further. Especially for protective top layers with a low permeability this might be a problem.

The residual strength, the strength after initial failure of the landward slope, of an earthen body as the Afsluitdijk might be large. Up till now this has not been taken into account. This is because there are so many uncertainties concerning the residual strength. However, it seems a waste not to take any residual strength into account. This might reduce the required strengthening of the dike considerably. Some research has been done on this subject, it would be nice if a certain minimum amount of residual strength can be calculated in order to take at least some of it into account.

It might be interesting to consider larger initial damages. This to take into account certain incidents, like explosion of cars, trucks or underground cables and pipelines. The presumption is that this leads to the use of residual strength, because the landward slopes top layer has already failed instantaneously.

The final general recommendation is that there should be an update of the guidelines on the design of dikes. Currently only very limited amounts of overtopping are allowed on which the design crest level of the dike is based. However, from testing it already appeared that up to 30 $l/s/m$ is an allowable amount of overtopping for each tested dike. This could be used as an allowable amount of average overtopping discharge in the guidelines for grass covered dikes, it is proven with tests. This should be done in combination with the limitations that the hinterland of the dike introduce. This would result in a reduction of the required height of the dike and therefore money can be saved on the strengthening of the Dutch water safety system. This money could be spent on the reinforcement of other parts of the water safety system.

8-2-2 Procedure

For other dikes the inventory of NWO's should be done again. Another variety in NWO's can be present on other dikes. The categorization might be sufficient for other dikes, but it can also be the case that the objects do not fit within the categorization presented in this report. This should always be checked to be sure.

The focus of the thesis lies on intermittent wave overtopping. For river and lake dikes, the process of wave overtopping might not be the dominant overtopping process, also surging overtopping flow and a combination of wave overtopping and surging overtopping flow can be important. This gives a different kind of load. From the research in this thesis it appeared that the intermittent wave overtopping is governing as load is concerned. Therefore it is worthwhile checking whether or not this has to be taken into account. A less resistant revetment might be required for river or lake dikes, which is cost reducing. This is especially interesting for other types of dikes than the Afsluitdijk, dikes where overflow occurs like river dikes. Overflow being less destructive is also observed at the summer dikes along rivers. A lot of water flows over these dikes, but little damage occurs. This is due to the absence of the pulsating character, which is present at wave overtopping flow.

In the thesis the new method in order to determine the distribution of the overtopping wave volume, in which the scale and shape factor of the Weibull distribution are determined in another way, has been used. A comparison between the old and new method has been made, this resulted in the fact that the new method results in a larger average overtopping discharge for the same conditions. This could mean that the overtopping discharge as determined by testing with the wave overtopping simulator is larger than originally was assumed and the tested dikes are actually more resilient to larger amounts of wave overtopping discharge. However, this requires a more comprehensive study than has been done in the thesis. It could be worthwhile to investigate this. For the results of the thesis this has a negligible influence because the overtopping velocities of the conducted tests would not change.

The above mentioned method in order to determine the distribution of overtopping wave volumes/velocities is based on a smooth outer slope. If the outer slope is replaced with a revetment with a higher roughness the distribution might change. If this is the case more research should be done on the effect of the roughness of the outer slopes on the distribution of overtopping wave volumes and velocities.

The friction factor f that has been used in order to determine the maximum velocity along the slope is based on steady state theory, which wave overtopping is not. It gives a good approximation for the maximum flow velocity along the landward slope, but the location along the slope is unknown. Also the assumed value of 0.01 is dependent on the type of revetment and has only been determined based on grassed slopes. Additional research for this friction factor should be done for other revetments than grass.

The effect of objects that are repeated at a short distance from each other, like the poles of the guardrail along the highway, should be investigated as well. When the flow patterns caused by these objects interfere with each other the effect of repeated objects can be larger than for a single object.

8-2-3 Afsluitdijk

The normative frequency of 1/10 000 per year is extensively discussed in the thesis. Whether or not this is the right condition for the Afsluitdijk depends on a lot of factors which should be investigated further. The load as determined in this thesis is based on this normative frequency, so if the conditions might reduce to an optimal normative frequency also the load most likely will reduce. This could save the government a lot of money. The risk approach might be a good way of solving this optimal point.

The current most common seaward profile of the dike has been taken as profile on which the calculated loads are based. While the thesis was ongoing, it appeared that the outer revetment of the dike is not sufficient to withstand the 1/10 000 year conditions. Therefore the outer slope revetment of the dike will also have to be strengthened, this strengthening gives an opportunity to change the amount of wave overtopping and can reduce the required protection level of the landward slope of the dike.

The final statement that should be made is knowing the Afsluitdijk is unable to deal with the velocities as a result of the 1/10 000 per year conditions, and much research has to be done on what strengthening measures should be taken, together with the fact that the outer revetment should be replaced as well this might not be the right way of strengthening of the Afsluitdijk. Reducing overtopping to the amounts the Afsluitdijk is able to deal with, as described above, is worthwhile investigating.

Appendix A

Interviews

In this appendix short reports on the interviews with Eric Regeling and Jentsje van der Meer are presented.

A-1 Eric Regeling 14-06-2013

Time:	10:00 tot 12:00
Date:	Friday 14-06-2013
Location:	Rijkswaterstaat Lelystad, Zuiderwagenplein 2
Company:	Rijkswaterstaat

Eerste beide even kort voorgesteld aan elkaar, wat Eric Regeling doet bij Rijkswaterstaat (RWS) en binnen het team Afsluitdijk. Hij geeft aan dat hij zich richt op het dijklichaam en niet op de constructies in de dijk en dat er al snel zo'n 30 tot 40 mensen binnen RWS zich bezig houden met het project. Hierna verteld hij de achtergrond van het hele Afsluitdijk verhaal, dat ze ongeveer in 2002 gestart zijn met de verkenning naar de Afsluitdijk. Vanaf dat moment was het wel duidelijk dat deze op lange termijn niet meer voldeed aan de eisen. In de eerste toetsronde primaire waterkeringen is de dijk nog niet meegenomen, vanaf de tweede wel. Waaruit naar voren kwam dat de dijk inderdaad niet voldeed. Na de derde testronde werd dit nog specifieker (en kritieker). Hierna is een marktverkenning is gestart en een aantal basisvarianten zijn ontwikkeld. Het doel van de verkenning was om met haalbare en betaalbare varianten te komen, de markt kwam echter met een integrale ontwerpvisie op de Afsluitdijk met allerlei varianten, waarbij door wat dan ook geld verdiend kon worden om de variant te doen slagen. Uiteindelijk is dit alles gepresenteerd in het rapport 'Dijk en Meer'.

De keuze van Rijkswaterstaat voor de overslagbestendige dijk

Vanwege de haalbaarheid en betaalbaarheid van alle varianten vielen allerlei exotische varianten snel af en bleven eigenlijk de muur op de dijk, verhogen en verbreden en de overslagbestendige variant over. Vanwege het feit dat verhogen en verbreden veel problemen zou geven met de oplossingruimte, natura 2000 en het om veel verzet gaat over 32 km lengte viel deze eigenlijk ook wel af (hoewel Eric Regeling aangeeft dat zijn voorkeur daar wel lag). Bij de variant met de muur op de kruin, verwachten ze problemen met de aansluiting van de muur op het dijklichaam en wilde dit liever voorkomen. De aanpasbaarheid van de varianten speelden hierbij ook een rol net als de esthetica (groene uitstraling en dergelijke). Dit alles meenemend en de kosten en faseerbaarheid daarbij opgeteld leidde tot de keuze voor de overslagbestendige dijk. De keuze was daarmee dus niet zo eenduidig als uit de Structuurvisie naar voren lijkt te komen.

De visie op de invulling van het concept overslagbestendigheid

Heel simpel gezegd, een profiel dat grote hoeveelheden overslag kan handelen. Ik gaf aan dat het mij opvalt dat in alle schetsjes en tekeningen de dijk volledig bekleed is met asfalt. Daar is bewust voor gekozen om de bedoeling duidelijk te maken, de precieze invulling is nog niet duidelijk vandaar deze beeldvorming. Het moet vooral een robuust concept zijn. De vegetatieve uitstraling aan het binnentalud speelt een belangrijke rol, dit zal waarschijnlijk zijn in de vorm van een soort overlaag waarin vegetatie kan worden aangebracht.

De aantoonbaarheid van het concept, wat houdt aantoonbaar in?

Met betrekking tot de aantoonbaarheid ging Eric Regeling gelijk over op de pragmatische invulling daarvan: Kijkproef met overslagsimulator. Het ontwikkelen van een formule waarmee ontworpen kan worden die dan door bijvoorbeeld ENW weer goedgekeurd wordt. Onzekerheid in de input parameters kan je eventueel ondervangen doormiddel van verdelingen van parameters dat geeft een bepaalde verdeling van de output, waaraan je een faalkans op kan hangen (probabilistisch dus). Tegenover probabilistisch ontwerpen stond Eric Regeling wel positief, het wordt al gebruikt, dus waarom hier niet? Hij geeft verder aan dat de dijk gewoon de 1/10.000 situatie aan moet kunnen en dat met betrekking tot aantoonbaarheid initieel falen al falen van de hele dijk is. Met reststerkte wordt dus niets gedaan. De dijk mag helemaal kapot gaan op een zwaardere situatie, want daar wordt hij immers niet op ontworpen. Op de vraag waarom er met de reststerkte niks gedaan wordt is het antwoord heel duidelijk, op dit moment is het simpelweg nog niet aan te tonen, er wordt wel onderzoek naar gedaan maar daar verwacht hij op korte termijn niet veel van, dat zal in de loop der tijd wel komen maar niet op tijd voor dit project. De aantoonbaarheid blijft gedurende het hele gesprek langskomen. Het woord waarschijnlijk wordt veel gebruikt , wat al aantoont dat er een hoop onzekerheid is met betrekking tot het onderwerp overslagbestendigheid. We hebben het ook gehad over de aantoonbaarheid van de sterkte van gras. Hij geeft aan dat er een rapport aan zit te komen, waarin voorgaande uitspraken worden gerelativeerd en waarbij hooguit 30 $l/s/m$ wordt toegelaten. Ik betwijfelde dit hardop aan de hand van een aantal voorbeelden en geef aan dat het met name gaat om de kritieke punten en niet zo zeer om de helling. Hij gaat daar in mee.
Het doel van Rijkswaterstaat is om straks de aannemende partij te kunnen controleren op het ontwerp van overslagbestendigheid en dat de aannemers/ontwerpers gebruik kunnen maken van het vooronderzoek als zijnde te volgen methode met betrekking tot de overslagbestendigheid (vooral voor W+B een belangrijk punt)

Het programma SBW (Sterkte en Belastingen Waterkeringen) en daarop volgende WTI 2017 (Wettelijk toetsinstrumentarium 2017)

Over dit punt hebben we het even kort gehad, Eric Regeling geeft inderdaad aan dat SBW en TOI onder de overkoepelende WTI 2017 zijn komen te vallen. Een overzicht van de programma's heeft hij zo niet, hij geeft wel wat toelichting op het WTI 2017. Als relevante onderzoeken van het programma geeft hij het piping en SBW erosie en grasonderzoek (welke ik al had gevonden). Daarnaast noemt hij ook SBW Reststerkte (tevens al gevonden) maar is gezien de aantoonbaarheid in zijn ogen niet heel erg van toepassing. Helpdesk water contacteren zou verhelderend kunnen zijn met betrekking tot SBW programma's.

