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Semi-probabilistic assessment of wave impact and runup on grass revetments

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Semi-probabilistic assessment of wave impact and runup on grass revetments

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Summary
This report describes semi-probabilistic assessment rules for wave impact and run-up on grass revetments. For both mechanisms, new models are available and safety factors can be calibrated.

For run-up a calibration has been carried out. Based on this calibration it has been decided to factor the representative value of strength parameter U_c with a β-dependent safety factor, as this is the most important strength parameter. From the calibration it follows that applying a safety factor γ_β of 1, combined with boundary conditions with exceedence probability equal to the cross-sectional reliability requirement ensures a sufficiently safe but not overly conservative assessment for the default failure probability budget. If changes are applied to the failure probability budget the safety factor will also change. It is also concluded that the results are very similar to the ones found in studies for the Design Instruments.

Grass revetments have negligible resistance against the impact of large waves. Therefore it is necessary to assess with boundary conditions with exceedence probability equal to the cross-sectional reliability requirement. For areas with large correlations only assessing for the highest water level is sufficient, but for areas with none or little correlation this gives a much too optimistic result as waves higher up the slope are lower. Therefore it is necessary to assess for different combinations of water levels and waves, with the same exceedence probability. The proposed assessment rule was verified for a set of cases in the riverine areas, which showed that it is safe, but not overly conservative.

References
See Chapter 8

Version	Date	Author	Initials	Review	Initials	Approval	Initials
01	Dec. 2015	Wouter Jan Klerk		Ferdinand Diermanse		Annemargreet de Leeuw	
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Nederlandse samenvatting

Dit rapport beschrijft het semi-probabilistische toetsvoorschrift voor het faalmechanisme GEBU (Gras Erosie Buiten Talud), voor de submechanismen falen door golfoploop en golfklap. Voor beide mechanismen zijn vernieuwde modellen beschikbaar en kunnen veiligheidsfactoren worden gekalibreerd om hiermee een semi-probabilistische toets uit te voeren.

Voor oploop is besloten om een veiligheidsfactor toe te passen op de representatieve waarde van sterkteparameter U_c, de kritieke oploopsnelheid van het golffront. Uit de kalibratie volgt dat met toepassing van een β-afhankelijke veiligheidsfactor (ofwel: de veiligheidsfactor die gerelateerd is aan de betrouwbaarheidseis) van 1,0, in combinatie met randvoorwaarden met overschrijdingskans gelijk aan de doorsnede-eis, een voldoende veilige maar niet bovenmatig conservatieve toetsing kan worden uitgevoerd. Uit een selectie van cases blijkt het gekozen safety format goed aan te sluiten op de in de toetsing te verwachten taluds. Deze waarde is gebaseerd op de standaard faalkansbegroting, wanneer deze wordt aangepast kan de veiligheidsfactor ook iets veranderen.

Voor golfklap is, vanwege de geringe weerstand van een grasbekleding, een kalibratie niet zinvol, zeker niet voor gebieden met hoge golven. De correlatie tussen waterstand en golfhoogte is van groot belang voor de toetsing. Wanneer er sterke correlatie is, betekent dit doorgaans dat golven die gepaard gaan met hoge waterstanden veel hoger zijn dan waar de bekleding tegen bestand is. Dit betekent dat in de praktijk al het gras beneden het waterstandsniveau corresponderend met de doorsnede-eis wordt afgekeurd; de overschrijdingskans van de waterstand in het ontwerppunt is dan immers nagenoeg gelijk aan de doorsnede-eis. Daarom moet voor watersystemen met hoge correlatie tussen waterstand en golven worden getoetst aan randvoorwaarden met overschrijdingskans gelijk aan de doorsnede-eis. In gebieden met minder/geen correlatie neemt de golfhoogte, bij gelijkе betrouwbaarheidseis, hoger op het talud af. Dit is aan de hand van een set test cases in het rivierengebied verder uitgewerkt. Hieruit blijkt dat het noodzakelijk is om te toetsen aan verschillende randvoorwaardencombinaties met overschrijdingskans gelijk aan de doorsnede-eis. Gecombineerd met een 5% standtijdlijn levert dit een voldoende veilige en niet onnodig conservatieve toetsing voor de testcases.

Voor de toetsing als geheel betekent het bovenstaande dat de grens tussen oploop- en klapzone niet meer op het niveau van toetspeil (of veiligheidsnorm) ligt, maar op het niveau corresponderend met een waterstand met overschrijdingskans gelijk aan de doorsnede-eis. Wanneer een overgang naar een grasbekleding onder dit niveau ligt moet worden getoetst op golfklap, voor verschillende combinaties van waterstand en golven. Wanneer de overgang boven dit niveau ligt moet worden getoetst op golfoploop, ook voor verschillende combinaties van waterstand en golfparameters. Met name in gebieden met getij geldt voor een cumulatief overslagmodel immers niet noodzakelijk dat de hoogste waterstand (en bijbehorende golven) ook de hoogste cumulatieve belasting opleveren.

Voor een volgende versie van het WBI bevelen we aan om beter te kijken naar de definitie van de verdeling van sterkte-parameter U_c. Met het gebruik van een lokale in plaats van regionale proevenverzameling en het kwantificeren van de onzekerheid ten aanzien van de vertaalbaarheid van experimenten voor golfoverslag naar golfoploop, kan de toetsregel mogelijk verder aangescherpt worden.
Een ander belangrijk aspect is dat de reststerktemodellering in het golfklapmodel tamelijk conservatief is. Zeker voor belastingcombinaties met een lage waterstand is de vraag of de aanname van beperkte reststerkte in de toetsing terecht is, aangezien de kans dat

golfklapbelasting tot doorgaand falen leidt zeer klein is. Dit kan met name in het rivierengebied veel schelen in het aantal km afgekeurde grasbekledingen.

Contents

Appendices

List of symbols

Parameter	Description	Unit
a	Constant in relation between wave height and strength duration for wave impact	[m]
b	Constant in relation between wave height and strength duration for wave impact	[1/hr]
c	Constant in relation between wave height and strength duration for wave impact	[m]
c_u	factor between runup level and maximum front velocity	[-]
d_c	Thickness of the combination of the top layer and the sub layer (clay)	[m]
D_{crit}	Critical value of cumulative overload	[m^2/s^2]
F_{sand}	Sand fraction in the clay	[-]
f	maximum allowable contribution of revetment failure to the probability of flooding	[-]
N	representative number of independent reaches in a segment	[-]
P_{max}	Failure probability of the dike segment	[-]
$P_{T,seq}$	Target failure probability of the dike segment	[-]
P_T	Target failure probability of the cross section	[-]
t_{storm}	Duration of the storm [in h]	[h]
U_c	Critical wave runup front velocity along the slope	[m/s]
Z_{eval}	Level of interest on the outer slope	[m NAP]
α_M	Factor for increased load at transitions and objects	[-]
α_S	Factor for decreased strength at transitions and objects	[-]
α_i	influence coefficient for stochastic variable X_i ($\sum \alpha_i^2=1$),	[-]
$\beta_{T,seq}$	target reliability index of the dike segment	[-]
β_T	target reliability index of the cross section	[-]
λ_1, λ_2, λ_3	Factors in the failure probability budget	[-]

Please note: this is a list of the most frequently used symbols. Symbols that are only used once are explained in the report itself.

1 Introduction

The Dutch primary flood defences are periodically tested against statutory flood protection standards. These standards are currently defined in terms of design loads. Recently, policymakers have accepted a proposal to define flood protection standards in terms of maximum allowable probabilities of flooding. To facilitate such a move, a new set of instruments for assessing the safety of flood defences is currently being developed: the WBI2017.

The WBI2017 will include probabilistic as well as semi-probabilistic assessment procedures. The latter rest on a partial safety factor approach and allow engineers to evaluate the reliability of flood defences without having to resort to probability calculus. To ensure consistency between probabilistic and semi-probabilistic assessments, the presently used partial safety factors have to be (re)calibrated. Important aspects within the standard WBI2017 calibration procedure concern the derivation of reliability requirements, the definition of design values on the basis of influence coefficients, and the handling of spatial correlations.

This background report of the WBI2017 concerns the semi-probabilistic assessment rule for the safety of grass revetments (in Dutch: “Gras Erosie BuitenTalud”, GEBU). This failure mechanism consists of two submechanisms: failure due to wave run-up and failure due to wave impact. Both submechanisms are considered in this report. As there is a strong relation between both failure mechanisms and their implications, which will also follow from the report, it is important to consider them together.

The report is organized as follows. Chapter 2 introduces several basic concepts in reliability analysis. Chapter 3 provides a brief overview of the physical background of the failure mechanisms and the relevance of assessing it. It also outlines the choice of the method to be used for deriving semi-probabilistic assessment rules. Chapter 4 outlines the derivation of safety factors for wave run-up, starting with an explanation of the procedure and a description of the calculation steps needed for deriving a semi-probabilistic assessment rule. Furthermore, it presents the resulting safety format and discusses the relation with other research, such as for the Design Instruments (OI). Chapter 5 presents the derivation of a safety format for wave impact. Chapter 6 integrates the findings of both analyses to an overall approach to assessing grass revetments. Conclusions and recommendations are given in Chapter 7.

2 Basic concepts

This chapter presents some basic concepts on failure probabilities, probabilistic and semi-probabilistic analysis. Section 2.1 gives an overview of basic concepts in reliability analysis, section 2.2 gives an explanation on the relations between semi-probabilistic and probabilistic analysis.

2.1 Failure probabilities, reliability indices and influence coefficients

A flood defence will fail when the load exceeds its resistance. The resistance parameters of a flood defence are, in principle, deterministic. In practice, however, they are uncertain due to spatial variability, a limited number of measurements and measurement uncertainties. Also, models such as GrassRunup, that are used to predict critical combinations of parameter values (i.e., combinations that would lead to revetment failure), might produce outcomes that are besides the (unknown) truth. Such model uncertainties also have to be taken into consideration in reliability analyses. This means that the resistance of a flood defence should be treated as a stochastic variable, just like the uncertain loads.

The probability of failure (P_f) equals the probability that load (S) exceeds resistance (R):

$$P_f = P\left(R - S < 0\right) \tag{2.1.1}$$

or

$$P_f = P\left(Z < 0\right) \qquad \text{with} \qquad Z=R\text{-}S \tag{2.1.2}$$

where
Z limit state function

The First Order Reliability Method (FORM) (Rackwitz 2001) is an efficient method to compute failure probabilities. It is also known as a level II approach. In a FORM-analysis, the limit state function is normalized and linearized in the design point. The design point is the combination of parameter values with the highest probability density for which Z=0. The linearized and normalized limit state function (Z_{II}) resulting from a FORM-analysis has the following form:

$$Z_{II} = \beta + \sum_{i=1}^{n} \alpha_i u_i \tag{2.1.3}$$

where
β reliability index
α_i influence coefficient for stochastic variable X_i ($\sum\alpha_i^2$=1),
u_i standard normally distributed variable (mean μ=0 and standard deviation σ=1).

An influence coefficient is a measure for the relative importance of the uncertainty related to a stochastic variable. The squared value of an influence coefficient corresponds to the fraction of the variance (σ^2) of the linearized and normalized limit state function that can be attributed to a stochastic variable.

Generally, a FORM-analysis yields a close approximation of the probability of failure:

$$P\left(Z_{II} < 0\right) \approx P\left(Z < 0\right) \tag{2.1.4}$$

Note that the failure probability estimate $P(Z_{II}<0)$ is exact when the limit state function is linear and all stochastic variables are independent and normally distributed.

The relationship between the probability of failure and the reliability index is as follows:

$$P(Z_{II} < 0) = \Phi\left(-\beta\right) \tag{2.1.5}$$

where
$\Phi(.)$ standard normal distribution function
β reliability index

2.2 The relations between probabilistic and semi-probabilistic assessments

Semi-probabilistic and probabilistic safety assessments are closely related. Both rely on predefined flood protection standards, limit state functions, and the statistical properties of the stochastic variables that represent the uncertain load and strength parameters. The same uncertainties play a role in semi-probabilistic and probabilistic assessments. Yet a semi-probabilistic assessment rests on a number of simplifications and approximations, giving it the appearance of a deterministic procedure.

In probabilistic safety assessments, analysts consider the probability that the ultimate limit state is exceeded, i.e. that load (S) exceeds resistance (R). The probability of failure, $P(S>R)$, should not exceed some maximum allowable ('target') value (P_T). In semi-probabilistic assessments, analysts consider the difference between the design values of load (S_d) and strength (R_d): S_d should not exceed R_d. Design values are defined in terms of representative values (characteristic values such as 5th or 95th quantiles or nominal values) and partial safety factors. This use of terminology is consistent with the Eurocode (the European code for assessing structural reliability). Readers should be aware that similar terms may have different definitions in other international standards.

The design values should be calibrated such that the condition $S_d \leq R_d$ implies that the probability of failure meets the reliability requirement: $P(S>R) \leq P_T$. The relationship between probabilistic and semi-probability safety assessments is illustrated in Figure 2.1.

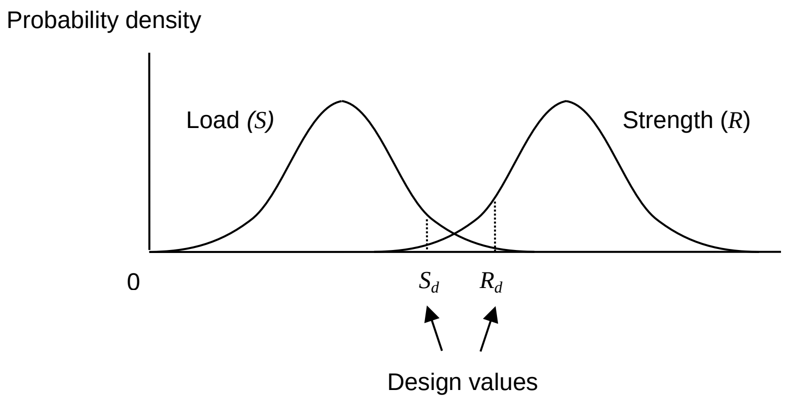

Figure 2.1. The probability density functions of load (S) and strength (R), and the design values of load and strength (S_d) and (R_d).