Kritieke punten met betrekking tot overslagbestendigheid

Meeste punten zijn wel langs gekomen gedurende het gesprek en worden hier nog even benadrukt. Met name de knikken en de NWO's, De 'sleepsterkte' en eventueel golfklappen spelen een rol. Met betrekking tot de knikken geeft Eric Regeling aan dat blijkt als deze maar genoeg worden afgerond dat deze dan minder kritiek worden. Ik bevestig dit aan de hand van enkele onderzoeken maar geef tegelijkertijd aan dat nergens uit blijkt hoe deze afronding dan precies moet zijn. Daar was hij het ook mee eens.
Een belangrijk punt is volgens Eric Regeling dat je op de Afsluitdijk te maken hebt met een berm van 40 meter lang. En hoe de tong van de overslaande golf zich spreid op de berm. Zijn de stroomsnelheden aan de zijde van het IJsselmeer net zo kritiek als aan de kant van de tuimeldijk? Kortom de relatie tussen de spreiding en de belasting ten gevolge van de overslaande tong. Eric Regeling wijst me ook nog op de verhoudingen, de twee meter speling tot de kruin is echt heel weinig, zeker als je meeneemt dat de golfhoogtes groter zijn dan 3 meter. Dat betekent dat de golf er zelf al bijna overheen slaat, zonder Run-up mee te nemen. Daarna geef ik aan dat de kruin zelf ook maar 2 meter breed is wat ook heel weinig is. Eric Regeling zegt dat al die dingen afzonderlijk geen probleem zijn, maar de optelsom de problemen geeft. Dat het hiermee een gedurfd concept is valt dan ook niet te ontkennen.

De door mij geformuleerde probleem en doelstelling (concept), en daarop volgende oplossingmethode

Mijn probleem en doelstelling en onderzoeksvragen heeft Eric Regling even kort bekeken en vond de probleemstelling goed en alles omvattend. Hij gaf aan het eens te zijn met mijn opbouw, de relatief brede probleemstelling en doelstelling en doormiddel van je onderzoeksvragen meer nuance aanbrengen. Hij pakte mijn vraag met betrekking tot de infiltratie eruit en gaf aan dat dit inderdaad een probleem is maar of het de vraag is of je dit als losse vraag moet behandelen. Hij zou niet kijken naar het proces van infiltratie zelf, maar meer naar wat áls het optreed en het schade veroorzaakt, wat heeft dat voor gevolgen op je erosie. Ik was het daar eigenlijk wel mee eens dat dit MIJN startpunt moest zijn. Dat Witteveen+Bos hier wel aandacht aan moet besteden dat werd ook wel duidelijk. Het blijft toch een belangrijk aspect, zeker met het feit dat nergens bij infiltratie onderzoeken op de afsluitdijk de hoogte van het freatische vlak sterk toeneemt zonder de hoge buitenwaterstand mee te nemen. Eric Regeling probeert tijdens het gesprek goed onderscheid te maken tussen mijn afstuderen en de doelen van Rijkswaterstaat (en dus W+B). Veel suggesties en ideeÃńn komen voorbij. Hij geeft een mathematisch model als suggestie en vind de beschouwing van de standaard secties opzich niet verkeerd, zeker gezien de lengte van sommige secties. De probabilistische benadering sluit hij zeker niet uit. Het zou mooi zijn als er uiteindelijk een methode (of nog mooier) formule uit komt waarmee bepaalde kritieke punten aantoonbaar bestendig gemaakt kunnen worden.

Waarop ontbreekt het nog aan kennis op dit moment, met name met betrekking tot de overslagbestendigheid en niet-waterkerende objecten eventuele verbreding

Een aantal zijn er al voorbij gekomen, zoals de ontwikkeling van de tong op de berm. Daarnaast heeft hij al benadrukt dat er twee kanten aan het verhaal zit, aan de ene zijde sterkte aan de andere kant belasting. Op beide vlakken zijn in zijn ogen nog kennisleemtes te vinden. Met name de stroomsnelheden van de tong en de stroomdieptes zijn hierin volgens hem belangrijk. Ik geef aan dat schade door overtopping een vermoeiingsprincipe is, dat de kleine volumes relatief weinig bijdragen aan de schade. Er is een bepaalde kritieke snelheid waarboven de golven invloed gaan hebben, De cummulative overload factor passeert, waarbij er nog veel onzekerheid inzit en nog niet goed is aangetoond is welke factoren er nou gebruikt moeten worden, voor de stroomdieptes en frontsnelheden. Hierin zitten we helemaal op een lijn.
De turbulentie en concentratie van stromingen rond NWO's is tevens een belangrijk onderzoeksaspect.

Net als de afronding van de knikken. Sterkte van gras is belangrijk. Verder geeft Eric Regeling aan dat er een rapport bestaat met een Gevoeligheidsanalyse naar de geotechnische gevolgen van de basisvarianten. (als ik dit niet kon vinden mocht ik hem er over mailen.)
Er is weinig bekend over de overslaande volumes van golven met een $H_s > 3m$. Aangezien we hier daar wel mee te maken hebben is dit een belangrijke kennisleemte volgen Eric Regeling.
Er wordt benadrukt dat parallellen trekken belangrijk is, denk aan havendammen, eventueel zelfs aan spilways. Als de dijk verhard wordt mag vanuit veiligheidsoogpunt GEEN vangrail meer worden toegepast, deze kan dan niet meer verplaatsen en verliest dan zijn fuctie. Er worden dan verschuifbare barrières geplaatst, deze kunnen natuurlijk weg spoelen bij grote hoeveelheden overslag.
De onzekerheid in de opbouw en sterkte van de Afsluitdijk is wat Eric Regeling betreft een hele belangrijke onzekerheid. Het is een extra argument om GEEN reststerkte te gebruiken.

Aan te bevelen rapporten en literatuur

- Derde toets primaire waterkeringen
- Dijk en Meer
- Structuurvisie toekomst afsluitdijk
- Nota ontwerpfilosofie Afsluitdijk
- Oplossingruimte varianten
- Nota van uitgangspunten en randvoorwaarden
- Hydraulische randvoorwaarden voor het ontwerp versterking Afsluitdijk
- Inventarisatie kennisleemtes
- Toekomst afsluitdijk antwoord op 5 onderzoeksvragen
- Factual report overslag metingen Afsluitdijk

Bovenstaande rapporten hebben we allemaal wel besproken of zijn kort genoemd (deze had ik van te voren al aangegeven). Verder heb ik veel van de onderwerpen aangegeven als zijnde literatuur. Eric Regeling verwijst verder nog naar een onderzoek van ene Kruiningen uit 1981 of iets in die richting, wat ook over overslag gaat. Een vergeten rapport noemt hij het. Daarnaast een rapport van een interdisciplinaire groep aan de TU over de sectie rond het monument. Dit rapport had ik al doorgenomen en daar stond mijn inziens weinig relevants in voor mijn onderzoek of dat van W+B. Verder heeft hij geen aanvullende literatuur tips.

A-2 Jentsje van der Meer 17-06-2013

Time:	10:10
Date:	Monday 17-06-2013
Location:	Ljouwerterdyk 55 A, 8491 ML, Akkrum
Company:	Van der Meer Consulting B.V.

Uiteraard aan elkaar even voorgesteld, ik heb verteld wat ik doe en Jentsje van der Meer vertelde dat hij dat wel de hoofdlijnen van mijn afstuderen goed op kon maken uit mijn gestuurde probleem definitie. Hij vertelde hoe hij te werk gaat, dat hij zzp'er is, een stap die hij een aantal jaren geleden gemaakt heeft. Al snel zijn we overgegaan op het inhoudelijke deel.
Om het overzichtelijk te houden even kort per punt de belangrijkste aspecten die tijdens het gesprek aan bod zijn gekomen.

De overslagbestendigheid, historie tot aan nu

Jentsje van der Meer heeft verteld over zijn eigen ervaring met overslagproeven en waar dat allemaal uitgevoerd is. Over Infram en veel projecten waar hij bij betrokken is geweest. Daarnaast de totstandkoming van de overtopping manual wat vooral een handleiding moet zijn om te ontwerpen en niet zozeer een kritische blik op het concept overtopping.

Visie op het concept overslagbestendige dijk

De overtopping simulator in Colorado kwam al snel aan bod. Jentsje van der Meer gaf aan dat wat ik en Coen Kuiper vermoedde eigenlijk wel juist was. De condities zijn niet vergelijkbaar met de Nederlandse. Het kilmaat is heel anders, de klei die als onderliggende laag gebruikt wordt, is zo goed verdicht en daarmee veel sterker dan in een echte situatie. De metingen daar zijn dan ook zeker niet representatief voor de Nederlandse situatie. Jentsje van der Meer zei het niet met zoveel woorden maar het kwam er op neer dat ik die metingen wel links kon laten liggen. Wat wel interessant is, op dit moment zijn er een aantal fabrikanten van verhardingen aan het testen of hebben getest bij de overtopping simulator in Colorado. De resultaten daarvan zijn waarschijnlijk een stuk bruikbaarder.

De cumulatieve overload factor is een hele interessante parameter met betrekking tot het concept overslag bestendigheid. Overtopping wordt dan inderdaad gezien als vermoeiing. De kritische snelheid is daar in de grootte onbekende, 4 m/s wordt vaak gezien als veilig. Sedimentatie tussen het gras kan daarbij een grootte rol spelen in de sterkte en daarmee voor de kritische snelheid. Er zijn daarnaast grastrekproeven ontwikkeld waarmee de sterkte van gras bepaald kan worden, de eerste resultaten daarvan worden op dit moment verwerkt voor een aantal locaties. Uiteindelijk willen ze hiermee langs alle testlocaties gaan waar ooit getest is met de simulator om een relatie te kunnen leggen tussen de grassterkte en de kritische snelheid of eigenlijk meer om deze te bevestigen. De kritische snelheid kan proefsgewijs bepaald worden. Er worden 3 x 50 golven op het talud losgelaten die resulteren in een bepaalde snelheid. Dus in het totaal 150 golven. Per set van 50 golven wordt de snelheid opgevoerd. Dus men start met 50 x 4 m/s dan 50 x 5 m/s dan 50 x 6 m/s. Op het moment dat na 150 golven de bekleding blijft liggen, dan ligt de U_c (kritieke snelheid) hoger dan 4 m/s. Dan wordt het verhoogt naar intiteel 5 m/s, 6 m/s, 7 m/s als hij dan wel bezwijkt dan is de U_c 5 m/s. Het gevolg is dat nu in plaats van met een week testen, eigenlijk in 6 uur tijd de kritische snelheid kan worden vastgesteld. Dit is wel onder voorbehoud dat de theorie van de cummulatieve overload klopt. Want golven die resulteren in een snelheid lager dan je U_c blijken dus eigenlijk niks te doen (iets wat overeenkomt met het vermoeiingsprincipe). In Nijmegen en Tholen zijn hier test mee gedaan. Over de grastrek proeven is een factual report beschikbaar maar nog geen officieel rapport.

Aantoonbaarheid van het concept

Jentsje van der Meer gaf aan dat het op dit moment wel degelijk aantoonbaar te maken valt of de dijk gaat voldoen. Een van de mogelijkheden is testen met de overtopping simulator of testen in de Deltagoot. Zelfs het bouwen van testsecties van 50-100 meter lang op de Afstluitdijk en deze vervolgens testen met de overtopping simulator noemt hij als haalbare optie. Echter de theoretische basis erachter is de grootte onbekende. Daar zit de grootte uitdaging in dezen en dus niet zo zeer in het daadwerkelijk aantonen, want dat zou praktisch gewoon kunnen. Dit onderscheid tussen theoretische en praktische aantoonbaarheid werkt in mijn ogen wel verhelderend, het is een onderscheid dat gemaakt dient te worden.

Kritieke punten met betrekking tot overslagbestendigheid

Het gaat niet zozeer om de stroomconcentratie, want dat kan de omgeving rond objecten vaak wel hebben, maar het gaat vooral om het effect van het object op de omgeving. Waar gaat de stroming naartoe en wat voor turbulentie wordt erdoor veroorzaakt en hoe.

Per toeval bleek het zo te zijn, tijdens eigenlijk run-up proeven, waarbij ze een simulator op de top van de dijk hadden gezet en dus run-down eerst gingen testen. Dat een asfaltstrook door de druk van het naar beneden stromende water (met een waterfront, waar je normaal tijdens run down geen last van hebt) ineens een paar centimeter omhoog kwam. Bij de A7 of het fietspad kan dit dus ook problemen geven. Al wordt hier vaak niet op gerekend (Verklarende tekenen in mijn aantekeningen).

De belastingen variëren heel erg op het talud, uit onderzoekt blijkt dat de stroomsnelheid erg kan oplopen vanaf de kruin tot aan lager op het talud. Dit zou ook deels kunnen verklaren waarom lager op het talud vaak initiële schade ontstaat. Dit blijkt uit recente metingen, waarvan de resultaten op het moment worden verwerkt.

Water op water heeft een heel groot dempend effect op de belasting. Dit heeft Jentsje van der Meer bij veel proeven gezien, er is nooit bewust op gemeten of iets dergelijks, maar op het moment dat er een erosie kuil gevuld was met water werd de erosie minder, de verwachting is dat dit ook gebeurd bij water dat eventueel op de berm van de dijk blijft staan. Dat dit dus een dempende werking heeft op de belasting op de berm. Op het talud speelt deze dempende werking veel minder gezien de snelle afstroming.

Ook Jentsje van der Meer geeft aan dat de overgangen gewoonweg het meest kritiek zijn.

De door mij geformuleerde probleem- en doelstelling en de daarop volgende oplossingsmethode

Met betrekking tot de onderzoeksvragen gaf Jentsje van der Meer hetzelfde aan als Eric Regeling, namelijk dat ik infiltratie moet afbakenen (wat ook een van mijn eigen doelen was). Hij gaf als suggestie dat ik ervan zou kunnen gaan dat er maatregelen getroffen worden die infiltratie of de gevolgen daarvan beperken en dat ik puur kijk naar de erosie. Dit lijkt me een mooi uitgangspunt. Daarnaast lijkt de beste methode voor mijn afstuderen te zijn het zo goed mogelijk vaststellen en in kaart brengen van de verschillende belastingen die er spelen. Daarnaast het raadplegen van literatuur met betrekking tot spillways, daar zijn meer parallen te trekken dan ik in eerste instantie dacht, met name in het buitenland. Het wordt dus met name 'relatief' makkelijk aan de belastingkant en lastiger aan de sterkte kant, dat laatste vooral zoeken in de verschillende te trekken parallellen. Vervolgens die twee aan elkaar koppelen, dat zou tot een mooi resultaat kunnen leiden. Er zijn dus inderdaad wel methodes waarmee je het kan aantonen, echter nog niet theoretisch daar moet de focus dan ook op liggen. De grootte aanpassing in mijn probleem- en doelstelling ligt dan ook daar. Eventueel een comflow simulatie kan ter validatie van de theorie, echter je kan ook een heel afstuderen richten alleen op comflow volgens Jentsje van der Meer is bovenstaande daarom interessanter. Er zijn ook vergelijkbare programma's in het buitenland.

Waarop ontbreekt het nog aan kennis op dit moment met betrekking tot overslagbestendigheid en niet-waterkerende objecten en discontinuïteiten

Met betrekking tot de kennisleemte die Eric Regeling aangaf met een $H_s > 3m$, vertelde Jentsje van der Meer dat recent onderzoek, waar de meeste mensen nog niet vanaf weten, al wel een verloop laat zien van golven met een H_s groter dan 3 m in relatie tot. Hij voegde daar nog aan toe dat het verloop met name afhankelijk is van de relative freeboard. Dit staat uitgebreid beschreven in een paper die nog moet verschijnen en een update is van de overtop manual. (De werktitel was: Wave overtopping: an update on the overtopping manual and new physical insights). Wat grappig was dat de relatie die batjes ooit heeft afgeleid op basis van theorie een zeer goede fit was voor door de nieuwe data (aan de hand van grafieken toegelicht). Hij zag dat dan ook niet meer als kennisleemte (waarschijnlijk belangrijk voor project W+B). Vooral de kritische snelheid is nog een grootte onbekende, dat is een mooie maat voor de sterkte. De spreiding van de overslaande tong is inderdaad ook een onbekende. De frontsnelheid is zeer interessant en de laagdikte is veel minder van belang. Hij gaat er eigenlijk vanuit dat het geen gras oplossing gaat worden op de Afsltuidijk, al denkt hij dat het wel een hele hoop kan hebben. Zeker als de kritische punten aangepakt worden.

Aan te bevelen rapporten en literatuur

- Factual report + SBW Rapport Deltares infiltratie en overslag proeven afsluitdijk
- Vietnam wave overtopping simulator tests
- Destructive wave overtopping tests on grass covered landward slopes of dikes and transitions to berms
- Overtopping manual
- Guidance on erosion resistance of inner slopes of dikes from three years of testing with the wave overtopping simulator
- Flow depths and velocities at crest and inner slope of a dike
- Destructive wave overtopping and wave run-up tests on grass covered slopes of real dikes
- Overtopping simulator Colorado 2011 and 2012

Al bovenstaande stukken zijn wel aanbod gekomen tijdens ons gesprek, daarnaast nog een hoop suggesties van Jentsje van der Meer. Paper van Hughes kwam ter sprake waarbij de relatie tussen de relative freeboard en de coëfficiënt c in de overtopping volumes anders wordt met toenemende overslag debieten. Eigenlijk lijkt het nu zo te zijn dat bepaalde gesimuleerde debieten een zwaarde last zijn dan tot nu toe werd gedacht. Zo is er bijvoorbeeld op de afsluitdijk uitgegegaan van 75 $l/s/m$, dit zou echter met de nieuwe coëfficiënt, welke dus gedeeltelijk gebaseerd is op de cummulative overload factor, gelijk kunnen zijn aan 125-150 $l/s/m$. Het zou dus wel eens een hele andere visie kunnen geven op de resultaten.
Die factor c met betrekking tot de overslaande volumes is in eerste instantie gebaseerd op beperkte kennis, echter de huidige stand van zaken geeft vernieuwde inzichten welke dus wel eens kan resulteren in een verlaging van je belasting!
Verder heb ik drie mappen vol met beschrijvingen van test en daaruit volgende literatuur tot en met de voorlaatste tests gehad. Hier staat een hoop bruikbare informatie in die ik allemaal nog door kan spitten. Verder zijn er een aantal papers in preparatie, deze stuurt Jentsje van der Meer mij toe in concepts, hij denkt dat ik vooral daar heel veel aan kan hebben.

Appendix B

Typical cross section Afsluitdijk

The sketch in figure B-1 on the next page depicts the 'standard' cross section of the Afsluitdijk. The sketch is based on the standard cross section as depicted in Deltares [2013].

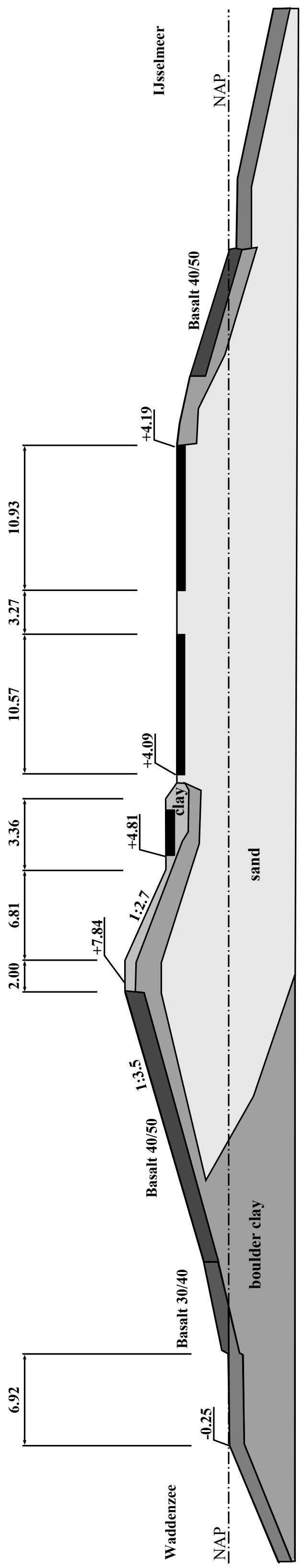

Figure B-1: Typical cross section Afsluitdijk

Appendix C

Inventory and categorization of NWO's and transitions

For the division of the Non Water Retaining Object(s) (NWO) an inventory has been done via a site visit and Google Street View [1]. In this section the different NWO's are given with, when possible, an example. The categorization that has been made can also be seen in table C-1 and is partly based on Morris [2012]. However, the categorization in the paper did not meet all the needed categories, therefore it is modified to fit the NWO's present at the Afsluitdijk. To clarify the different structures, objects and transition described in the table C-1, in figure C-1 to C-29 screen shots taken from Google Street View and pictures taken during a site visit are shown. In the caption of each figure the number refers to the 'Nr.' row in table C-1. For some of the objects or transitions there aren't separate figures, because these appear in some of the other pictures or because these are not visible at all, this is for:

3: old foundations (not visible)

4: old revetments (not visible)

33: guard rail (poles) (visible in several figures, example figure C-9)

34: reflector poles (visible in several figures, example figure C-19)

26: pathway over the crest, see figure C-4 and C-16

54: slope to horizontal (visible in several figures, example figure C-12)

23: highway A7 (visible in several figures, example figure C-8)

24: cycling path (visible in several figures, example figure C-29)

One unknown object is found during the inventory of objects on the Afsluitdijk, figure C-30. It could be a culvert, this is however not expected at the Afsluitdijk. It is located near hectometer sign 72.6 and seems to be made of concrete.