The design values of normally distributed resistance and load variables are:

$$R_d = \mu_R - \alpha_R \beta_T \sigma_R = R_{rep} / \gamma_R \quad \text{(resistance parameter)} \quad (2.2.1)$$

$$S_d = \mu_S - \alpha_S \beta_T \sigma_S = S_{rep} \gamma_S \quad \text{(load parameter)} \quad (2.2.2)$$

Where

α_R	influence coefficient for stochastic variable R
β_T	target reliability index
μ_R	expected value of stochastic variable R
σ_R	standard deviation of stochastic variable R
R_{rep}	representative value of R (e.g. 5% quantile)
γ_R	partial safety factor

Similar definitions apply to the load parameters. Note that $\alpha_S \leq 0$ while $\alpha_R \geq 0$ and that the representative value of a load variable (S_{rep}) might be the 95% quantile or a value with a probability of exceedance of e.g. 1/10,000 per year.

In short, probabilistic and semi-probabilistic assessments *both* require:

1 A failure mechanism model;
2 Probability density functions for all stochastic variables (based on statistical data and/or engineering judgment);
3 A reliability requirement.

The essential difference between probabilistic and semi-probabilistic assessments is:

1 In a probabilistic assessment, a failure mechanism model is fed with all possible parameter values and their probabilities (probability density functions);
2 In a semi-probabilistic assessment, a failure mechanism model is fed with unique, ‘sufficiently safe’ values (design values). How safe ‘sufficiently safe’ is, depends ultimately on the reliability requirement and a calibration criterion. To ensure sufficient consistency between probabilistic and semi-probabilistic assessments, calibration exercises are indispensable.

3 Modelling the failure of grass revetments

This chapter gives an overview of the mechanisms that may lead to erosion of grass revetments, some considerations on the models used for the semi-probabilistic assessment. Simple assessment rules for the GEBU-mechanism can be found in van Hoven (2015b). Section 3.1 gives an overview of the run-up mechanism, how it is modelled, assessed and where it is relevant. Section 3.2 gives the same overview for the wave impact mechanism.

3.1 Failure of grass revetments due to wave run-up

3.1.1 Modelling of the failure mechanism

Wave run-up failures are caused by erosion of grass layers due to wave run-up. Figure 3.1 gives an overview of the mechanism and its behavior. Under the influence of the uprunning waves, the grass revetment is damaged, after which sublayers erode, after which a breach is initiated. The lowest resistance to run-up is generally found at the transition between a hard (e.g. block/asphalt) revetment and grass, so this is usually the starting point of the erosion. In the model used for run-up residual strength is not considered, therefore the part of the mechanism considered in the assessment is that of the first two figures in Figure 3.1.

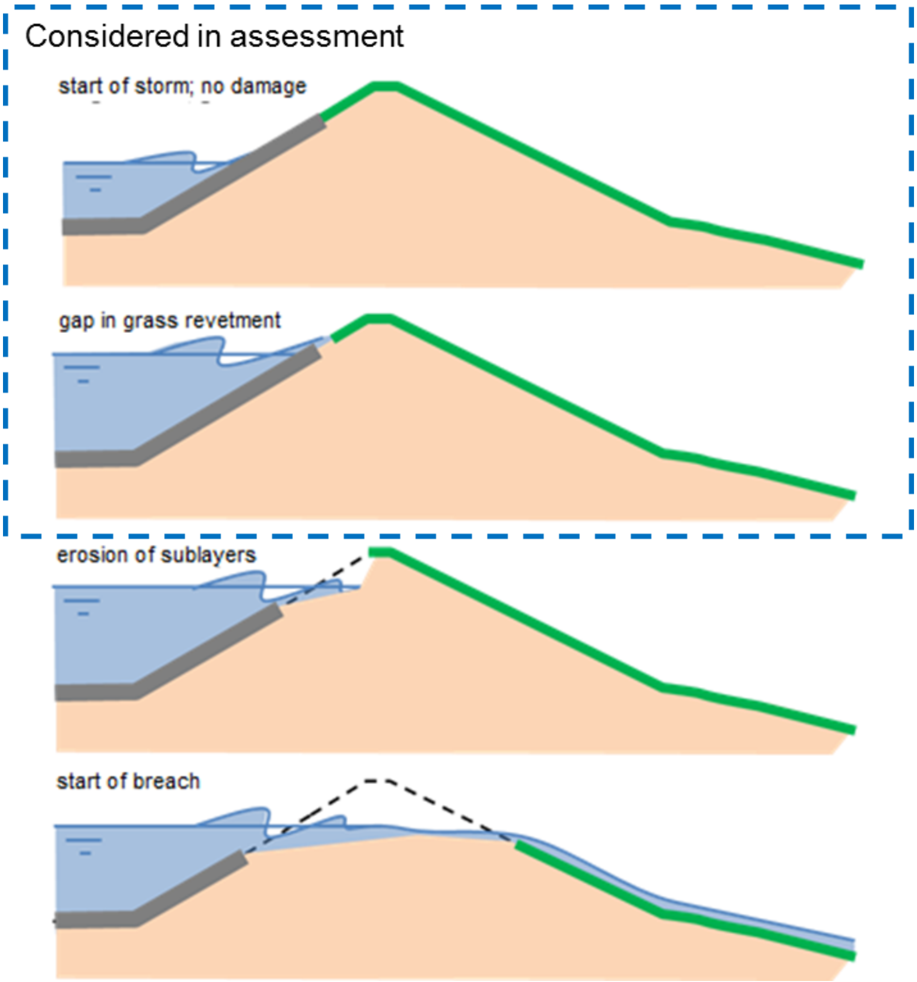

Figure 3.1 Different stages in a failure due to run-up

The failure of the grass revetment due to run-up is described by a cumulative overload model, the cumulative overload is described by the following formula:

$$D = \sum_{i=1}^{N} \max(\alpha_M U_i^2 - \alpha_S U_c^2; 0)$$

with:

D	Cumulative overload [m^2/s^2]
N	Number of waves [-]
U_i	Velocity of wave front of running up wave i out of N [m/s]
U_c	Critical flow velocity, strength parameter of the sod [m/s]
α_s	Strength reduction factor in case of a transition [-]
α_m	Load increase factor in case of a transition [-]

The failure criterion is that if $D > D_{crit}$ the revetment fails. From experiments it follows that D_{crit} is around 7000 m^2/s^2. The cumulative overload is evaluated at the most critical level (Z_{eval}), which is usually the transition between ‘hard’ (e.g. block, asphalt) and grass revetment. Appendix A gives a full overview of the parameters in the failure mechanism model. For more details on the model van Hoven & de Waal (2015) can be consulted.

Grass revetments can have different qualities. An important factor is whether the sod is open or closed. The strength of grass revetments with an open sod is significantly smaller than with a closed sod. In principle, due to the envisioned continuous maintenance (‘continu inzicht’, zorgplicht), open sods should not be present, if the grass revetment is of importance for the assessment result. In such cases dike managers should improve revetments with an open sod such that the sod can be considered closed. However, the safety factors derived here will also be applicable for assessing open sod revetments, provided that certain conditions are met. This will be further discussed in section C.2.3.

3.1.2 Types of conditions relevant for wave run-up failures

Grass revetments are located all over the Netherlands, both in riverine areas, coastal areas and along lakes. Failure due to wave run-up usually occurs at transitions between, for instance, block revetments and grass revetments, as this is the weakest point of the grass revetment. Run-up is generally not a relevant failure mechanism for riverine areas, for two major reasons:

- Transitions between block and grass revetments in riverine areas can generally be found around the daily water level, as blocks are used for preventing damage from ship-induced waves, rather than waves under extreme circumstances. This type of transition is loaded daily and weak spots should be repaired in routine or corrective maintenance (note: these transitions are mostly located below the 1/10 per year water level, which is the lower boundary of the revetment to be considered in the safety assessment). Thus, locations where a transition has to be assessed in the safety assessment will be rare.
- As wave impact (see next section) is considered dominant over run-up, if an assessment will be carried out for wave impact, the revetment will automatically satisfy the requirements for wave run-up. Due to the fact that in riverine areas waves are generally not (strongly) correlated with water levels, the probability of encountering a high wave, high up the slope where wave impact is not assessed, is extremely small. Therefore a sufficiently safe assessment for wave impact will also ensure a sufficiently low failure probability for wave run-up.

Other types of levees can be found along the coast and lakes in the Netherlands. At various locations, grass revetments have been applied to the top part of the revetment as well as the crest. The transitions at current dikes are typically located between 0.2 and 1 meter above the assessment level (i.e. ‘toetspeil’) of the old safety standards. In the past, levees have been designed according to exceedence probabilities of loads, but due to the change in level and definition of safety standards this rule of thumb will no longer be valid for existing revetments. As a consequence, in the assessment the level of the transition compared to the assessment water level will vary per location.

For coastal and lake dikes the same boundary conditions can be used as in the calibration for asphalt and block revetments. Waves are generally high and wave heights are strongly correlated with water levels. It has to be noted that as the run-up model is a cumulative load model, it doesn't necessarily mean that the storm with the highest water level also gives the highest load on the transition, as this also depends on the storm duration and shape of the hydrograph. Therefore multiple combinations of water level and waves have to be considered in the safety assessment.

Of course there can always be local differences in boundary conditions (e.g. higher tide, lower waves), however the aim of a semi-probabilistic assessment rule is not to give a tailor-made solution for every location, but a sufficiently safe general rule. In Chapter 4 a calibration of safety factors for this assessment is carried out.

3.2 Failure of grass revetments due to wave impact

3.2.1 Modelling of the failure mechanism

The failure mechanism relates to failures due to the damage caused by waves that hit the slope at different levels during a storm. Similar to wave run-up a cumulative overload model is used. For each level at the slope, the cumulative overload of the waves is calculated and evaluated based on a standard strength relation for grass revetments, the resistance-duration curve (‘standtijdlijn’). Figure 3.2 shows the schematized effect of wave impact.

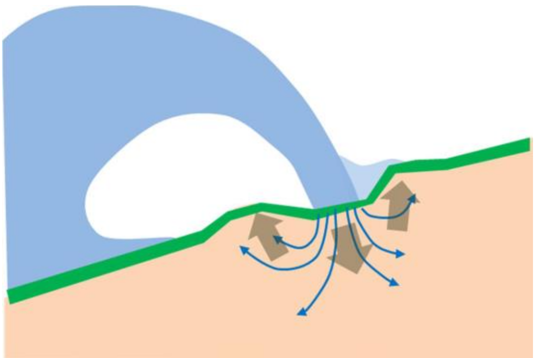

Figure 3.2 Schematic representation of wave impact on a grass slope.

In the assessment, residual strength is also considered. Whereas in previous assessments, as well as the assessment for run-up, only the first 2 images in Figure 3.3 were considered, for wave impact now also the residual strength of clayey sublayers is considered. It has to be noted that it is assumed that the clay layer thickness is limited to 1 meter, which might be a conservative estimate, especially for revetments at the lower end of the slope.

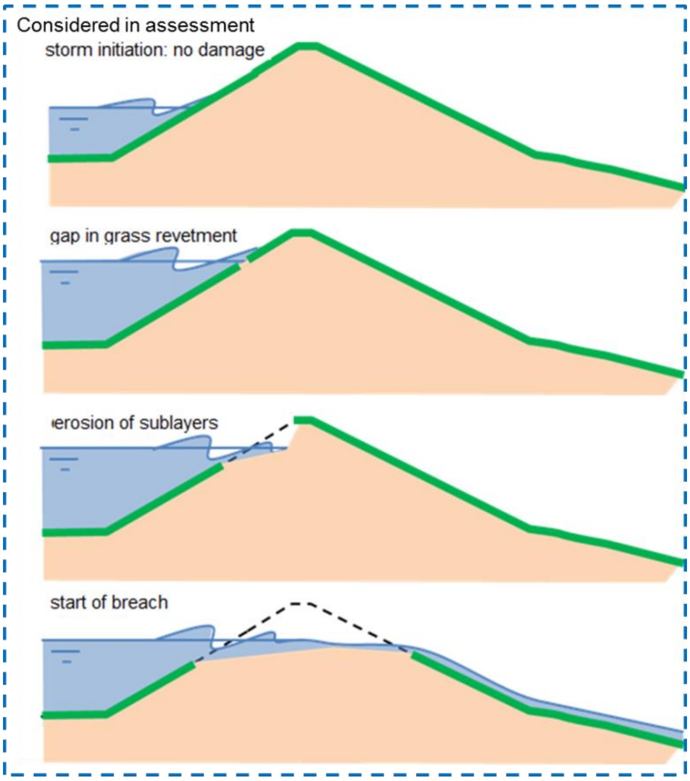

Figure 3.3 Different stages in the failure of a grass revetment due to wave impact

Figure 3.4 shows the resistance-duration curve for a grass revetment. The higher the significant wave height at a certain level the shorter the duration the revetment can withstand the waves. A list of important parameters in the model is given in Appendix A. The implemented functionality in the calculation kernel is described in van Hoven & de Waal (2015a).

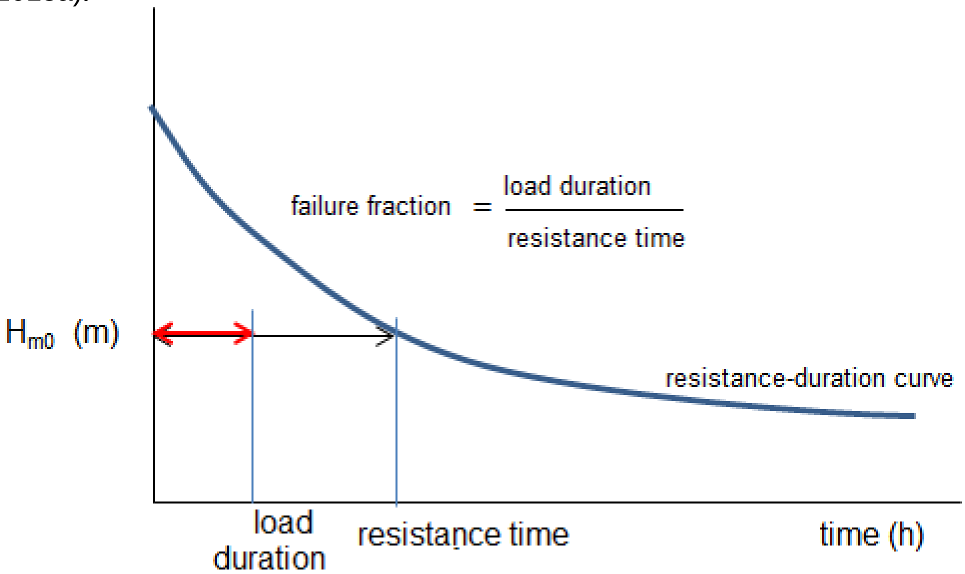

Figure 3.4 Resistance-duration curve, representing the duration that a grass revetment can withstand a significant wave height H_{m0}

The resistance-duration curve from the WTI2011 is shown in Figure 3.5, it has to be noted that some of these lines are slightly conservative and that none of them take residual strength into account.