[1] maps.google.nl

C-1 Categories

Vegetation has been chosen as a different category because the trees and bushes, which are in this category, are not man made and therefore undefined in size and presence. The vegetation 'mixes' with the dike it self, an aspect that is not valid for the other categories. The mixing differs for each kind of vegetation, therefore not an abrupt transition. The mixing might have a positive effect on the strength of the top layer. The tree it self does influence the overtopping wave and the root system influences the strength of the top layer. Testing has been done for trees of various sizes and erosion did occur Trung et al. [2011c] and Steendam et al. [2012b].

Structures are man made, and often have an abrupt transition between the structure and the dike. Due to the large variety in structures a subdivision has been made in order to better describe the difference in influence each structure has on the load and strength of the landward slopes top layer. The division is based on whether or not the structures are under ground and if they penetrate the landward slopes top layer.

Buried structures, these structures are under ground and in principle do not penetrate the landward slope top layer. However, if a part of the top layer is eroded these structures can be at the surface, after which they do have an influence on the erodibility. At first this is a case of external erosion, but if the top layer has eroded enough also contact erosion may become important. Old foundations are an example of a NWO that might be in the top layer but is buried. The old foundations don't have to be present on the Afsluitdijk. Due to the age of the dike it is likely that certain objects are present, but are not shown. This can for example be due to overgrowing (looking at the outer revetment made from basalt, which is also overgrown for large parts). However, these non visible objects could be a hazard during extreme wave overtopping conditions. This also has been shown in test with the overtopping simulator. Old foundations were tested and during another test an old brick revetment showed after the grass cover was eroded.

Partly buried but visible structures do penetrate the landward slope top layer (fully or partly) and influence the flow of the overtopping wave. The erodibility around these structures is greatly influenced, because both the flow and strength are influenced. Both external and contact erosion between soil and hard structure can occur. These structures are visible and therefore easier to detect than buried structures. The flow is influenced in a more complicated way then with buried structures. Also within this category a large variety of structures can be found, the influence they have do however, al have the same characteristics.

External structures aren't buried in the subsoil at all. These structures do more or less only have contact with the landward slope top layer surface. As an example of this, Morris [2012] takes roads and surface protection. However, the definition chosen in the thesis is such that only NWO are under consideration here. Separately treated are the water retaining objects, what a protective revetment is, which can have a certain transition but is not a NWO. This is because surface protections can be designed to handle certain loads. Therefore the protective surface will not be taken as an external structure in the thesis but can be found at the transitions. The road, parking lots and cycling path are treated as external structures, because these do not have a water retaining function. The external structures do not penetrate the top layer. Flow can occur underneath, so between the protective top layer and the external structure it self. This can cause erosion because of increased pressures and flow velocities. This is a contact erosion between the soil and the hard structure. The transition at the edge of the external structures will also cause contact erosion. In addition to the above described structures a lot of loose objects are present on the Afsluitdijk and can be seen at different pictures. These vary from containers and dustbins to movable road signs. For example, the gutters, benches and tables and agricultural tanks are also vulnerable to both types erosions. However, the expectation for the last two is that these will be flushed away and cause little or no damage to the landward slopes top layer. These do not have a connection with the dike and only can cause damage as 'floating debris'. Which can result in other NWO's being damaged or holes in the landward slope of the dike. The objects it self do not have a connection to the dike and therefore have no influence on the erosion.

Cables and pipelines are a separate category. However, these will not be taken into account in the thesis, because they have no influence on the surface erosion unless they fail and cause damage to the dike. After which the top layer is already failed completely. This kind of damage is important for the failure of the dike as a whole but can be treated as a separate failure mechanism.

Poles are presented as a separate category because they are present in such a large variety themselves and differ from the other structures. For example all poles are penetrating the top layer (partly or fully) and would have not been subdivided in the different categories for structures. The poles have a relatively small influence area and are mostly present as individual objects (some with a relative constant repetition, like the poles from the guard rails and reflector poles at some locations along the dike). Within the poles category a clear separation can be made into poles with or without foundations. The poles with foundations might be seen as the same as 'structures partly buried but visible'. The poles without a foundation cannot be seen as such, due to the above mentioned aspects. The division in larger or smaller than 15 cm is because from testing with the Wave Overtopping Simulator it appears that objects smaller than this diameter don't have an additional influence on the erosion rate of the landward slope top layer. For poles with a foundation this division doesn't have any influence due to the fact that poles with a smaller diameter simply don't have a foundation.

Not only the different NWO's are shown in the pictures and table C-1, but also the *transitions* that are present at the Afsluitdijk are shown. Some transitions might be seen as NWO's and vise versa. There is an overlap between some of the transitions and structures. The transitions aren't treated as objects, because they are part of the dike it self. This is for example the oblique driveway and between different type of revetment. Some aspects of certain transitions might be used for other transitions or even for structures.

Table C-1: NWO's at Afsluitdijk

Nr.	Category NWO's
	Vegetation
1	Bushes
2	Trees
	Structures
	Buried structures
3	Old foundations
4	Old revetments
	Partly buried but visible structures
5	Pedestrian walkway near the monument
6	Pedestrian walkway near the Kornwerderzand lock complex
7	Bridge Breezanddijk
8	Guard rail (burrowing in subsoil)
9	Lock house Kornwerderzand lock complex
10	Lock house Den Oever lock complex
11	Gas station Breezanddijk
12	House den Oever lock complex
13	Bunkers (in Dutch: Kazematten)
14	Outflow openings of rain water drainage channels A7
15	Concrete beam emergency phone
16	Cabinets for electricity
17	Radio signal tower
18	Stairs on landward slope
19	Statues
20	Bridge Kornwerderzand

Table C-1: NWO's at Afsluitdijk

Nr.	Category NWO's
21	Unknown object at hm 72.6
	External structures
22	Benches and tables
23	Highway A7
24	Cycling path
25	Parking lots
26	Pathways over the crest
27	Gutters Kornwerderzand landward slope (side of Friesland)
28	Agricultural tank
	Cables and pipelines
	Not considered in my thesis
	Poles
	Poles with foundations
	<15 cm
	>15 cm
29	Portals
30	Beacon at hectometer 93.8
31	Singular portals
32	Windmill Kornwerderzand
	Poles without foundations
	<15 cm
33	Guard rail (poles)
34	Reflector poles
35	Fences
36	Fences transverse to the dike
37	Climate pole
38	Flagpoles
39	Speed signs
40	Construction zone signs
	>15 cm
41	Emergency phones
42	Landmark Friesland/Noord-Holland
43	Concrete pole, marker hectometer on dike crest
44	Barrier
45	Direction signs
46	Lamp posts
47	Solar panels
48	Warning signs
49	Gas station sign
	Transitions
50	Oblique driveways landward slope (unpaved)
51	Oblique driveways landward slope (paved)

Figure C-1: 1 and 2 Bushes and trees

Figure C-2: 5 Pedestrian bridge Monument

Figure C-3: 6 Pedestrian bridge Kornwerderzand

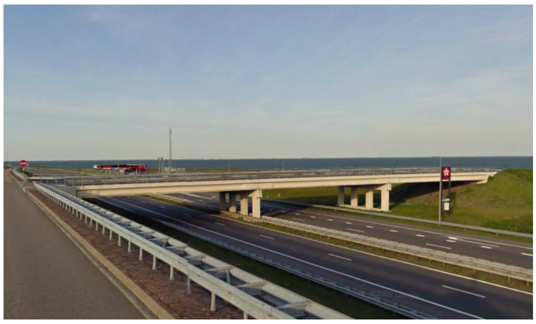

Figure C-4: 7 and 49 Bridge Breezanddijk and gas station sign

Table C-1: NWO's at Afsluitdijk

Nr.	Category NWO's
52	Plates over the crest (near Den Oever lock complex)
53	Transitions between different dike sections
54	Slope to horizontal
55	Between different revetment types

Figure C-5: 8 Example guard rail buried in subsoil

Figure C-6: 9 and 12 Lock house Kornwerderzand and house

Figure C-7: 10 Lock house Den Oever

Figure C-8: 11, 38 and 46 Gas station, flag poles and lamp post

Figure C-9: 13 Bunkers

Figure C-10: 14 Outflow opening

Figure C-11: 15 and 41 Emergency phone with concrete beam

Figure C-12: 16, 29 and 52 Electricity cabinets, portal and plates over crest

Figure C-13: 17 Radio signal tower

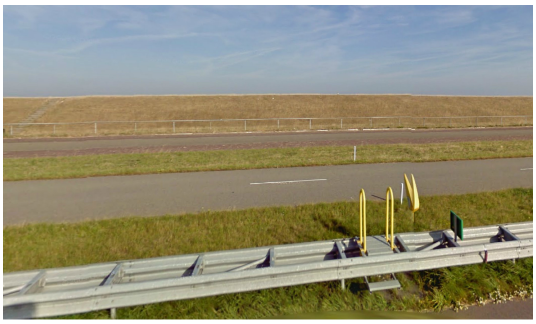

Figure C-14: 18 and 25 stairs landward slope and parking lot

Figure C-15: 19 Statues

Figure C-16: 20, 45 and 46 Bridge Kornwerderzand, direction signs

Figure C-17: 22 Benches and tables

Figure C-18: 27 Gutters Kornwerderzand

Figure C-19: 28, 35 and 36 Agricultural tank and fences

Figure C-20: 30 Beacon

Figure C-21: 31 Singular portal

Figure C-22: 32 Windmill

Figure C-23: 40 Construction zone sign

Figure C-24: 42 and 43 Land mark or hectometer marker

Figure C-25: 44 and 52 barrier and paved driveway

Figure C-26: 47 Solar pole

Figure C-27: 48 Warning signs

Figure C-28: 51 Driveway unpaved

Figure C-29: 50 Transition between different dike segments

Figure C-30: 21 Unknown object

Appendix D

Probabilistic calculation

In this appendix a start with a probabilistic calculation is presented. The goal of this was to see what the influence the different parameters on the wave overtopping resilience is.

Because it might be interesting to know which parameters have a large influence on the outcome of the overtopping resilience of the Afsluitdijk a probabilistic analyses can be a interesting method to investigate this. From the procedure as described in section 5-1 it appeared that there is not a formula possible to couple the load and the strength. Therefore the probabilistic analyses only concerns the overtopping discharge q which eventually leads to the load caused by wave overtopping. This probabilistic part can be seen as an addition to the above procedure in order get an overview of which required research is the most efficient to execute. This is only a first attempt to see whether or not this will work with the current knowledge and formulations.

This probabilistic calculation consists of nine different steps according to Kortenhaus et al. [2002]:

1. Select structure properties through preliminary deterministic design
2. Identify most relevant failure modes
3. Formulate limit state equations (LSEs)
4. Determine uncertainties of stochastic parameters & load models for load & resistance
5. Limit state equations including uncertainties
6. Perform level II/III reliability analysis P_f for each failure mode
7. Fault tree analysis (correlation between failure modes)
8. Calculation of P_f for whole system
9. Minimize $P_f - P_f^t$ or $P_f - P_{f optimal}$ by improving structure properties

It is however an iterative process and step 3 and 7 have similarities. Step 1 has been executed in section 5-1-2, the deterministic calculation (results in table 5-3), step 2 earlier in the report as well (overtopping is the main failure mode for the thesis, other mechanism are caused by overtopping). The limit state equation for the overtopping discharge, $z = R - S$ can be described by:

$$z = \frac{q}{\sqrt{g \cdot H_{m0}^3}} - \frac{0.067}{\tan \alpha} \cdot \gamma_b \cdot \xi_0 \cdot \exp\left(4.75 \frac{R_c}{\xi_0 \cdot H_{m0} \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{D-1}$$

The other equations that are used to calculate the parameter in order to determine the overtopping discharge are 2-6, 2-7 and 2-8. These are thus implicit present in the equation o the z-function, equation D-1.

The uncertainties for the parameters according to step 4 of equation D-1 can in first instance be given only describing the known uncertainties for the model parameters these are presented in table D-1 (Schiereck [2001] and Pullen et al. [2007]), the other variables are set as deterministic values. These will be used in the limit state equation. Because only wave overtopping is taken into account

Table D-1: Variables z function

Variable	Distribution	σ	μ
4.75	Normal	4.75	0.5000

in this thesis step 6 and 8 are actually the same for this calculation. Instead of taking a resistance and a solicitation for the limit state function. Now the overtopping discharge is taken as limit. As long as $z > 0$ the deterministic overtopping discharge is not exceeded, if $z < 0$ it is. With the deterministic values and the known distribution for the coefficient 4.75 the probability of occurrence of the deterministic calculated discharge becomes 0.5 per year, thus 50% per year. This means that approximately half of the probabilistic calculations have a larger wave overtopping discharge then the deterministic calculated value of 159 $l/s/m$. This can be seen in figure D-1, were the number of times the z function is negative is somewhat larger than for it to be positive. Negative values means that the used value of 159 $l/s/m$ is being exceeded, positive means that the value is below. The question is whether or not this 50 % probability per year is allowable. The Afsluitdijk has to be able to withstand conditions that belong to 1/10 000 per year. So the conditional probability, the probability of occurrence of this wave overtopping discharge knowing that the 1/10 000 year conditions occur, of failure should be determined.

Currently in the Netherlands this is 10% of the norm for failure mechanisms other than wave-overtopping or run-up. For these failure mechanisms no specific criteria were given, only that the failure probability of the other loads has to be negligible. However, the probability of these mechanisms had to be negligibly small in relation to the exceedanc frequency per year for the run-up and overtopping according to Schiereck [1998b]. This also meant that it can be stated that the conditional probability of failure is maximum 0.01 to 0.1, which is 1 to 10%. For the other failure mechanisms it was later interpreted that these contribute no more then 10 % to the flood risk Weijers and Tonneijck [2009]. In dune design the failure probability of 10 times the norm is actually used. Therefore a probability of occurrence between 0.01 and 0.1 for the wave overtopping discharge will be used.

For the probabilistic calculation the value for the wave overtopping discharge has been varied. Each value of the wave overtopping discharge results in a different percentage of occurence. This deterministic value resulted in a probability of occurance of 50 %. The value that is strived for is a probability of occurance of between 1 and 10 %. From the variation of the wave overtopping discharge it appeared that the 1 % value for overtopping becomes 340 $l/s/m$ as can be seen in figure D-3. The value for the wave overtopping discharge that gives a probability of occurance of 10 % was 240 $l/s/m$ as can be seen in figure D-2. The figures depict the number of times the probabilistic calculation results in a certain wave overtopping discharge. So when the value of the overtopping discharge is on the left part of the graph it has a large probability of occurence, if it is on the right part it has a small probability of occurence. Knowing the values that result in 1 % and 10 % probability of occurence the choice should be made what value should be used. The 1 % value is the on the safe side because this percentage should be for the failure and not the probability of occurence. Due to the fact that the resistance has not jet been taken into account, the right better choice is the 10% value of 240 $l/s/m$.

The above has only been done by assigning distributions to parameters which have known distributions. Most of the other parameters are given boundary conditions like the wave sea state of the Waddenzee and the lay-out of the Afsluitdijk The procedure can be done by determining/estimating a distribution for every variable used in equation D-1 and then calculating the probability of occurrence. However, the landward slope strength has not been taken into account. These should be taken into account with a calculation like this. However, due to the lack of knowledge this is not possible until a complete theoretical method is available which does link the strength and load. Therefore the choice has been made to use the values from the 'deterministic calculation' as described in section 5-1-2 for further calculations.

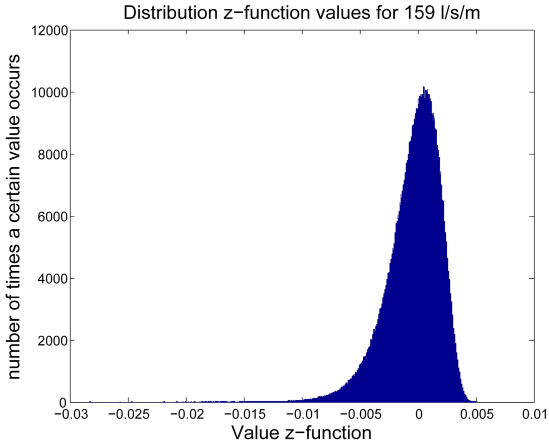

Figure D-1: Distribution z function for 159 $l/s/m$

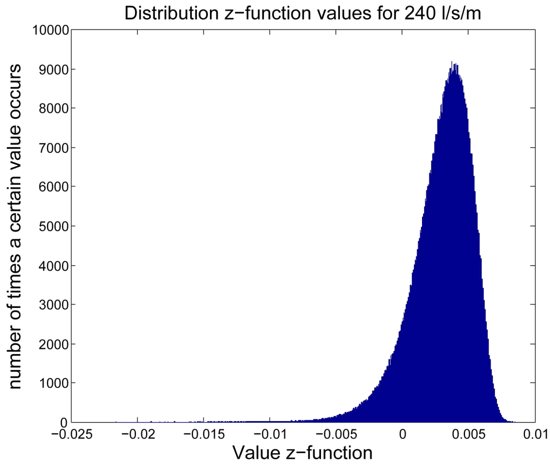

Figure D-2: Distribution z function for 240 $l/s/m$

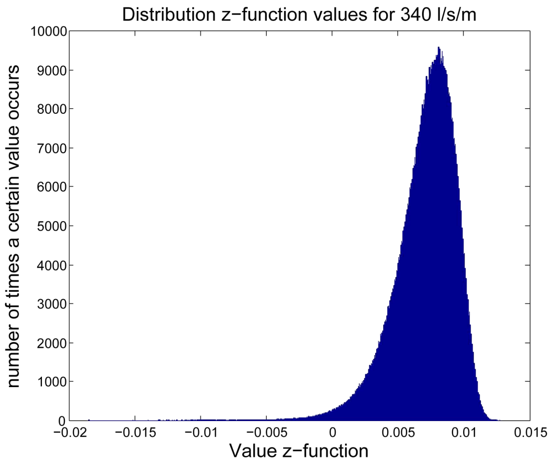

Figure D-3: Distribution z function for 340 $l/s/m$

Instead of only taking into account the variation of the parameters at one cross section also the spatial variation along the dikes course can be taken into account. Wave height and period, crest height, slope steepness, berm width and height are all examples of parameters that vary along the course of the dike. The spatial variation could be implemented by taking these variables and fit a distribution to the extremes along the course of the dike which gives a probability of occurrence along the dike as a whole, or determine for each dike section the probability distribution from the uncertainties of each parameter and with all these distributions calculate the probability distribution of the Afsluitdijk as a whole.

This can be done by seeing the different dike sections as a serial system, this means that if one of the dike sections fails the whole dike fails. This means that the probability of failure is not equal to the probability that the governing dike section fails (this can be equal to this value but that is the most favorable probability, or the lower boundary). The limits of failure are given by equation D-2 according to Schiereck [2001].

$$\max P_{Fi} \leq P_{F_{tot}} \leq \sum P_{Fi} \tag{D-2}$$

It is most likely that this section fails first but another dike section can fail at an earlier stage. Again this is not likely but it is possible. Therefore in order to get to the maximum probability of failure of the dike as a whole the probability of failure of all the different dike sections should be added (this is the upper boundary, thus the maximum probability of failure). This is given by equation D-3.

$$P_F = 1 - ((1 - P_{F1}) \cdot (1 - P_{F2}) \cdots (1 - P_{Fn})) \tag{D-3}$$

In which n is the number of dike sections. So the larger a dike is, the larger the probability that a certain dike section fails. However this is for a independent serial system. However, for the failure mechanism of wave overtopping dependency is present. This means that the formulation of equation D-3 would result in a probability of failure that is to low. This would require more additional research before this calculation can be done. However, at the Afsluitdijk it is very hard to ignore the so called length effect due to the length of 32 km. According to Rijkswaterstaat [2011] a large effect of the length is present if

- a large spread in strength or load probabilities
- fast spatial fluctuations
- lack of knowledge on the real values of the strength and/or the load on different locations and time.

Which can all be checked at the Afsluitdijk. And this is only for the failure mechanism of wave overtopping. However, the knowledge on both load and especially strength is not detailed enough to make a reliable calculation for the total probability of failure of the dike as a whole. Therefore, this can only be used to try and see what the sensitivities are as far as the load is concerned.

Currently because only the known distribution is taken into account as a distribution with an uncertainty and for the other boundary condition parameters a deterministic value have been taken only one parameter has an uncertainty value α and this is the value of 4.75 in equation D-1. This value thus becomes 1. By only calculating the overtopping discharge with a probabilistic analysis this method is not suitable to asses the influence of parameters that are important to asses the overtopping resilience of the Afsluitdijk.