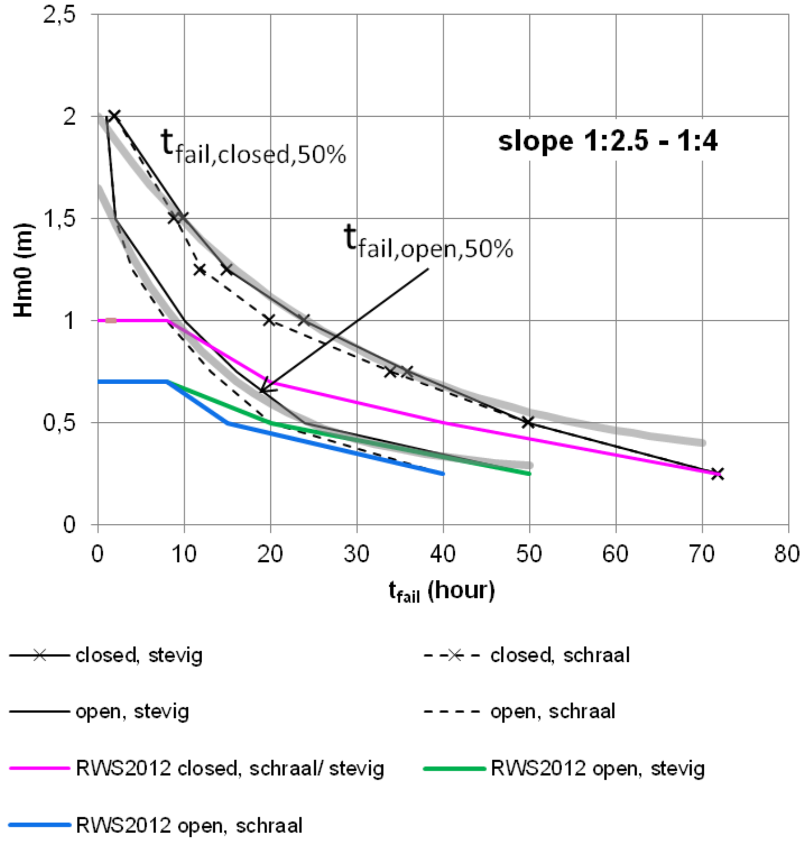

Figure 3.5 Resistance-duration curves for different types of grass revetments as derived in Appendix G. The grey lines are based on experiments, the colored lines represent various (conservative) lines from Rijkswaterstaat (2012).

The resistance-duration curves can be approximated with the following relation:

$$H_{m0} = a \cdot e^{b \cdot t_{fail}} + c$$

where t_{fail} is the time to failure in hours, H_{m0} is the significant wave height in meters and a, b and c are parameters for the resistance-duration curve that are defined by multiple factors, including the grass quality. A derivation of the resistance-duration curves is given in Appendix G.

3.2.2 Types of conditions relevant for wave impact failures

What can be seen is that grass has very limited resistance against wave impact. In the simple assessment, cases with waves smaller than 0.25 meters are approved. Between 0.25 and 1 meters, they have to be assessed based on the curves from the previous figure. Above wave heights of 1 meter they cannot be approved. In many areas the wave height and water level are correlated (e.g. most coastal areas). As the level corresponding to the cross-sectional reliability requirement is much higher than the level corresponding to the statutory safety level, the low resistance of grass revetments against wave impact may cause the grass located directly above the design water level to have a failure probability which is several orders of magnitude higher than the safety requirement,. In Chapter 5 this is discussed in greater detail.

3.3 Overall assessment of grass revetments

To distinguish between wave impact and wave run-up, in previous assessments the assessment water level had a very important role. If the transition was above this level only run-up was considered (i.e. the wave run-up zone), for cases where the transition was below this level only wave impact was considered (i.e. the wave impact zone). For the different zones see Figure 3.6.

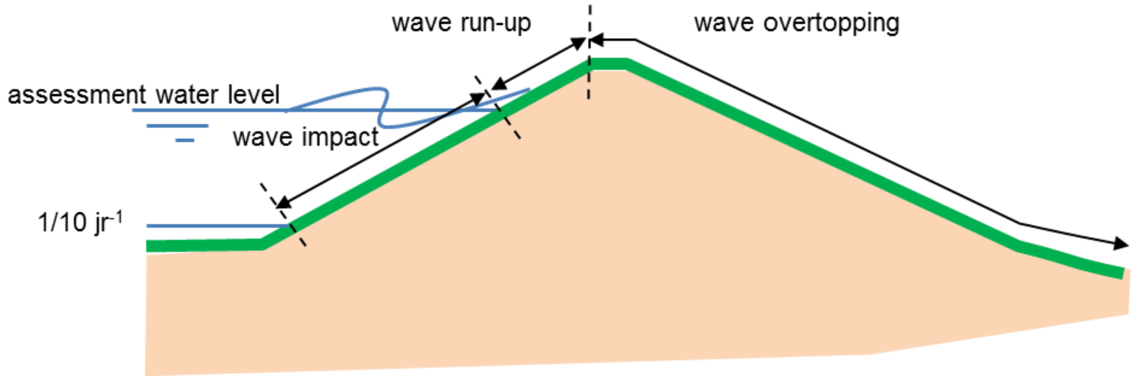

Figure 3.6 Different zones for wave impact, wave run-up and wave overtopping as defined in previous versions of the assessment of grass revetments.

The distinction between the two zones is an approach which is typical for an assessment based on exceedence probability and a given design storm. In the following chapters therefore also the level at which the boundary of the two zones should be located is considered.

4 Semi-probabilistic assessment of wave run-up on grass revetments

This chapter describes the derivation of a semi-probabilistic assessment rule for wave run-up. The procedure used is outlined in section 4.1, after which it is carried out in sections 4.2 to 4.4. In sections 4.5 and 4.6 the relation with current practice and recent research for the Design Instruments is further discussed.

4.1 Calibration procedure

The objective of the calibration procedure is to develop a semi-probabilistic assessment rule for the failure of grass revetments under wave run-up. The procedure used for calibration is similar to the one used in other calibration reports for WBI 2017, and for grass revetments it is as follows (after Jongejan (2013)):

Calibrate partial safety factors without accounting for residual strength:

1 Establish the reliability requirement. This requirement is defined as a maximum allowable probability of failure for the failure mechanism under consideration for an entire segment. The length effect is not yet considered in this step (characterising the length effect requires probabilistic analyses, while defining a maximum allowable probability of failure for an entire segment does not). The length effect is taken into account in step 3.3, when deciding which partial safety factors may be considered sufficiently safe.

2 Establish the safety format. Based on the outcomes of probabilistic calculations and practical considerations, define representative values and decide on the partial safety factors that are to be included in the semi-probabilistic assessment rule.

3 Establish safety factors. This step comprises the following activities:

- 3.1 Establish, on the basis of representative influence coefficients and a target reliability index, the values of all but one partial safety factor. Herein, these partial factors will be called β_T-invariant safety factors (β_T stands for the required, or target, reliability index).
- 3.2 For each test set member, determine the required value for the design parameter (i.e. the parameter with the β_T-dependent safety factor) so that R_d=S_d, for a range of values of the remaining β_T-dependent safety factor. When this condition is fulfilled, each (modified) test set member would just pass a semi-probabilistic assessment. Then calculate the probability of failure of each (modified) test set member. The objective of this step is to establish a relationship between the value of the β_T-dependent safety factor and the probability of failure, for each test set member
- 3.3 Apply a calibration criterion to select the appropriate value of the β_T-dependent safety factor. The calibration criteria provide a reference for deciding which design values are sufficiently safe. An analysis of the length-effect is part of this evaluation. The failure probability of a segment should, on average, be smaller than the flood protection standard that applies to the segment. A segment typically consists of a number of different sections.

Interpret the results and establish the semi-probabilistic assessment rule

4 Decide on appropriate partial safety factors on the basis of the results of the previous steps, the uncertainties in the analyses, and the practical implications of differentiating between groups of cases.

5 Compare the semi-probabilistic assessment rule from step 4 with the present-day rule (WTI2011) and estimate the potential consequences for the assessment.

An overview of the abovementioned steps is presented in Figure 4.1.

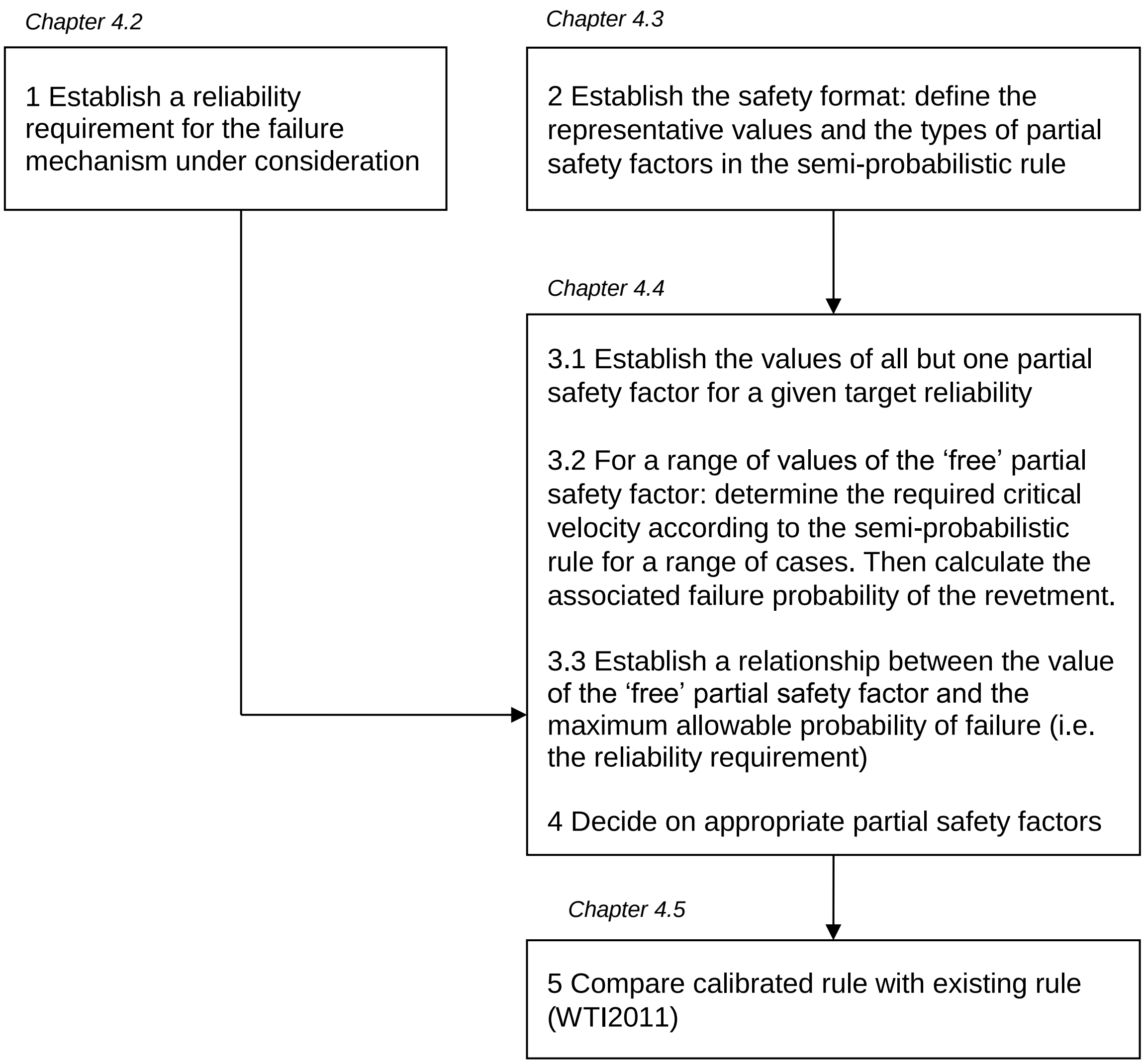

Figure 4.1 Flow chart for the calibration procedure

4.2 Step 1: Establishing a reliability requirement

This chapter discusses the establishment of the reliability requirement that forms the basis for the semi-probabilistic assessment rule. It starts with a safety standard (section 4.2.1), from which the reliability requirement for revetments is derived (section 4.2.2). Because there are numerous types of revetments and (sub)failure mechanisms, the latter requirement has to be translated into a reliability requirement for the stability of grass revetments under run-up loads. This is the subject of section 4.2.3.

4.2.1 Maximum allowable probabilities of flooding

The flood protection standards will be defined in terms of maximum allowable probabilities of flooding. These standards will apply to segments. A segment is a levee system or part thereof. Segments are rarely over 20 km long, they are never located along more than one water system (e.g. lake, river or sea) and in general have fairly uniform orientations. Segments may consist of numerous dike sections and/or hydraulic structures.

4.2.2 Reliability requirement for revetments in general

For calibrating a semi-probabilistic assessment rule for a particular failure mechanism, a reliability requirement for that failure mechanism is needed. Such a reliability requirement can be derived from a fault tree analysis. Each failure mechanism may lead to flooding, the fault trees top event. The combined probabilities of the various failure mechanisms may not exceed the flood probability defined by the safety standard. To ensure this requirement is met, the failure probabilities for the failure mechanisms, their ‘failure probability budgets’, should be defined in such a manner that their combined value does not exceed the total flood probability.