Appendix E

Matlab script from input to load

```
%% Load 2
clear; close all; clc;
%script for determining load
%% Definitions
%General
g=9.81;                                  %Gravitational acceleration              (m/s^2)
%Geometry of dike
HW=5.28;                                 %Highest high water level                (m)
Hcrest=7.84;                             %Crest Heigh                             (m)
gamma_r=1;                               %Roughness factor outer slope            (-)
Bb=6.92;                                 %Reduction factor berm                   (-)
gamma_f=1;                               %Reduction factor friction               (-)
gamma_v=1;                               %Reduction factor vertical wall          (-)
angleslope=3.5;                          %1 in angleslope                         (-)
Hberm=-0.25;                             %Level of the berm                       (m)
%Boundary conditions
Hm0=3.92;                                %Zeroth order significant wave height    (m)
Tm10=6.3;                                %Mean wave period spectral               (s)
Nw=6*60*60/Tm10;                         %Number of waves during 6 hour super storm (-)
beta=18;                                 %Angle of incomming waves                (-)
%Fitting parameters
A=1.65;                                  %Curve fitting parametets Run-up         (-)
B=4;                                     %Curve fitting parametets Run-up         (-)
C=1.5;                                   %Curve fitting parametets Run-up         (-)
%% Elapsed time starts to count
tic;                                     %Start counting time                     (-)
%% Parameter determination
Tm=0.9*Tm10;                             %Mean wave period approximation          (s)
gamma_beta=1-0.0033*beta;                %Recution factor wave angel              (-)
Lb=angleslope*Hm0*2+Bb;                  %Bermlength                              (m)
hb=HW-Hberm;                             %Water on berm                           (m)
x=2*Hm0;                                 %Berm depth                              (m)
gamma_b=1-Bb/Lb*(0.5+0.5*cos(pi*(hb/x))); %Reduction factor berm                  (-)
Rc=Hcrest-HW;                            %Freeboard                               (m)
alpha=atan(1/angleslope);                %Angle of slope                          (-)
L0=g*Tm10^2/(2*pi);                      %Deep water wave length                  (m)
xi0=tan(alpha)/sqrt(Hm0/L0);             %Surf similarity parameter               (-)
%% Determing the Volume according to a certain probability
b=exp(-0.2*Rc/Hm0)+(0.56+0.15*cot(alpha));
    %Shape factor Weibull distribution overtopping      (-)
Ru2=min(A*gamma_b*gamma_r*gamma_beta*xi0*Hm0,Hm0*gamma_r*gamma_beta*(B-C/sqrt(xi0)));
    %Run-up 2% level                                    (-)
Pov=exp(-(sqrt(-log(0.02))*(Rc/Ru2))^2);
    %Probability of overtopping                         (-)
Now=Nw*Pov;
    %Probability of overtopping                         (-)
Q=(0.067/sqrt(tan(alpha)))*gamma_b*xi0*exp(-4.75*Rc/(xi0*Hm0*gamma_f*gamma_b*gamma_beta*gamma_v));
    %Dimensionless overtopping                          (-)
```

```
q=Q*(sqrt(g*Hm0^3));      %Average overtopping discharge      (m^3/s/m)
qmax=sqrt(g*Hm0^3)*0.2*exp(-2.6*(Rc/(Hm0*gamma_f*gamma_beta)));      %Maximum average overtopping discharege      (m^3/s/m)
a=1.13*tanh(1.31*b)*(q*Tm10/(Pov));      %The shape parameter ofdistribution of wave volume  (-)
precision=1;      %The discretization of the vectors      (-)
%% Determining the probability of overtopping of a certain wave volume
PvV=(0:(1/(Now*precision)):1);      %Probability distribution overtopping wave volume      (-)
V=a*nthroot((-log(-PvV+1)),b);      %Overtopping wave volume      (m^3/m)
pvV=(b/a)*(V/a).^(b-1).*exp(-(V/a).^b);      %Probability density function overtopping wave volume   (-)
%% Determining flow velocity u and flow depth h
u=4.5*(V).^0.3;      %Flow velocity      (m/s)
h=0.1*(V).^0.75;      %Flow depth      (m)
au=(4.5*(a)^0.3);      %Scale parameter flow velocity distribution      (m^3/m)
bu=b/0.3;      %Shape parameter flow velocity distribution      (-)
PvU=1-exp(-(u./au).^(bu));      %Probability distribution overtopping flow velocity      (-)
pvU=(bu/au)*(u/au).^(bu-1).*exp(-(u/au).^bu);      %Probability density fuction overtopping flow velocity   (-)
%% Determination parameters number of waves to overtopping wave volumes
Now=round(Now);      %Number of overtopping waves      (-)
Nowr=Now*PvV;      %Number of overtopping waves according to the distr.     (-)
Nowru=Now*PvU;      %Number of overtopping waves according to the distr. U  (-)
%% old method for the overtopping volumes per discharge
aold=0.84*q*Tm10/(Now/Nw);      %Scale parameter old      (m^3/m)
bold=0.75;      %Shape paramter old      (-)
PvVold=1-exp(-(V/aold).^bold);      %Probability distribution overtopping volumes      (-)
Nowrold=Now*PvVold;      %Number of overtopping wave accoding to the distr.      (-)
% Integrals
qnew=trapz(Nowr, V*1000)/(3600*6);      %Overtopping discharge new method      (m^3/s/m)
qold=trapz(Nowrold, V*1000)/(3600*6);      %Overtopping discharge old method      (m^3/s/m)
%U63=trapz(u(1,1:32001),pvU(1,1:32001));
%======================================================================
%Largest 10%waves
%======================================================================
use=find(PvU>0.9);      %Determining 10% largest wave velocities      (-)
ugov=u(1,use(1,1):end);      %Highest 10% of the overtopping wave velocities      (m/s)
Nowgov=Nowru(1,use(1,1):end);      %Number of waves accoreding to 10% overtopping waves     (-)
%% Umax along the slope (test calculation)
%======================================================================
%parameter definition
%======================================================================
f=0.0058;      %friction factor schuttrumpf
u0=u(1,use(1,1));      %starting velocity      (m/s)
h0=0.4;      %starting flow depth      (m)
betai=atan(1/2.7);      %angel of landward slope      (-)
ltalud=6.81;      %the horizontal length of the landward slope      (m)
%======================================================================
%friction factor f=0.0058
%======================================================================
f=0.0058;      %friction factor schuttrumpf      (-)
s(1)=0;
for j=1:(ltalud+0.25)/0.25
    s(j+1)=0+0.25*(j-1);
    umax(1)=u0;
```

```
    for i=1:1000
        umax(i+1)=(u0+((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(u0*h0./umax(i))/f)*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2))/(1+(f*u0/((u0*h0./umax(i))*(sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))))*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2));
        p(i)=umax(i+1)-umax(i);
    if abs(p(i))==0.0001 , break, end
    end
hmin(j+1)=u0*h0/(max(umax));
umax1(j+1)=max(umax);
end
%==========================================================================
%friction factor f=0.01
%==========================================================================
f=0.01;
s(1)=0;
for j=1:(ltalud+0.25)/0.25
    s(j+1)=0+0.25*(j-1);
    umax(1)=u0;
    for i=1:1000
        umax(i+1)=(u0+((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(u0*h0./umax(i))/f)*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2))/(1+(f*u0/((u0*h0./umax(i))*(sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))))*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2));
        p(i)=umax(i+1)-umax(i);
    if abs(p(i))==0.0001 , break, end
    end
hmin1(j+1)=u0*h0/(max(umax));
umax2(j+1)=max(umax);
end
%==========================================================================
%friction factor f=0.015
%==========================================================================
f=0.015;
s(1)=0;
for j=1:(ltalud+0.25)/0.25
    s(j+1)=0+0.25*(j-1);
    umax(1)=u0;
    for i=1:1000
        umax(i+1)=(u0+((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(u0*h0./umax(i))/f)*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2))/(1+(f*u0/((u0*h0./umax(i))*(sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))))*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2));
        p(i)=umax(i+1)-umax(i);
    if abs(p(i))==0.0001 , break, end
    end
hmin(j+1)=u0*h0/(max(umax));
umax3(j+1)=max(umax);
end
%==========================================================================
%friction factor f=0.02
%==========================================================================
f=0.02;
s(1)=0;
for j=1:(ltalud+0.25)/0.25
    s(j+1)=0+0.25*(j-1);
    umax(1)=u0;
    for i=1:1000
        umax(i+1)=(u0+((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(u0*h0./umax(i))/f)*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2))/(1+(f*u0/((u0*h0./umax(i))*(sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))))*tanh((sqrt(2*f*g*sin(betai)/(u0*h0./umax(i))))*(-(u0/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*s(j+1)/(g*sin(betai)))))/2));
        p(i)=umax(i+1)-umax(i);
    if abs(p(i))==0.0001 , break, end
    end
hmin(j+1)=u0*h0/(max(umax));
umax4(j+1)=max(umax);
end
%% Determine the resulting needed critical velocity along the crest and along the slope
%==========================================================================
%Uc at the crest
%==========================================================================
Ucrest(1)=1;
for i=1:1:100
```

```
Ucrest(i+1)=Ucrest(i)+0.1;
use1=find(u>Ucrest(i+1));
ucum=u(1,use1(1,1):end);
Nowcum=Nowru(1,use1(1,1):end);
D(i)=sum(1/precision*(ucum.^2-Ucrest(i+1)^2));
if D(i)<=6000, break, end
end
Uccrest=max(Ucrest)
Percentageofwaves=(length(ucum)/length(u))*100*Pov

Ucrest35(1)=1;
for i=1:1:100
Ucrest35(i+1)=Ucrest35(i)+0.1;
use1=find(u>Ucrest35(i+1));
ucum35=u(1,use1(1,1):end);
Nowcum35c=Nowru(1,use1(1,1):end);
D(i)=sum(1/precision*(ucum35.^2-Ucrest35(i+1)^2));
if D(i)<=3500, break, end
end
Uccrest35=max(Ucrest35)
Percentageofwaves35crest=(length(ucum35)/length(u))*100*Pov
%=======================================================
%recalculating the values to slope velocities
%=======================================================
f=0.01;
for j=1:length(u)
    elm=u(j);
elm2=h(j);
umax(1)=elm;
h0(1)=elm2;
for i=1:1000
umax(i+1)=(elm+((sqrt(2*f*g*sin(betai)/(elm*h0./umax(i))))*(elm*h0./umax(i))/f)*tanh((sqrt(2*f*g*sin(betai)/(elm*h0./umax(i))))*(-(elm/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*ltalud/(g*sin(betai)))))/2))/(1+(f*elm/((elm*h0./umax(i))*(sqrt(2*f*g*sin(betai)/(elm*h0./umax(i))))))*tanh((sqrt(2*f*g*sin(betai)/(elm*h0./umax(i))))*(-(elm/(g*sin(betai)))+sqrt((umax(i)^2/(g^2*sin(betai)^2))+(2.*ltalud/(g*sin(betai)))))/2));
p(i)=umax(i+1)-umax(i);
if abs(p(i))<=0.001 , break, end
end
hmin(j)=elm*h0/(max(umax));
umaxslope(j)=max(umax);
end
%=======================================================
%Uc at slope
%=======================================================
Ucslope(1)=1;
for i=1:1:1000
Ucslope(i+1)=Ucslope(i)+0.1;
use2=find(umaxslope>Ucslope(i+1));
ucumslope=umaxslope(1,use2(1,1):end);
Nowcum2=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumslope.^2-Ucslope(i+1)^2));
if D(i)<=6000, break, end
end
Ucslope=max(Ucslope)
Percentageofwaves60slope=(length(ucumslope)/length(u))*100*Pov

Ucslope35(1)=1;
for i=1:1:1000
Ucslope35(i+1)=Ucslope35(i)+0.1;
use2=find(umaxslope>Ucslope35(i+1));
ucumslope35=umaxslope(1,use2(1,1):end);
Nowcum35=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumslope35.^2-Ucslope35(i+1)^2));
if D(i)<=3500, break, end
end
Ucslope35=max(Ucslope35)
Percentageofwaves35slope=(length(ucumslope35)/length(u))*100*Pov
%=======================================================
%recalculating the amplified slope transition values
%=======================================================
alphaM=sqrt(2*sqrt(sin(atan(1/2.7))));
for i=1:length(u)
    Uamptranssh(i)= umaxslope(i)*alphaM;
end
%=======================================================
```

```
%Uc slope to hor.
%==============================================================
Uctranssh(1)=1;
for i=1:1:1000
Uctranssh(i+1)=Uctranssh(i)+0.1;
use2=find(Uamptranssh>Uctranssh(i+1));
ucumtranssh=Uamptranssh(1,use2(1,1):end);
Nowcum3=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumtranssh.^2-Uctranssh(i+1)^2));
if D(i)<=3500, break, end
end
Uctranssh=max(Uctranssh)
Percentageofwavestranssh=(length(ucumtranssh)/length(u))*100*Pov
%==============================================================
%recalculating the amplified trees velocity values
%==============================================================
Kshape=1;
alphaMt=1.2*Kshape;
for i=1:length(u)
    Uamptree(i)= u(i)*alphaMt;
end
%==============================================================
%Uc tree
%==============================================================
Uctree(1)=1;
for i=1:1:1000
Uctree(i+1)=Uctree(i)+0.1;
use2=find(Uamptree>Uctree(i+1));
ucumtree=Uamptree(1,use2(1,1):end);
Nowcumtr=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumtree.^2-Uctree(i+1)^2));
if D(i)<=3500, break, end
end
Uctree=max(Uctree)
Percentageofwavestree=(length(ucumtree)/length(u))*100*Pov
%==============================================================
%recalculating the amplified poles without velocity values
%==============================================================
Kshape=1;
alphaMpz=1.2*Kshape;
for i=1:length(umaxslope)
    Uamppolez(i)= umaxslope(i)*alphaMpz;
end
%==============================================================
%Uc poles without foundation
%==============================================================
Ucpolez(1)=1;
for i=1:1:1000
Ucpolez(i+1)=Ucpolez(i)+0.1;
use2=find(Uamppolez>Ucpolez(i+1));
ucumpolez=Uamppolez(1,use2(1,1):end);
Nowcumpolez=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumpolez.^2-Ucpolez(i+1)^2));
if D(i)<=3500, break, end
end
Ucpolez=max(Ucpolez)
Percentageofwavespolez=(length(ucumpolez)/length(u))*100*Pov
%==============================================================
%recalculating the amplified rectangeular structure velocity values
%==============================================================
Kshape=1.2;
alphaMsr=1.2*Kshape;
for i=1:length(umaxslope)
    Uampsr(i)= umaxslope(i)*alphaMsr;
end
%==============================================================
%Uc rectangular structures
%==============================================================
Ucsr(1)=1;
for i=1:1:1000
Ucsr(i+1)=Ucsr(i)+0.1;
use2=find(Uampsr>Ucsr(i+1));
ucumsr=Uampsr(1,use2(1,1):end);
Nowcumsr=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumsr.^2-Ucsr(i+1)^2));
```

```
if D(i)<=3500, break, end
end
Ucsr=max(Ucsr)
Percentageofwavessr=(length(ucumsr)/length(u))*100*Pov
%===============================================================
%recalculating the amplified rectangeular structure velocity values
%===============================================================
Kshape=1.1;
alphaMsr3=1.2*Kshape;
for i=1:length(umaxslope)
    Uampsr3(i)= umaxslope(i)*alphaMsr3;
end
%===============================================================
%Uc rectangular structures
%===============================================================
Ucsr3(1)=1;
for i=1:1:1000
Ucsr3(i+1)=Ucsr3(i)+0.1;
use2=find(Uampsr3>Ucsr3(i+1));
ucumsr3=Uampsr3(1,use2(1,1):end);
Nowcumsr3=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumsr3.^2-Ucsr3(i+1)^2));
if D(i)<=3500, break, end
end
Ucsr3=max(Ucsr3)
Percentageofwavessr3=(length(ucumsr3)/length(u))*100*Pov
%===============================================================
%recalculating the amplified rectangeular structure velocity values
%===============================================================
Kshape=1;
alphaMsr5=1.2*Kshape;
for i=1:length(umaxslope)
    Uampsr5(i)= umaxslope(i)*alphaMsr5;
end
%===============================================================
%Uc rectangular structures
%===============================================================
Ucsr5(1)=1;
for i=1:1:1000
Ucsr5(i+1)=Ucsr5(i)+0.1;
use2=find(Uampsr5>Ucsr5(i+1));
ucumsr5=Uampsr5(1,use2(1,1):end);
Nowcumsr5=Nowru(1,use2(1,1):end);
D(i)=sum(1/precision*(ucumsr5.^2-Ucsr5(i+1)^2));
if D(i)<=3500, break, end
end
Ucsr5=max(Ucsr5)
Percentageofwavessr5=(length(ucumsr5)/length(u))*100*Pov
%% elapsed time
elapsedtime=toc;

%% Plotting figures
%Probability of overtopping wave volumes
loc = { 'North' , 'South' , 'East' , 'West' ,...
'NorthEast' , 'NorthWest' , 'SouthEast' , 'SouthWest' ,...
'NorthOutside' , 'SouthOutside' , 'EastOutside' , 'WestOutside' ,...
'NorthEastOutside' , 'NorthWestOutside' , 'SouthEastOutside' ,...
'SouthWestOutside' , 'Best' , 'BestOutside' };
figure (1)
plot(V,PvV,'b')
hold on
plot(V,pvV,'--r')
xlabel('Overtopping volume V (m^3/m)', 'Fontsize',18); ylabel('Probability (-)', 'Fontsize',18)
legend('Cumulative probability', 'Probability density', 'location', loc{3})
set(gca,'Fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\
    plots\probvolumes')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\
    Report\Figures\plots\probvolumes.eps')

Tstorm=1:0.1:6;
stormduration=[1;2;3;4;5;6];
crestsmall=      [4.2;    5;      5.3 ;    5.5 ;    5.7 ;    5.8];
slopesmall=      [6.9;    7.5;    7.8;     8;       8.2 ;    8.3];
transslopeberm=  [7.6;    8.3;    8.6;     8.8;     9;    9.1];
treescrest=      [5.6;    6.4;    6.7;     6.9;     7;       7.1];
```

```
poleswithout=   [8.7;   9.4;    9.7;    9.9;    10;     10.1];
rectlw3=        [9.8;   10.5;   10.8;   11;     11.2;   11.3];
rectlw1=        [10.9;  11.6;   11.9;   12.1;   12.3;   12.4];
crest6000=      [3.1;   4.4;    4.9;    5.2;    5.3;    5.5];
slope6000=      [6.1;   7;      7.4;    7.7;    7.8;    8];
figure(41)
scatter(stormduration,rectlw1,'b')
hold on
plot(Tstorm, -3.623*Tstorm.^(-0.2966)+14.53) %slope 3500 rect
scatter(stormduration,rectlw3, '.')
plot(Tstorm,-3.623*Tstorm.^(-0.2966)+13.43) %slope 3500 disc rect
scatter(stormduration,poleswithout, '*')
plot(Tstorm, -2.2191*Tstorm.^(-0.5264)+10.99) %slope 3500 disc rect
scatter(stormduration,slopesmall, '+')
plot(Tstorm, -6.097*Tstorm.^(-0.1459)+13)
scatter(stormduration,transslopeberm, 'o')
plot(Tstorm, -3.623*Tstorm.^(-0.2966)+11.23,'b')
scatter(stormduration,treescrest, 'd')
plot(Tstorm, -2.147*Tstorm.^(-0.6631)+7.748)
scatter(stormduration,crestsmall, 'x')
plot(Tstorm,-2.929*Tstorm.^(-0.435)+7.135)
scatter(stormduration,crest6000, 'xb')
plot(Tstorm,-3.218*Tstorm.^(-0.7462)+6.319,'b')
scatter(stormduration,slope6000, '+b')
plot(Tstorm,-3.579*Tstorm.^(-0.4152)+9.68)
xlabel('Storm duration (hours)', 'Fontsize', 16)
ylabel('Required critical velocity U_c (m/s)', 'Fontsize', 16)
legend('rect.lw1','','rect.lw3','','poleswithout','','slope3500','','transslopeberm','','treescrest','','crest3500','','crest6000','','slope6000','','location', loc{11})
set(gca,'fontsize', 16)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\inflstormdur')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\inflstormdur.eps')

%Comparing old and new
figure(2)
plot(PvV,V)
hold on
plot(PvVold,V, 'r--')
xlabel('Probability', 'Fontsize',18); ylabel('Overtopping volume V (m^3/m)', 'Fontsize',18);
legend('new method', 'used for overtopping simulator')
set(gca,'fontsize', 18)

%flow velocities and volume
figure (3)
plot(V,u)
xlabel('Overtopping volume V (m^3/m)', 'Fontsize',18); ylabel('Overtopping flow velocity u (m/s)', 'Fontsize',18);
set(gca,'fontsize', 18)

%Proability of overtopping flow velocities
figure (4)
plot(u, PvU,'b')
hold on
plot(u, pvU, '--r')
xlabel ('Overtopping flow velocity u (m/s)', 'Fontsize',18); ylabel('Probability (-)', 'Fontsize',18)
legend('Cumulative probablitly', 'Probability density', 'location', loc{4})
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\probvelocity')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\probvelocity.eps')

%Comparison of old and new method
figure(5)
plot(Nowr, V*1000)
hold on
plot(Nowrold, V*1000,'r--')
xlabel('Number of overtopping waves', 'Fontsize',18); ylabel('Overtopping volume per wave (l/m)', 'Fontsize',18)
legend('New formulations', 'Formulations used for overtopping simulator')
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\volumevsnow')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\volumevsnow.eps')
```

```matlab

% Development of the flow velocity along the slope
figure(6)
plot(s,umax1)
hold on
plot(s,umax2,'--b')
plot(s,umax3,'-.r')
plot(s,umax4,':r')
legend('f=0.0058 Schüttrumpf and Oumeraci [2005]','f=0.01 Steendam et al. [2012a]','f=0.015', 'f=0.02','location', loc{7})
xlabel('horizontal distance along the slope (m)', 'Fontsize',18); ylabel('Velocity along the slope (m/s)', 'Fontsize',18);
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\maxflowvel')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\maxflowvel.eps')

% Development of flow depth along teh slope f=0.01
figure(7)
plot(s,hmin1)
xlabel('horizontal distance along the slope (m)', 'Fontsize',18); ylabel('Flow depth along the slope (m)', 'Fontsize',18);
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000crestreport')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000crestreport.eps')

% Number of waves with a certain flow velocity along the crest
figure(8)
plot(u, Nowru)
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\crestvelocity1')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\crestvelocity1.eps')

figure(22)
plot(h, Nowru)
xlabel('Flow depth h (m)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\crestdepth1')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\crestdepth1.eps')

%Number of waves with a certain flow velocity along the slope
figure(9)
plot(umaxslope, Nowru)
hold on
plot(u,Nowru, '--r')
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth slope','smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\slopevelocity1')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\slopevelocity1.eps')

%highest waves result in this flow volocity at crest
figure(10)
plot(ucum, Nowcum)
xlabel('Maximum flow velocity at crest U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth crest D=6000 m^2/s^2', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000crestreport')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000crestreport.eps')

%highest waves result in this flow volocity at crest
figure(11)
plot(ucum35, Nowcum35c)
xlabel('Maximum flow velocity at crest U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth crest, D=3500 m^2/s^2 ', 'location', loc{6});
```

```
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d3500crestreport')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d3500crestreport.eps')

%highest waves result in this flow volocity at slope
figure(12)
plot(ucumslope, Nowcum2)
xlabel('Maximum flow velocity at slope U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth slope, D=6000 m^2/s^2 ', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000slopereport')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d6000slopereport.eps')

%highest waves result in this flow volocity at slope
figure(13)
plot(ucumslope35, Nowcum35)
xlabel('Maximum flow velocity at slope U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('smooth slope, D=3500 m^2/s^2 ', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d3500slopereport')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\d3500slopereport.eps')

%Number of waves with a certain flow velocity transslopeberm
figure(14)
plot(Uamptranssh, Nowru)
hold on
plot(u,Nowru, '--r')
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('slope-berm \alpha_M=1.09 ','smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtransshdistr')
print('-depsc2','D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtransshdistr.eps')

%highest waves result in this flow volocity at transslopeberm
figure(15)
plot(ucumtranssh, Nowcum3)
xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('transition slope-berm \alpha_M=1.09 ', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtransshdam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtransshdam.eps')

%Number of waves with a certain flow velocity tree
figure(16)
plot(Uamptree, Nowru)
hold on
plot(u,Nowru, '--r')
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('trees on crest \alpha_M=1.2 ','smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtreedistr')
print('-depsc2','D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtreedistr.eps')

%highest waves result in this flow volocity at tree
figure(17)
plot(ucumtree, Nowcumtr)
xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('trees on crest \alpha_M=1.2 ', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtreehdam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumtreedam.eps')

%Number of waves with a certain flow velocity poles without
figure(18)
plot(Uamppolez, Nowru)
```

```
hold on
plot(u,Nowru, '--r')
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18);legend('poles \alpha_M=1.2 ', 'smooth crest', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumpolezdistr')
print('-depsc2','D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumpolezdistr.eps')

%highest waves result in this flow velocity at poles without
figure(19)
plot(ucumpolez, Nowcumpolez)
xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('poles no foundation \alpha_M=1.2 ', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumpolezdam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumpolezdam.eps')

%Number of waves with a certain flow rectangular structures
figure(20)
plot(Uampsr, Nowru)
hold on
plot(Uampsr3, Nowru,':r')
plot(Uampsr5, Nowru,'-.')
plot(u, Nowru, '--r')
xlabel('Flow velocity u (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18);legend('l/b=1 \alpha_M=1.44 ','l/b=3 \alpha_M=1.32 ','l/b=1 \alpha_M=1.2 ', 'smooth crest', 'location', loc{13});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsrdistr')
print('-depsc2','D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsrdistr.eps')

%highest waves result in this flow velocity rectangular structures
figure(21)
plot(ucumsr, Nowcumsr)

xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('rectangular structures l/b=1 \alpha_M=1.44 ', 'location', loc{7});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsrdam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsrdam.eps')

%highest waves result in this flow velocity rectangular structureslb3
figure(24)
plot(ucumsr3, Nowcumsr3)
xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('rectangular structures l/w=3', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsr3dam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsr3dam.eps')

%highest waves result in this flow velocity rectangular structureslb5
figure(26)
plot(ucumsr5, Nowcumsr5)
xlabel('Maximum flow velocity at transition U (m/s)', 'Fontsize',18); ylabel('number of waves (-)', 'Fontsize',18); legend('rectangular structures l/w=5', 'location', loc{6});
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsr5dam')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\plots\ucumsr5dam.eps')
%=================================================================
%trend u and pcow
%=================================================================
uplot=0.01:0.01:20;
Pcowall=[Percentageofwaves35crest; Percentageofwavestree; Percentageofwaves35slope; Percentageofwavestranssh; Percentageofwavespolez; Percentageofwavessr5; Percentageofwavessr3; Percentageofwavessr;];
Ucall=[Uccrest35; Uctree; Ucslope35; Uctranssh; Ucpolez; Ucsr5; Ucsr3; Ucsr;];
figure(30)
```

```
scatter(Ucall, Pcowall)
hold on
%derived with cftool
plot(uplot, (33.47*exp(-0.1387*uplot)))
plot(uplot, (-1.341*uplot+22.06), '--r')
Ylim([0 30])
xlabel('Required critical velocity U_c (m/s)' , 'Fontsize', 18); ylabel('Percentage critical
    overtopping waves (%)', 'Fontsize', 18); legend('data points', 'exponential trend', 'linear
    trend')
set(gca,'fontsize', 18)
hgsave('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\Report\Figures\
    plots\trend')
print('-depsc2', 'D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Report final thesis\
    Report\Figures\plots\trend.eps')
%==========================================================
%save workspace
%==========================================================
 save('D:\Paul\TU Delft\MSc\Graduation\20130507LANP2\Afstuderen\Scripts\load8.mat')
```