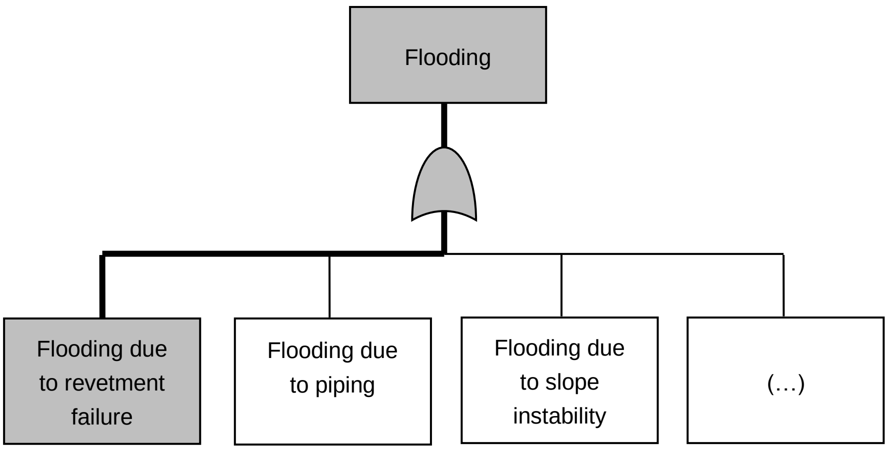

Figure 4.2. A fault tree with different failure mechanisms.

The maximum allowable contributions of the different failure mechanisms to the total probability of flooding are shown in Table 4.1. The percentages in Table 4.1 are based on the expected importance of the different failure mechanisms if all levee systems were to meet their (assumed) flood protection standards. These estimates are based on calculations with PC-Ring and VNK2-data as well as a number of expert sessions with representatives of research institutes (TNO, Deltares, Delft University of Technology), engineering consultancies, water boards, and Rijkswaterstaat. For further details about the maximum allowable failure probabilities per failure mechanism, the reader is referred to Jongejan (2013).

Table 4.1. Maximum allowable failure probabilities per failure mechanism, defined as a percentage of the probability of flooding.

Type	**Failure mechanism**	**Type of segment**	
		Sandy coast	**Other (levees)**
Levee and structure	Overtopping	0%	24%
Levee	Piping	0%	24%
	Macro instability of the inner slope	0%	4%
	Revetment failure and erosion	**0%**	**10%**
Structure	Non-closure	0%	4%
	Piping	0%	2%
	Structural failure	0%	2%
Dune	Dune erosion	70%	0% / 10%
Other		30%	30 /20%
Total		100%	100%

The choice for the term 'revetment failure *and erosion*' in Table 4.1 is deliberate, even though the failure mechanism is commonly referred to as 'revetment failure' only. If we were to ignore residual strength in levee safety assessments, this would be equivalent to assuming that the probability of flooding conditional on a revetment failure is equal to one. For some failure mechanisms residual strength is taken into account (e.g. failure of block revetments), for some it is not. For run-up on grass revetments residual strength is taken into account in the value for critical overload (D_{crit}).

4.2.3 Reliability requirement for grass revetments under run-up loads

The 10%-value in Table 4.1 relates to all revetments, not only grass revetments, and to a range of (sub-)failure mechanisms, see Figure 4.3. This chapter is concerned solely with the stability of grass revetments under run-up loads. A reliability requirement for grass revetments and this particular failure mechanism can, again, be derived from a fault tree analysis. It has to be noted that in the fault tree the failure mechanisms for grass which are mentioned are only the ones for failure of the outer slope. Failure mechanisms for the inner slope (GABI and GEKB) are considered under the main failure mechanism of overtopping.

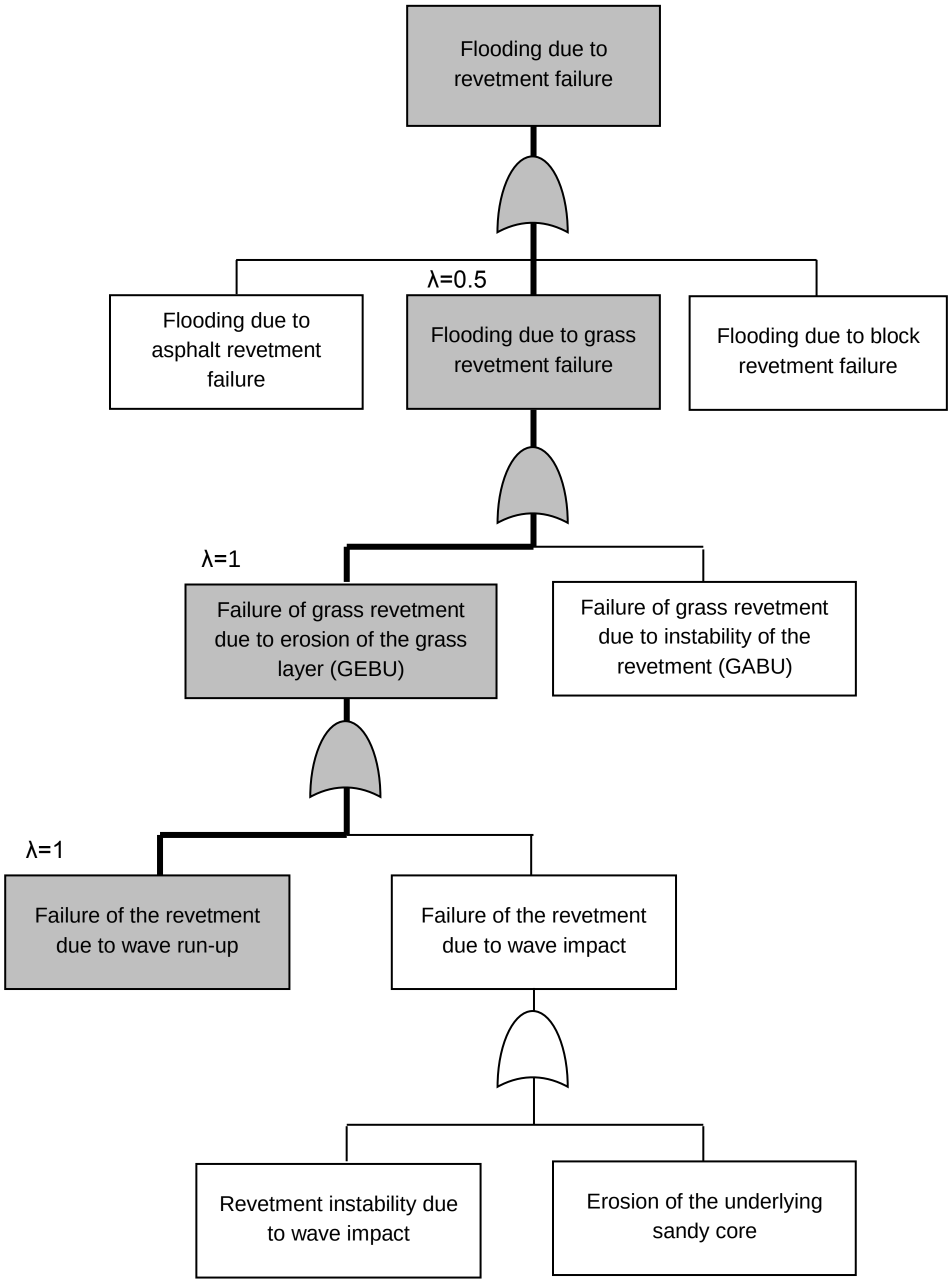

Figure 4.3. Fault tree for flooding due to revetment failure. The parts of the fault tree that this study is concerned with have been highlighted.

Let f be the maximum allowable contribution of revetment failures to the probability of flooding (f=0.10, see Table 4.1), let λ_1 be the contribution of *grass revetments* to the probability of flooding due to revetment failures (all types), λ_2 the contribution of failures of grass revetments due to failure of the grass layer, and let λ_3 be the contribution of failures of grass revetments caused by *wave run-up* to the overall probability of failure of a grass revetment. The reliability requirement for the stability of grass revetments under wave run-up loads then becomes:

$$P_T = f \lambda_1 \lambda_2 \lambda_3 P_{Norm} \tag{4.2.1}$$

where
P_T probability of flooding due to the series of events triggered by the instability of a grass revetment under wave run-up that lead to flooding

The values for λ_1 λ_2 and λ_3 have been established on the basis of the results of the VNK2-project and expert judgment:

λ_1: Compared to other calibration studies for asphalt respectively block revetments values for λ_1 are 0.33 and 0.5. As grass revetments are generally present along the entire segment assuming a value of 0.5 for the failure space of grass revetments seems reasonable. This will lead to a total value of λ_1 of slightly larger than 1, which is not a problem as it is very unlikely that all revetments at the entire dike segment are exactly at their limit.
λ_2: For failures of grass revetments at outer slopes there are two main mechanisms. The first one is GEBU ('Gras Erosie BuitenTalud'), which concerns the erosion of the grass layer. The second one is GABU ('Gras Afschuiving BuitenTalud') which concerns the instability of the outer slope. As the probability of erosion is usually much larger than instability, an appropriate value for λ_2 would be 0.9. For simplicity sake a value of 1 is used.
λ_3: The GEBU-mechanism is caused by two main driving forces: wave impact and wave run-up. As mechanisms for failure of (grass) revetments are correlated it is unlikely that all mechanisms will be at their limit state and use their entire probability budget. Therefore a value of 1 is chosen for λ_3.

If an unduly high (lenient) value of f, λ_1, λ_2, λ_3 is chosen for 1 mechanism, in principle this implies an unduly stringent reliability requirement for another failure mechanism (see also Jongejan, 2013). For revetments, the probability that all three (types of) revetments are at the maximum of their failure probability is very small. Therefore the budgets are dealt with in a lenient way and the total is slightly higher than 1. It has to be noted that minor changes in the failure probability budgets do not have a significant influence on the reliability requirements, as in most cases it will only cause a change of approximately a factor of 2, which is small in terms of probability.

The abovementioned values of f, λ_1, λ_2, λ_3 and the resulting maximum allowable failure probabilities for grass revetments under wave attack (P_T) are shown in Table 4.2. The reliability requirements are also expressed in terms of reliability indices ($\beta_{T,seg}$).

Table 4.2. Reliability requirement for a range of flood protection standards.

f (-)	λ_1 (-)	λ_2 (-)	λ_3 (-)	P_{max} (yr^{-1})	**Reliability requirement (entire segment)**	
					$P_{T,seq}=f\lambda_1\lambda_2\lambda_3 P_{max}$ (yr^{-1})	$\beta_{T,seq}=-\Phi^{-1}(P_{T,seq})$ (on an annual basis)
0.10	0.5	1	1	1/300	1.67E-04	3.59
				1/1000	5.00E-05	3.89
				1/3000	1.67E-05	4.15
				1/10000	5.00E-06	4.42
				1/30000	1.67E-06	4.65
				1/100000	5.00E-07	4.89

It should be noted that the reliability requirements ($P_{T,seg}$ or $\beta_{T,seg}$) in Table 4.2 apply to segments. These should *not* be confused with cross-sectional reliability requirements. Due to the length effect, cross-sectional reliability requirements will have to be more stringent than reliability requirements for entire segments. The relationship between the reliability requirement for entire segments ($P_{T,seg}$ or $\beta_{T,seg}$) and cross-sectional failure probabilities is discussed in greater detail in section 4.4.1

4.3 Step 2: Establishing the safety format

4.3.1 Defining a test set

To obtain insight into the relative importance of the numerous stochastic values, probabilistic analyses were carried out for a large number of test set members. The test set members reflect the wide variety of geometries and load conditions found throughout the Netherlands. While the test set members were inspired by actual levees, they are fictitious in the sense that they cannot be linked to specific locations. The test set members were defined based on two sources:

- Experimental data, mostly from overtopping tests.
- Expert judgement of representative parameter ranges.

Based on these two sources test set members with combinations of parameters were defined. For some parameters one representative distribution was analysed, for others the average values were varied along intervals, to represent variation in revetments. By combining the selected averages with default standard deviations, coefficients of variation and distribution types, the probability density functions of all parameters were defined. For further details about the test set, see Appendix C.

4.3.2 Partial safety factors and representative values

4.3.2.1 Representative values

From the initial analysis of some revetments, influence coefficients for different parameters were found (for an explanation on the meaning of influence coefficients see section 2.1). This resulted in the (squared) values shown in Figure 4.4, for the Wadden Sea at a safety standard of 1/10000 and a slope of 1:4. The cases selected all have a resulting reliability index between 3 and 6, which corresponds to realistic cross-sectional reliability requirements.

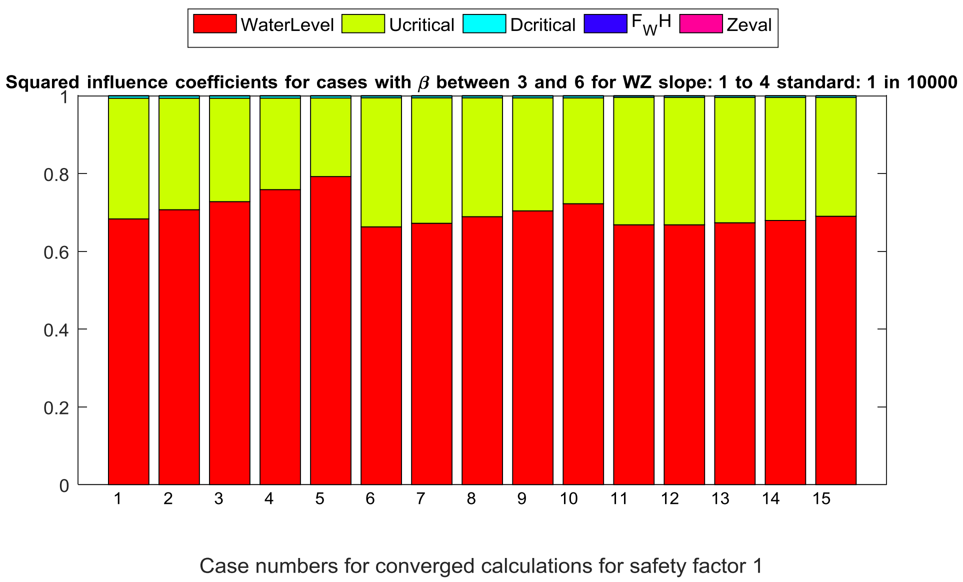

Figure 4.4 Squared influence coefficients for a selection of test set cases. The case number (between 1 and 16) represents the case in the test set, the value between 1 and 1.5 the safety factor applied in the calculation.

The load, represented by the water level, and U_c, the critical run-up velocity, have high influence coefficients. Therefore it is not appropriate to represent these with a mean value, but a more conservative representative value should be chosen. For U_c, given that it is a strength parameter, a 5%-quantile value is chosen. Within WBI, the load with an exceedance probability equal to the safety standard of the segment is generally used as the representative load. However, as for grass impact failures it is necessary to assess with an exceedance probability equal to the cross-sectional reliability requirement (see chapter 5), in order to obtain a consistent safety format. This results in the representative values shown in Table 4.3.

4.3.2.2 *Partial safety factors*

Strictly speaking a ‘perfect’ safety format should cover all uncertainties in all parameters depending on the target reliabilities and influence coefficients. As the uncertainty of most variables has a minor influence on the failure probability, the complexity of the safety format can be reduced by applying a limited number of safety factors rather than 1 for every parameter (i.e. all other partial safety factors are set to 1). In this case a β_T-dependent safety factor is applied to U_c.
This choice is made for several reasons. First, this is the most important strength parameter; secondly experts have ‘feeling’ for this parameter, which enables them to evaluate the resulting safety factors based on expert judgment (“sanity check”). This reduces the risk that safety factors are unrealistically stringent or optimistic. The format of the semi-probabilistic rule thus becomes:

$$D_{load} > D_{crit} \tag{4.3.1}$$

with:

$$D_{load} = \sum_{i=1}^{N} \max\left(\alpha_{M,z} U_{i,z}^2 - \alpha_{S,z} \left(\frac{U_c}{\gamma_\beta} \right)^2 ; 0 \right) \text{ and } D_{crit} = 7000 \text{ m}^2/\text{s}^2. \tag{4.3.2}$$

So the value of U_c is reduced with a beta-dependent safety factor γ_β. For the assessment the representative values are defined as indicated in Table 4.3.

Table 4.3 Parameters with their representative values and partial safety factors to be used in the assessment.

Parameter description	Symbol	Representative value	Partial safety factor
Factor for increased load at transitions and objects	α_m	mean value	1
Factor for decreased strength at transitions and objects	α_s	mean value	1
Critical value of cumulative overload	D_{crit}	mean value	1
Critical wave runup front velocity along the slope	U_c	5%-quantile value	beta-dependent
factor between runup level and maximum front velocity	c_u	1.1	1
Load	-	equal to cross-sectional reliability requirement	1

A summary on how to apply the assessment rule is given in Appendix A.

4.3.3 Model uncertainties in the wave run-up model

In the cumulative overload model there is no model uncertainty factor. This implies that the model uncertainty is covered by the different random variables in the model. Thus there will not be a β-invariant model factor covering this uncertainty.
It is however important for the interpretation of the results what uncertainties are at least qualitatively known, as this can help interpreting the results and directing future research:

- Uncertainty in c_u: the factor c_u which relates wave height to wave run-up shows a high variability per wave. Based on analysis of different experiments the c_u for individual waves is found to have a mean of 1 and a coefficient of variation of 0.25. For entire storms a value of 1.1 is estimated to be a safe but not too conservative estimate, but experimental results also show variability for different cases. Currently there is insufficient knowledge on the distribution for c_u per storm. Therefore the value of 1.1 is used in the calibration. As said, there is quite some uncertainty on this parameter, the value for c_u can be interpreted as a beta-invariant model factor for this specific parameter, for which 1.1 is a first estimate. This requires further investigation in the future. A more thorough explanation on the value for c_u is given in van Hoven (2015) and Appendix D.
- Uncertainty in U_c: the critical run-up velocity is the most important strength parameter and it is quite uncertain. The uncertainty comes from two main sources:
  - The first is that, based on expert opinions and experiences from experiments, it is likely that U_c-values for run-up are higher than for overtopping. As the default distribution for run-up is based on U_c-values for overtopping this is, most likely, a conservative distribution.
  - The second source of uncertainty is in the distribution type of the distribution for U_c. This distribution is based on 10 experimental cases in the Netherlands. There are two main potential issues with this distribution:
    - The coefficient of variation of U_c for a single dike section is most likely much smaller than the one obtained from taking the average of 8 separate cases. Thus in fact the distribution is not representative for a single dike section.
    - In the experiments, values of U_c larger than 8 m/s could not be measured (due to limitations in equipment). There are a few cases in the distribution which likely have a U_c larger than 8 m/s. Thus the distribution is conservative. This has been partially solved by subjectively modifying the data set.

  Future research should focus on a better description of U_c, as this is found to be the most important strength parameter.
- Uncertainty in α_m and α_s: currently the influence factors for transitions are assumed to be 1. However, experts do not agree on the value that should be used for α_m, and estimates vary between 1.2 and 1.8. As this research is not yet implemented, it has been decided to use a value of 1 for both factors. This is most likely an optimistic assumption, considering the findings of the aforementioned research.

Considering the uncertainties discussed above, not all uncertainties are incorporated in the random variables. Therefore a model uncertainty factor should be used in the model. However, most of the aforementioned uncertainties have not been quantified yet, so no sensible value for the model uncertainty can be used at this moment. It is advised to further investigate this in the future.

4.4 Step 3: Calibrating partial safety factors

This section deals with the calibration of the partial safety factor γ_β. As assessments are done per cross section, the first step is to determine the relation between reliability requirements for segments and cross sections, which is done in section 4.4.1. Section 4.4.2 describes the calibration of the safety factor. Section 4.4.3 discusses the results and the consequences in terms of safety factors to be used in different cases.

4.4.1 The calibration criterion and dealing with length effects

According to the WBI2017 calibration criteria, the failure probability of a segment should, on average, be smaller than the safety standard (Jongejan 2013). When relating the cross-sectional reliabilities of individual test set members to reliability requirements that apply to entire segments, the length effect has to be accounted for. Ideally, this is done on the basis of probabilistic analyses for entire segments, using the computational techniques available in Hydra-Ring (or PC-Ring). Unfortunately, sufficient (probabilistic) load and resistance data are not yet available. The length effect has therefore been characterised on the basis of an evaluation of the various contributors to the length effect.

For describing the length effect one method is to characterise it by a factor N, which may be interpreted as a number of independent, equivalent revetments. The following condition should be met:

$$N \cdot P_{cross,avg} \leq P_{T,seg} \tag{4.4.1}$$

which is equivalent to:

$$P_{cross,avg} \leq \frac{P_{T,seg}}{N} = P_T \tag{4.4.2}$$

where

$P_{cross,avg}$	average cross-sectional probability of failure
P_T	cross-sectional target failure probability
N	Number of independent, equivalent revetments
$P_{T,seg}$	target failure probability of the segment

This characterization is especially useful if contributing factors to the length effect are not length dependent (e.g. different orientations). This characterization has been used for describing e.g. the length effect for block revetments. If the spatial variability of the uncertain resistance is large, such as for asphalt revetments, then it can also be described by the length of an independent, equivalent section b, for which it holds that $b=L/N$. For such cases the length of a segment has a strong influence on the relation between cross-sectional failure probabilities and the safety standard.

For other revetment types, e.g. block revetments, it is assumed that $N = 4$, while for asphalt revetments it is assumed that $b=L/N=1000$ meters. For asphalt revetments the factor N is thus strongly dependent on the length of the segment. What kind of characterization should be used should follow from an analysis of the different contributors.

As the strength of the grass revetment for run-up loads is determined by the transition between two types of revetments there is no vertical length effect for closed sod grass layers (for open sods there might be, but these are not considered in this assessment). Thus there are two remaining major contributors to the length effect:

1 Spatial variability of the strength in horizontal direction: the spatial variability of the strength in horizontal direction strongly determines the length-effect. The spatial variability of the strength for grass revetments, given that they have a uniform substrate and a similar maintenance regime is expected to be fairly small. For overtopping, which relies on the same cumulative overload model, a factor of 1 between the failure probability of a statistically homogeneous stretch and a cross section has been assumed in the OI2014v3 (Rijkswaterstaat 2015). This is implies that the spatial variability of the resistance of grass resistance is expected to be very small relative to the spatially correlated epistemic/knowledge uncertainty. In line with the OI2014v3, a factor 1 is assumed here.

2 Different orientations: A length effect may arise from cross sections having different orientations. Most segments have fairly uniform orientations, especially as segments are smaller than the dike rings used before. The variation in orientation of the levees within a segment is typically smaller than 90-180 degrees. In the OI2014v3, the factor for different orientations was assumed to be between 1 and 3 for overflow. For sea and lake dikes the factor is 2 or 3, in this report a factor 3 will be used.

Multiplying the aforementioned factors this would result in a factor N = 3 for most relevant locations. In the remainder of this report a default value of N = 3 will therefore be used. Please note that the influence of using N = 2 or 3 is very small. Reliability requirements for cross sections are given in Table 4.4.

Table 4.4 Resulting cross-sectional reliability requirements.

f (-)	λ_1 (-)	λ_2 (-)	λ_3 (-)	P_{max} (yr^{-1})	Reliability requirement (entire segment)	**Reliability requirement (cross-section)**	
					$P_{T,seg}=f\lambda_1\lambda_2\lambda_3P_{max}$ (per year)	$P_T=f\lambda_1\lambda_2\lambda_3P_{max}/N$ (yr^{-1})	$\beta_T=-\Phi^{-1}(P_T)$ (on an annual basis)
0.10	0.5	1	1	1/300	1.67E-04	5.56E-05	3.86
				1/1000	5.00E-05	1.67E-05	4.15
				1/3000	1.67E-05	5.56E-06	4.39
				1/10000	5.00E-06	1.67E-06	4.65
				1/30000	1.67E-06	5.56E-07	4.87
				1/100000	5.00E-07	1.67E-07	5.10

4.4.2 Calibrating β-dependent safety factor

To calibrate the β-dependent overall safety factor (γ_β), first the test set members are modified such that their limit state functions would equal 0 in semi-probabilistic assessments given a certain safety factor γ_β. The critical velocity of the grass revetment is used as design variable while other values are taken as representative values as outlined in section 4.3.2. The required critical velocity to satisfy the criterion is subsequently determined (including its back-calculated distribution, assuming the required critical velocity is the 5% lower bound).

Next, a reliability index is calculated for each test case given the back-calculated distribution of the critical velocity. This results in a (increasing) relation between reliability indices and safety factors for the whole test set as the assessment criterion becomes more stringent. Finally, a relation is fitted through the resulting cloud of safety factors (γ) and reliability indices. For more information about the calibration process, see Jongejan (2013). An example result for the Western Scheldt, for a 1/10,000 year safety standard is given in Figure

4.5. Here it can be seen that the cases(red dots) are quite close together, other calibrations (for other systems/safety standards) give a similar impression.

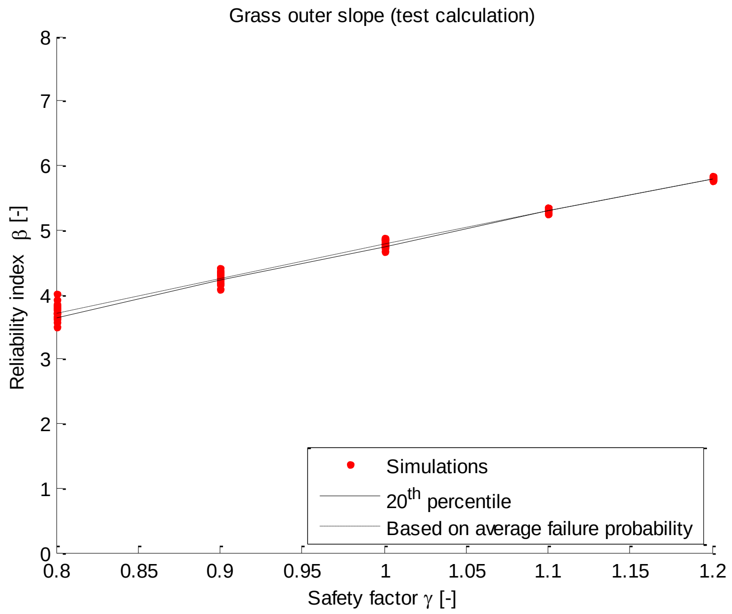

Figure 4.5 Calibration result for the Western Scheldt for a safety standard of 1/10,000 year^{-1}. Red dots indicate individual cases, black (dashed) lines the fits based on the 20th percentile of the reliability index and the average failure probability.

Figure 4.6 shows the relation between the calibrated safety factor and reliability index following for the Wadden Sea. It also includes the fitted relations. Generally target reliabilities for cross sections are in a range of 4 to 5.5, depending on the safety standard. The relations were fitted to the calibration results for a corresponding range.

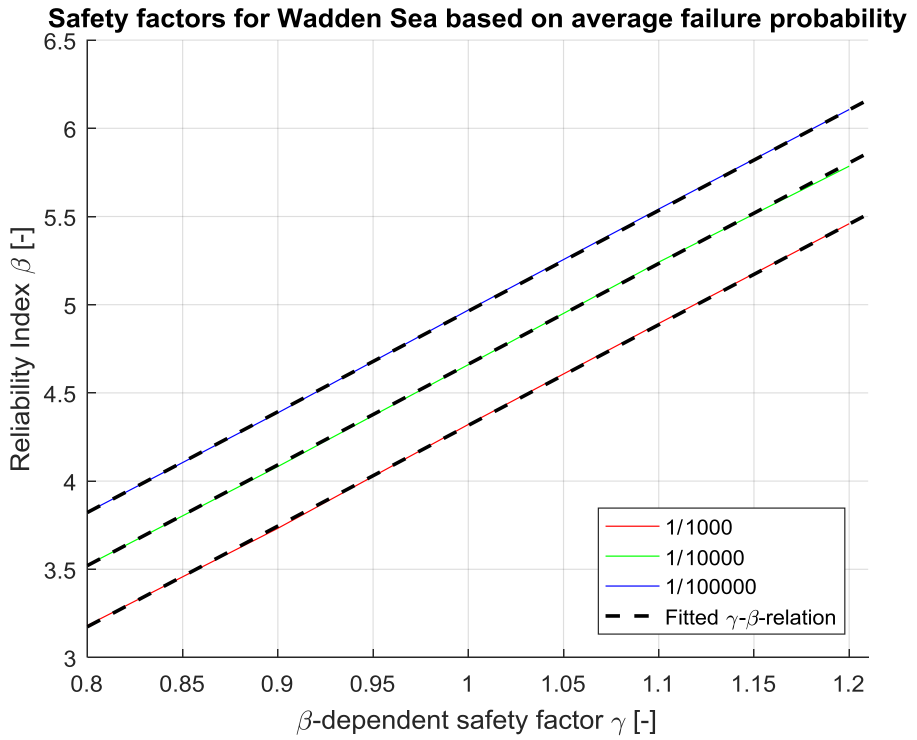

Figure 4.6 Resulting β-γ relations for the Wadden Sea. The legend shows different values of the safety standard

For Lake IJssel the results of the calibration are shown in Figure 4.7, for the Western Scheldt in Figure 4.8.