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List of Symbols

Symbol	Definition	Unit
A	1. parameter curve fitted run-up formula	-
	2. hydraulic load	m^3/s
a	scale parameter Weibull distribution	m^3/m
B	1. parameter curve fitted run-up formula	-
	2. crest width	-
Bb	berm width	m
b	shape factor for Weibull-distribution	-
C	parameter curve fitted run-up formula	-
$C_{headcut}$	headcut coefficient depending on material properties	s^2
c	cohesion for clay	kN/m^2
c_1	parameter for slope fitting transitional wave height	-
c_2	parameter for slope fitting transitional wave height	-
d	1. depth of the foreshore	m
	2. Assumed layer thickness perpendicular to slope	-
D	damage number	m^2s^2
E_{soil}	erosion parameter	m/s
f	1. frequency	-
	2. friction factor landward slope	-
g	gravitational acceleration	m/s^2
h	flow depth of overtopping wave	m
h_b	water depth on top of berm	m
h_0	flow depth of overtopping wave on the crest	m
H	headcut height	m
H_{m0}	wave height calculated from the zero-th moment of the spectrum	m
H_{tr}	transitional wave height	m
K_{shape}	shape factor	-
k_1	factor for calculating flow velocity landward slope	s
L_0	deep water wave length	m
L_b	berm length	m
N_{cow}	number of critical overtopping waves	-
N_{ow}	number of overtopping waves	-
N_w	number of waves	-
P_{cow}	probability of critical overtopping	-
P_F	probability of failure	-
P_{ov}	probability of overtopping	-

Symbol	Definition	Unit
P_v	probability of a certain overtopping wave volume	-
Q	dimensionless wave overtopping	-
$Q*_{tot}$	dimensionless total wave overtopping discharge	-
$Q*_d$	limit value dimensionless wave overtopping discharge	-
q	average wave overtopping discharge	$m^3/s/m$
q_1	discharge over the crest	m^2/s
q_2	total overtopping discharge	m^2/s
R	value of the resistance in the limit state function	*depends*
R_c	freeboard (crest level above still water level)	m
$R_{u2\%}$	run-up level exceeded by 2% of the waves	m
r_0	depth averaged relative turbulence intensity	-
S	value of the solicitation (load) in the limit state function	*depends*
s	horizontal position along the landward slope	m
T	lifetime	y
T_m	mean wave period	s
t	1. time	s
	2. flow duration	
t_m	representative wave duration	s
U	flow velocity of overtopping wave	m/s
U_c	critical flow velocity of overtopping wave	m/s
u_0	flow velocity on the crest of overtopping wave	m/s
U_m	depth averaged flow velocity of overtopping wave	m/s
V	volume of overtopping wave	m^3/m
Z	value of the limit state function	*depends*
$Z_{m,e}$	equilibrium depth	m
α	1. slope of the revetment	-
	2. slope of foreshore	-
	3. turbulence constant	-
α_M	amplification factor velocity for obstacles	-
β	1. angle of attack	-
	2. angle of the landward slope	-
Δx	width of slice of ground parallel to the slope	m
η_a	concentration of air in the overtopping flow	-
γ	partial safety factor	-
γ_f	reduction factor permeability and roughness	-
γ_b	reduction factor berm	-
γ_β	reduction factor angle of attack	-
$\gamma_{m,c}$	material factor cohesion	-
$\gamma_{m,\phi}$	material factor angle of internal friction	-
$\gamma_{m,\rho}$	material factor volumetric mass	-
γ_r	reduction factor roughness	-
γ_v	reduction factor vertical wall	-
ϕ	angle of internal friction	-
ψ_c	critical Shields parameter	-
ρ_g	volumetric mass of soil	kg/m^3
ρ_w	volumetric mass of water	kg/m^3
τ_0	average bottom shear stress	N/m^2
τ_c	critical bottom shear stress	N/m^2
ξ	Iribarren number (surf similarity parameter)	-
y_m	maximum erosion depth	m

Glossary

List of Acronyms

ACB	Articulated Concrete Block system
CEG	Faculty of Civil Engineering and Geosciences
HPTRM	High Performance Turf Reinforcement Mats
KNMI	Koninklijk Nederlands Meteorologisch Instituut
NAP	Normaal Amsterdams Peil
NWO	Non Water Retaining Object(s)
RCC	Roller Compacted Concrete
RWS	Rijkswaterstaat
TU Delft	Delft University of Technology
SBW	Sterkte en Belastingen Waterkeren (Strength and loads on water defences)
VTV	Voorschriften Toetsen op Veiligheid
W+B	Witteveen+Bos
WTI	Wettelijk Toetsinstumentarium