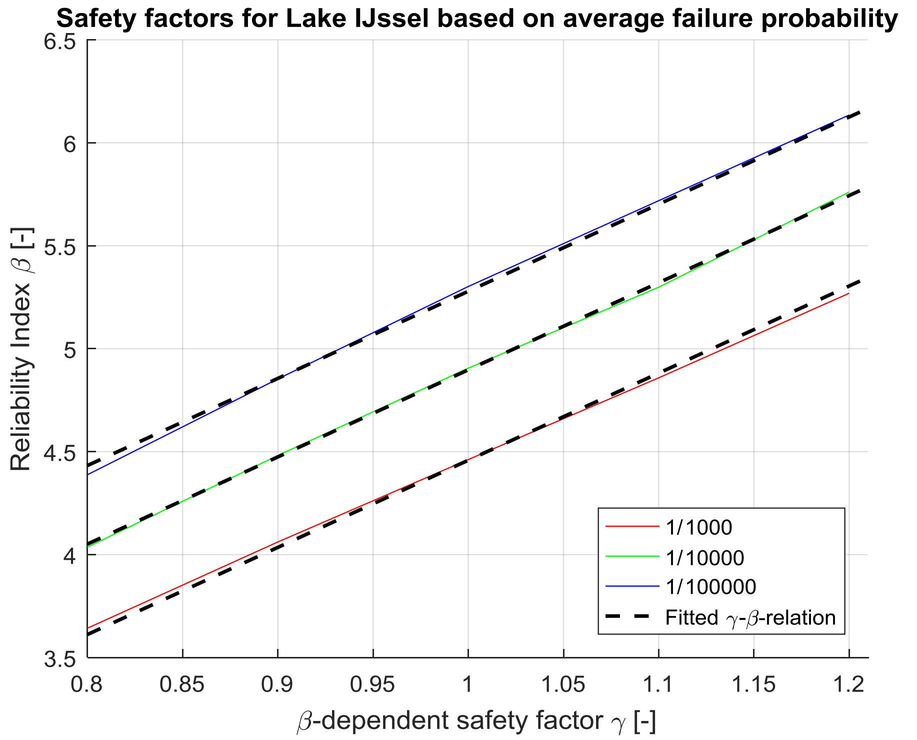

Figure 4.7 Resulting β-γ relations for Lake IJssel. The legend shows different values of the safety standard

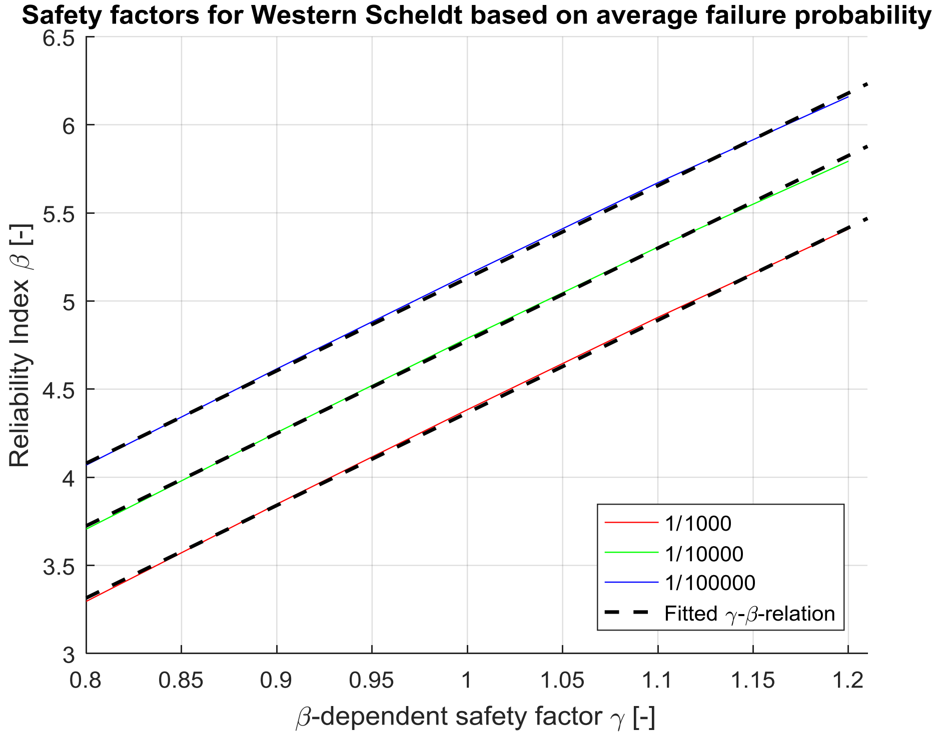

Figure 4.8 Resulting β-γ relations for the Western Scheldt. The legend shows different values of the safety standard.

The derived lines were based on the fitting procedure as described in Appendix F. For all cases the results for γ_β between 1 and 1.3 were included for fitting, as this is the most relevant range. The resulting fitted formulas for the safety factor can be used for deriving a safety factor given a safety standard and cross-sectional reliability requirement. The formulas for the three subsystems are shown in Table 4.5.

The relation between reliability index β_T and safety factor γ_β has the following format:

$$\gamma_\beta = c_a \cdot (\beta_T - c_b) + c_{norm} \quad \text{with:} \quad \beta_{T,cs} = -\Phi^{-1}\left(min(\frac{f\lambda_1\lambda_2\lambda_3 P_{max}}{N}) \right) \tag{4.4.3}$$

Where:

c_a,c_b	constants
c_{norm}	a coefficient that depends on the safety standard
f	the maximum allowable contribution of revetment failure to the probability of flooding (f=0.1)
λ_1	the contribution of *grass revetments* to the probability of flooding due to revetment failures (all types) (λ_1=0.5)
λ_2	the contribution of failure of the grass revetment to the overall probability of failure of a grass revetment (λ_2=0.9)
λ_3	the contribution of failures of grass revetments caused by *wave run-up* to the total probability of failure of a grass revetment (λ_3=1)
P_{max}	target failure probability of the dike segment
N	representative number of independent reaches in a segment [-]
$\beta_{T,cs}$	Cross sectional target reliability index [-]

To derive the formulas in Table 4.5, a least squares fit was made for the calibration results. The fitted lines are represented by the black dashed lines in the figures on the previous pages. More information on the fitting method can be found in Appendix F.

Table 4.5 $\beta_{T,cs}$ -γ_β relations for the different subsystems

Water system	**β_T-dependent safety factor**
Western Scheldt	$\gamma_\beta = 0.191(\beta_{T,cs} + 2.894) - 0.124\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
Wadden Sea	$\gamma_\beta = 0.175(\beta_{T,cs} + 3.095) - 0.097\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
IJssel lake	$\gamma_\beta = 0.236(\beta_{T,cs} + 1.931) - 0.165\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$

4.4.3 The resulting partial safety factors

The lines derived from the calibration calculation yield virtually identical safety factors for different water systems. Table 4.6 shows the resulting safety factors for the failure probability budget and length effect factor as outlined in section 4.4, using the formulas from Table 4.5.

Table 4.6 Different safety standards and safety factors to be used in the assessment

P_{norm}	$\beta_{T,cs}$	Lake IJssel	Wadden Sea	Western Scheldt
1/300	3.86	0.92	0.96	0.95
1/1000	4.15	0.93	0.97	0.96
1/3000	4.39	0.93	0.98	0.97
1/10000	4.65	0.94	1.00	0.98
1/30000	4.87	0.95	1.01	0.98
1/100000	5.1	0.96	1.02	0.99

From this table it follows that the safety factors vary only slightly per safety standard and water system but are all close to 1. In general it has to be noted that the values in the table should be interpreted as an indication of the required value of the safety factor rather than a precise value. In this case most values are slightly lower than 1, but from a usability perspective a safety factor of 1 leads to a clear and easy-to-use safety format. Therefore it is proposed to assess all revetments in all coastal areas and lakes with a partial safety factor γ_β equal to 1, provided that the default probability budget is used. The safety factor should be applied to the 5%-quantile value of the critical run-up velocity U_c. If the failure probability budget is changed the formulas given in Table 4.5 have to be used.

It has to be noted that this choice means that the safety format is a combination of load with exceedance probability equal to the cross-sectional reliability requirement, combined with a 5%-value of the strength. This is a pragmatic choice in order to obtain a clear and usable safety format. From a basic analysis of influence coefficients it can be concluded that this safety format cannot directly follow from the design points found in the calibration, as the chosen representative value for the load implies an influence coefficient of the load α_S=1 and the chosen representative value for the strength implies an influence coefficient α_R≈0.4 (for a β of 4). This leads to a sum of α^2 larger than 1 which is theoretically impossible. However, if a safety format with e.g. an adapted exceedance probability of the load would be used this would result in a reduction of the exceedance probability of for instance a factor 3, which is marginal in terms of probability. If that would be done the safety format would be considerably less clear and such reduction factors would have to be derived for all combinations of water systems and safety standards.

4.4.4 Sensitivity analysis

This section presents some sensitivity analyses to ensure the test set has properly represented the variety of possible revetments. It has to be noted that the sensitivity analyses were carried out with the test set as defined in a preliminary version of the report. In this version there was a variation between values for c_u and the influence factors of the transition were still taken into account. In this chapter for α_R[1] a truncated normal distribution with boundaries 1 and 2, coefficient of variation of 0.2 and a variable mean of 1.2, 1.4, 1.6 or 1.8 was used. For α_S a normal distribution with mean 0.95, coefficient of variation 0.05 and lower and upper bounds of 0.8 and 1 was used. Also the exceedence probability of the load was assumed equal to the safety standard. As it is a sensitivity analysis the findings are not influenced by this, but resulting safety factors differ from the ones found in the previous section.

4.4.4.1 Slope

In a preliminary calibration study a distinction was made between different slopes in the test set. Several slope angles were considered in that study. However, from the results it appeared that the resulting safety factors were insensitive to the slope angle. Figure 4.9 shows the results for three variations on the test set used for the calibration. It considers the same case for the Wadden Sea, with P_{max} = 1/10,000 and various slopes.

From this it appears that the influence of the slope on the result is relatively small. It has to be noted that for the case of a slope of 1:6 for higher safety factors less cases converged, as it was impossible to find a design value for U_c corresponding to Z=0.

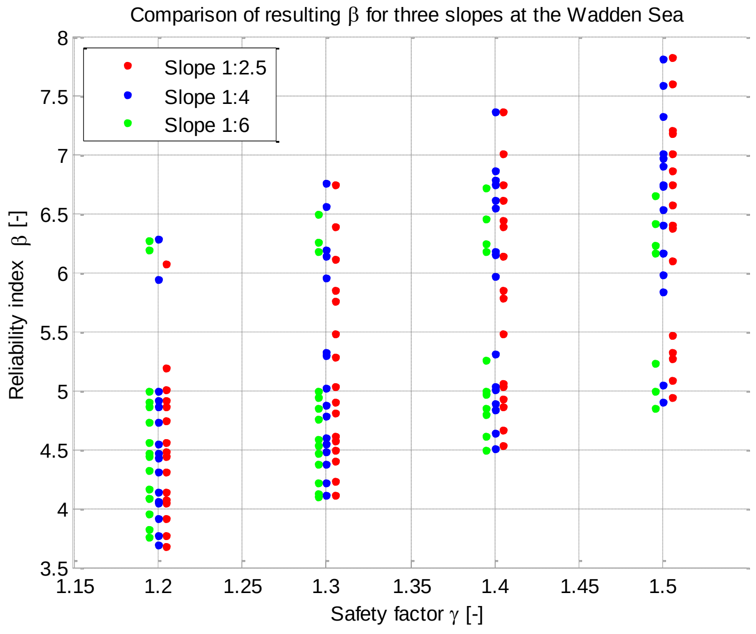

Figure 4.9 Resulting reliability indices for different slope angles in the calibration for a test case at the Wadden Sea. Not all cases have the same number of dots in the figure, mainly due to convergence issues for very shallow slopes.

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[1] *Please note: the α-values mentioned here are not influence coefficients from a probabilistic calculation but parameters in the resistance model*

4.4.4.2 *Sensitivity analysis wave steepness*

An important parameter for the cumulative load is the wave steepness (k), as this determines the number of waves in a storm (if the duration is fixed). A sensitivity analysis was carried out with wave steepnesses of k = 0.04 (as used in the calibration), k = 0.05 and k = 0.06[2]. Figure 4.10 shows the results for a case at the Wadden Sea. From this figure it appears that there is no notable influence of the wave steepness to the calibration result.

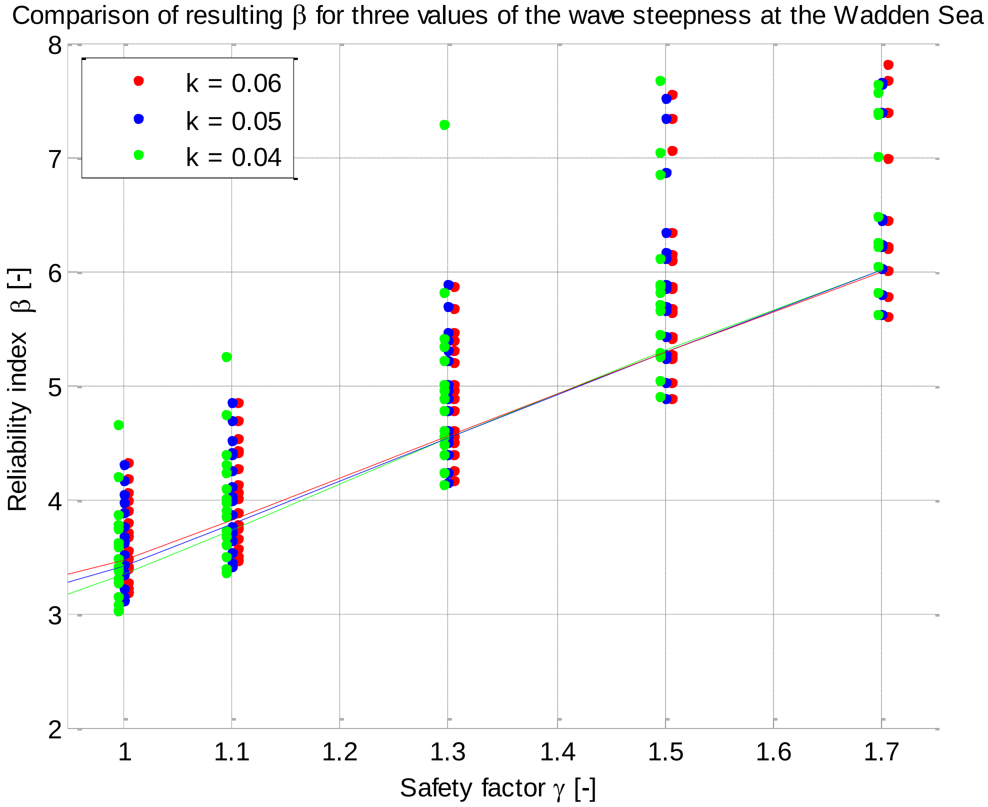

Figure 4.10 Results for a calibration calculation for the Wadden Sea with safety standard 1/10000 for different values of the wave steepness. It can be observed that the effect on the resulting β-γ relations is negligible. The green line corresponds to the value used in the calibration.

4.4.5 Summary of the assessment rule

This section outlines the different steps to be taken when doing a semi-probabilistic assessment of wave run-up on a grass revetment of a dike section in a dike segment with overall probability of flooding P_{max}.

1. Determine the geometry and relevant input parameters for the run-up model
2. As input value for the critical run-up velocity U_c use: $\frac{U_{c,5\%}}{\gamma_\beta}$ with γ_β = 1 (i.e. the 5%-quantile value divided by the beta-dependant partial safety factor). If the standard failure probability budget is not used, calculate γ_β using one of the following formulas:

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[2] *The wave steepness k [-] is calculated using the following formula:* $k = \frac{H_s}{L}$ *,where H_s is the significant wave height and L is the wave length of such a wave, both in meters.*

Water system	β_T-dependent safety factor
Western Scheldt	$\gamma_\beta = 0.191\left(\beta_{T,cs} + 2.894\right) - 0.124\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
Wadden Sea	$\gamma_\beta = 0.175\left(\beta_{T,cs} + 3.095\right) - 0.097\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
IJssel lake	$\gamma_\beta = 0.236\left(\beta_{T,cs} + 1.931\right) - 0.165\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$

3. Use the above values as input for the run-up calculation with different boundary condition combinations (see Chapter6) with exceedance probability equal to P_T, to be calculated by Ringtoets. For other strength parameters, mean values are used.

4. The revetment complies to the safety standard if the resulting D < 7000 m^2/s^2

4.5 Step 6: Comparison with current practice and analysis of consequences

4.5.1 Comparison with VTV2006

In the VTV2006, erosion of the grass revetments in the wave run-up zone is assessed using the grass erosion model derived from CIRIA curves. The CIRIA curves were developed for the design of grass waterways and based on tests where a young grass revetment (1 or 2 growing seasons old) was subjected to a stationary flow. In writing the VTV2006 it was recognized that both the erosive load in the wave run-up zone and the strength of a fully developed grass sod are quite different from the performed stationary tests on young grass.

The grass erosion model in WBI2017 is the cumulative overload model, which is based on wave overtopping tests on fully developed grass sods on actual primary water defences in the winter season. Validation of the model for wave run-up conditions was not entirely successful because not all of the wave run-up tests led to failure of the grass sod. However the model is believed to be an improvement with regard to the CIRIA based model in the VTV2006 and it is in line with the current model for wave overtopping conditions.

The first difference between the VTV2006 and the WBI2017 assessment model is the grass quality description. In the VTV2006 the classifications were ‘good’, ‘moderate’ and ‘bad’, while in the WBI2017 these are ‘closed’, ‘open’ of ‘fragmented’. The latter having no erosion resistance at all.
The sod description is now aimed at the presence of open spots in the root system, while previously the root density in depth, was considered more important than the presence of open spots.

The second difference is the possibility to incorporate the effect of transitions and objects in the assessment. The VTV2006 did not take into account the effect of the transition from the hard revetment in the wave impact zone to the grass revetment higher up the slope, the new model offers that possibility.

The third difference is in the load. Where the VTV2006 model calculates a constant design value of a flow velocity in the run-up zone, the WBI2017 takes the maximum front velocity calculated from the run-up height for each wave in a storm condition.

4.5.2 Implications of the proposed safety format

Previously the safety format was formulated with boundary conditions with exceedance probability P_{max} and a safety factor of 1.2. However, to achieve a better consistency with the assessment for wave impact this was changed to the safety format presented here. This raises two questions:

- Is one of the two safety formats more conservative?
- Which safety format is best applicable to the cases to be assessed?

4.5.2.1 Is one of the two safety formats more conservative?

This can be analyzed by deriving assessment results for both safety formats. This is done using the cases from the Design Instruments, as boundary conditions for actual cases are available (a map of the locations is given in Figure 4.14). The slope is assumed to be 1:4 in all cases and the transition is located 1 cm above the boundary between wave impact and run-up zone. It has to be noted that only run-up is assessed. From this figure it is shown that there are no big differences between the results for both safety formats. In absolute terms the difference between the safety formats can be considerable (e.g. for VL2) but as the cumulative overload term is quadratic this is to be expected.

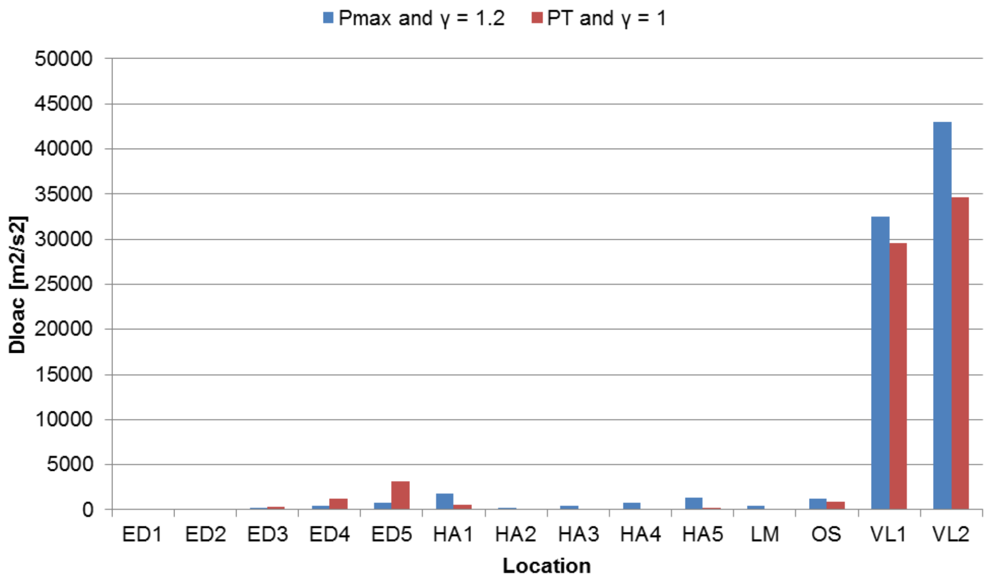

Figure 4.11 Resulting values for the cumulative overload for two safety formats for 14 cases. A value of D_{load}>7000 m^2/s^2 leads to disapproval.

From the figure it can be concluded that neither of the safety formats is clearly more conservative than the other. There can be some differences in the assessment, which is caused by the difference between the water levels during the storm and the level of the transition that is assessed, but the overall picture is that for a realistic level of the transition the results of both safety formats are fairly consistent. However, it has to be noted that for transitions higher up the slope this might change. This is covered in the next section.

4.5.2.2 Which format is most applicable to the cases to be assessed?

In the overall safety assessment, run-up will only be assessed for transitions above the level $F_h^{-1}(1-P_T)$, as below that level wave impact is assumed to be dominant. Therefore, the most important requirement is that the safety format is accurate for transitions from $F_h^{-1}(1-P_T)$ up to

2 or 3 meters above that level. An important aspect in the run-up model is that the run-up height of waves is limited and the damage to the revetment decreases for levels higher up the slope. This is illustrated in Figure 4.12, where the cumulative overload is shown for transitions at a range of levels for the case ED3 (Eemshaven-Delfzijl). Here it can be seen that when assessing with a boundary condition with a lower water level the peak of the cumulative overload is also at a lower level (below the level $F_h^{-1}(1\text{-}P_T)$). However, as the assessment is carried out for transitions above this level the proposed safety format (red line) is more accurate, as this better represents the loads in the relevant range.

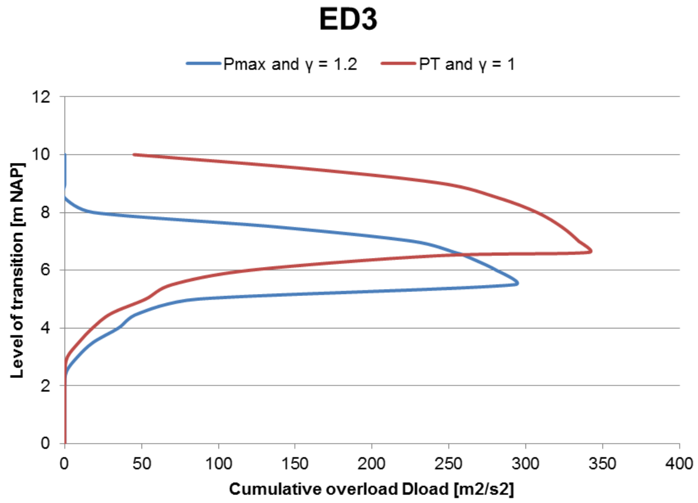

Figure 4.12 Relation between cumulative overload and level of the transition for the two compared safety formats.

It is also important to compare the analysis with a probabilistic calculation. From the Design Instruments it is known that for the cases VL1 and VL2 (Vlissingen) the required level of the transition is 9.3 and 10 m + NAP respectively (Klein Breteler et al. 2016). Here it can be seen that the required level is exactly reproduced for the case VL1 and that the result for VL2 is very close. The other safety format, with boundary conditions with exceedance probability P_{max} is much less accurate. Thus it can be concluded that the proposed safety format is more adequate for the cases to be assessed.

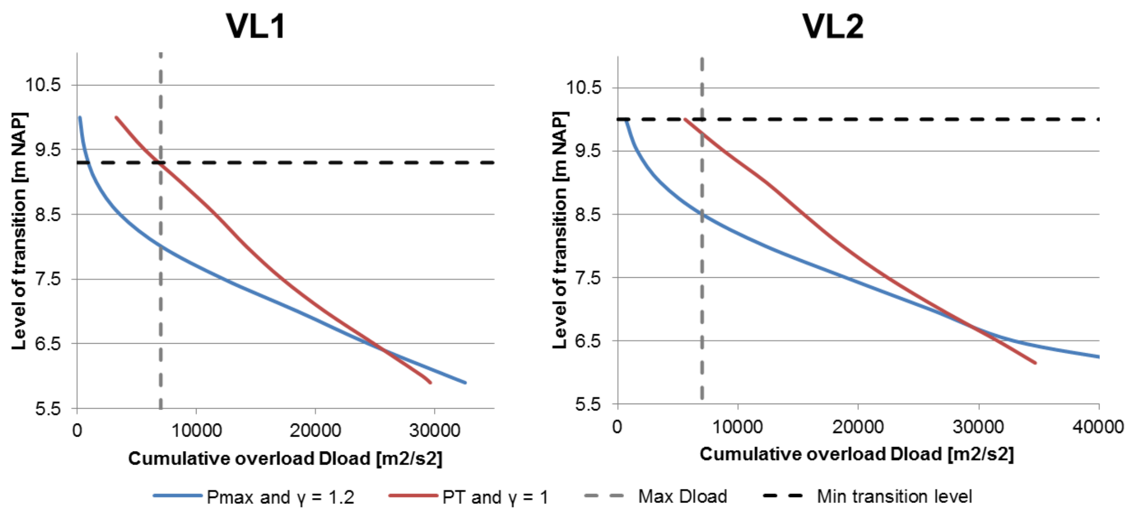

Figure 4.13 Relation between transition level, cumulative overload and requirements for D_{load} and the study for the Design Instruments (Klein Breteler et al. 2016)

4.6 Relation to Design Instruments (OI) for revetments

4.6.1 Introduction

Klein Breteler et al. (2016) derived a design rule for the required level of the transition from a hard to a grass revetment. Although the findings in the report are very similar to the ones in this report, Klein Breteler et al. (2016) have used different boundary conditions compared to our analysis. In order to show that the boundary conditions are mutually consistent, a calibration study is carried out for the locations studied in Klein Breteler et al. (2016). Five locations were considered in the design study:

- Eemshaven - Delfzijl (ED)
- Lauwersmeerdijk (LM)
- Harlingen (HA)
- Ossenisse (OS)
- Vlissingen (VL)

This choice is based upon the fact that these locations are representative for the range of possible boundary conditions along the Dutch coast. Locations along the lakes were not considered, the locations are shown in Figure 4.14.

Figure 4.14 Overview of locations (Klein Breteler et al. 2016)

4.6.2 Calibration for safety factors for 5 locations

If a calibration for these 5 locations yields the same safety factors as in the previous sections, this proves that both studies are consistent and that the safety factors are applicable to all coastal locations. Therefore we carried out the same calibration procedure as described in previous sections, except that now the boundary conditions from Klein Breteler et al. (2016) were used. The boundary conditions were derived using Hydra-K version 3.6.5, where also different correlations between wave height and water levels can be taken into account. With the models used for the calibration in the previous section this is not possible. Figure 4.15 shows an example of the relation between water level and wind speed (u_{10}) for an exceedence probability of 1/10,000 years for location Harlingen. It can be seen that for this exceedence probability, both water level and wind speed (and thus wave height) vary.

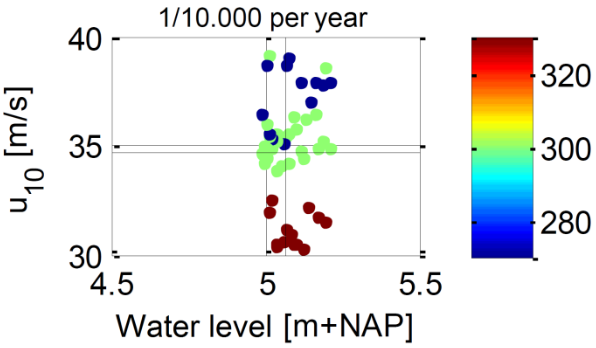

Figure 4.15 Relation between wind speed at 10 meters (u_{10}) and water level for location Harlingen (HA). The colors of the dots indicate the wind direction (Klein Breteler et al. 2016).

Using these boundary conditions the calibration procedure was carried out, Figure 4.16 shows resulting safety factors for the 5 locations with a safety standard of 1/1000 years.

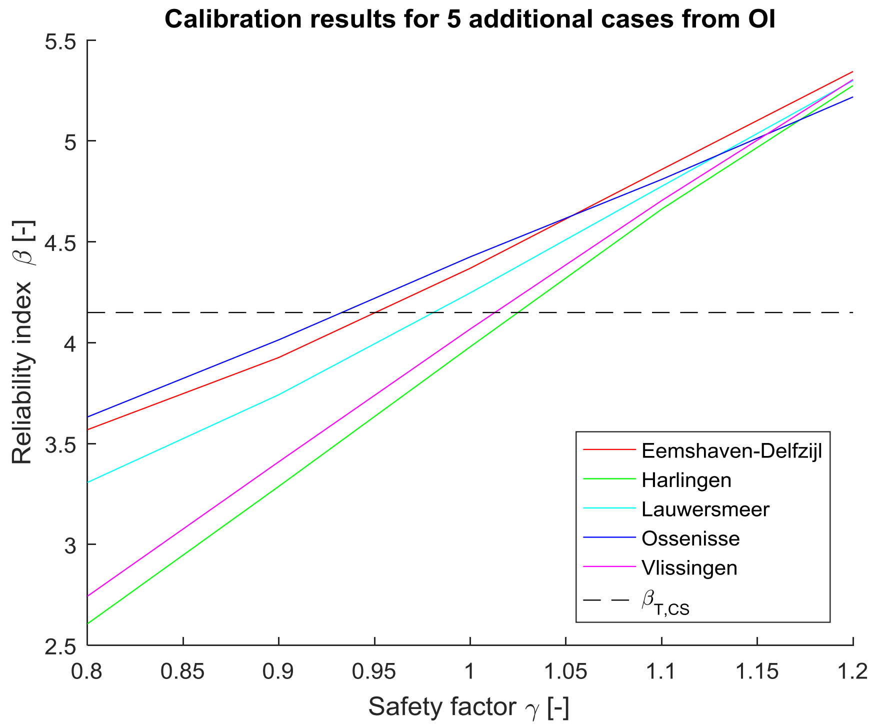

Figure 4.16 Resulting β-γ relations for the 5 locations from the OI, $\beta_{T,cs}$ is the cross-sectional reliability requirement.

From this figure it can be seen that safety factors derived with the alternative boundary conditions of *Klein Breteler et al.* (2016) are between 0.93 and 1.02 for all locations. As this is the same range as found in the previous sections this shows that the proposed safety factor of 1 is a robust estimate. It has to be noted that no locations in non-tidal areas were considered here.

5 Semi-probabilistic assessment of wave impact on grass revetments

This chapter considers the derivation of a semi-probabilistic assessment rule for wave impact on grass revetments. Due to the very low resistance of grass when subject to wave impact loads, as outlined in section 3.2, a different approach is taken compared to the analysis for run-up failures. In this chapter distinction is made between assessing grass revetments in coastal areas and around lakes (section 5.1), and for riverine areas (section 5.2.) In both sections first some general considerations are given, after which a set of cases is analysed probabilistically. Afterwards, the different options for carrying out the assessment are given. Section 5.3 considers the representative values in the assessment, and section 5.4 presents the resulting assessment rule.

5.1 Assessing grass impact at outer slopes in coastal and lake areas

The first case to be considered is a grass revetment in a coastal area. In coastal areas there is, generally, a strong correlation between wave height and water level, meaning that high water levels are accompanied by high waves. Figure 5.1 shows the relations between exceedance probability of the water level and significant wave heights for the three water systems used for the calibration of safety factors for block and asphalt revetments, as well as for the calibration of safety factors for run-up on grass revetments. These are typical for the different systems, although there is, of course, local variability in wave conditions. Based on these relations and the resistance-duration curves shown in section 3.2, it is highly unlikely that a grass revetment in a coastal area will ever be approved for wave impact. As wave heights are generally higher than any wave a grass revetment can withstand, assuming full correlation between wave height and water level (meaning that loads increase further up the slope) means that the influence coefficient of the load (α_S) will be very close to 1. This means that the exceedance probability of the load in the design point will be approximately equal to the failure probability of the revetment (i.e. it fails at the lowest point where grass is located).

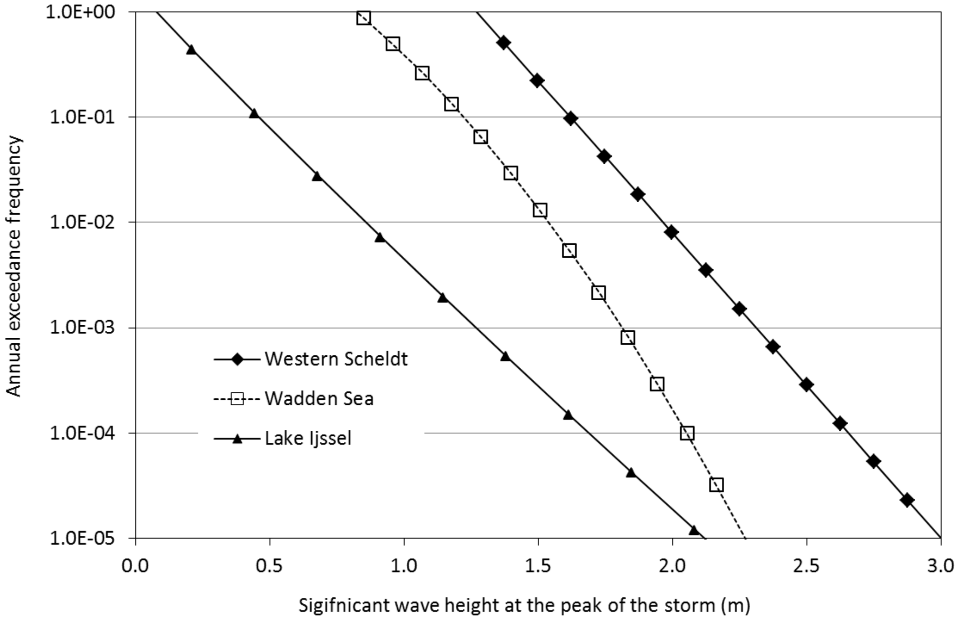

Figure 5.1 Relation between exceedance frequency of water levels and significant wave height at the peak of a storm for three water systems (Kaste & Klein Breteler 2012).

In order to determine whether a semi-probabilistic assessment in coastal areas is at all relevant, and to determine what this assessment should look like, a number of probabilistic analyses has been carried out. For this, the four cases shown in Figure 5.2 are considered:

- Case 1: grass up to the level $F_H^{-1}(1\text{-}\ P_{max})$, in accordance with the initially proposed WBI2017 assessment rule. The part of the revetment above $F_H^{-1}(1\text{-}\ P_{max})$ is not considered.
- Case 2: Slope covered entirely with grass.
- Case 3: grass only present above $F_H^{-1}(1\text{-}\ P_{max})$+ 5 cm.
- Case 4: grass only present above $F_H^{-1}(1\text{-}\ P_T)$.

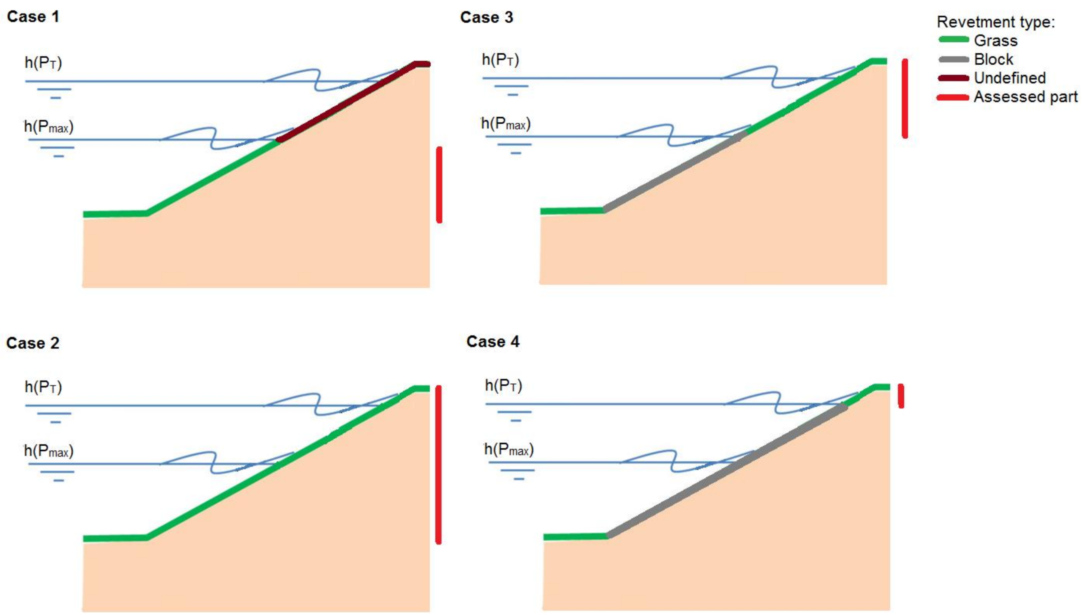

Figure 5.2 Four cases for coastal grass revetments

5.1.1 Probabilistic analysis of a grass revetment at Lake IJssel

The analysis of the four cases is carried out for a fictitious dike profile at Lake IJssel. Table 5.1 shows the failure probabilities for the different cases, where it is assumed that P_T = P_{max}*0.05. The lower boundary of the assessment is assumed at a level $F_H^{-1}(1$ - 1/10 per year) = 0.60 m NAP. The top boundary of the revetment is assumed at a level of 3.5 m +NAP. For boundary conditions the load model for Lake IJssel as used in the calibration for block and asphalt revetments is used. Note that this is the most favourable of the three water systems mentioned, as wave heights are significantly lower than for the Wadden Sea and Western Scheldt. However, Figure 5.1 gives a wave height of 1.3 m for a level with an exceedance probability equal to a P_{max} of 1/1000 per year. This means that, considering the commonly used values for the resistance-duration curves, it is highly unlikely that any revetment at Lake IJssel will be approved. For the resistance-duration curve, the parameters as given in Appendix H are used. The four cases are evaluated using Monte Carlo with Importance Sampling, convergence plots for the calculations are given in Appendix E.

Table 5.1 Probabilistic evaluation of 4 cases for a grass revetment at Lake IJssel

		Case 1 Assessment of grass below $h(P_{max})$	**Case 2** Grass on entire revetment	**Case 3** Grass above $h(P_{max})$ + 5 cm	**Case 4** Grass above $h(P_T)$
Strength parameters	Constant in resistance-duration curve (a) *[m]*	Lognormal distribution: L(1.82,0.62)			
	Constant in resistance-duration curve (b) *[1/hr]*	Constant: -0.035			
	Constant in resistance-duration curve (c) *[m]*	Constant: 0.25			
	Sand content clay layer (F_{sand}) *[-]*	Lognormal distribution: L(0.35,0.07)			
	Storm duration (t_{storm}) *[in hours]*	35			
Assessment parameters	P_{max} *[-/year]*	1/1000			
	P_T *[-/year]* (β_T) *[-]*	1/20000 (3.89)			
	Minimum level of grass *[in m NAP]*	0.6024	0.6024	1.3515	1.7715
	Maximum level of grass *[in m NAP]*	1.3015	3.5	3.5	3.5
Results	Failure probability P_f	**$2.3 \cdot 10^{-3}$**	**$2.3 \cdot 10^{-3}$**	**$2.9 \cdot 10^{-4}$**	**$3.7 \cdot 10^{-5}$**
	Reliability index β	2.83	2.83	3.44	3.96
	Influence coefficient of the load α_s	0.940	0.940	0.975	0.996

In Case 1 the grass revetment is evaluated up to the assessment water level. This is accordance to the previously proposed WBI2017 assessment procedure with a strict distinction between impact and run-up zone. This results in the same failure probability as when assessing the entire revetment (Case 2), as the failure probability is dominated by failures at lower parts of the slope, due to the fact that the influence coefficient of the load is very large, and the strength is negligible. Thus, failures higher up the slope have a minor influence on the failure probability, due to the large influence coefficient of the load. This makes sense as, with a very high influence coefficient of the load, the exceedance probability of the load in the design point will be very close to the failure probability. Thus a failure with boundary conditions with an exceedance probability of 10^{-4} is about 10 times less likely than a case with boundary conditions with exceedance probability 10^{-3} per year[3].

3 *Considering the values from Case 1: α_s is 0.94. Then $\alpha_s * \beta$=2.66 which corresponds to a probability of exceedance of the parameter value for the load in the design point of $3.9 \cdot 10^{-3}$, which is close to the $2.3 \cdot 10^{-3}$ found for the failure*

For Case 3, where it is assumed that the grass revetment is only located above $F_H^{-1}(1- P_{max})$, the failure probability is still well above P_T. Only when assessing a revetment located above $F_H^{-1}(1- P_T)$, as is done in Case 4, the failure probability is in the right order of magnitude compared to the requirement. Thus if we would design this revetment, there would be no grass revetment allowed at levels below $F_H^{-1}(1- P_T)$. From these cases we can draw the following conclusions:

- It is highly unlikely that any grass revetment at a wind-dominated location in coastal and lake areas will be approved below a level almost equal to $F_H^{-1}(1- P_T)$. The cases considered used the Lake IJssel load model, which is still relatively favourable in terms of load conditions. Yet, when assessing grass revetments below $F_H^{-1}(1- P_{max})$ and $F_H^{-1}(1- P_T)$ failure probabilities are much higher than the requirement. A possible exception are locations with orientation towards the east. These might behave more like cases in riverine areas and are covered by that part of the analysis.
- In order to prevent false positives in the assessment, boundary conditions with exceedance probability $F^{-1}(1- P_T)$ have to be used. For instance in Case 3, where the grass revetment starts above $F_H^{-1}(1- P_{max})$, in the initially proposed assessment we would not assess it for wave impact as the grass revetment starts above $F_H^{-1}(1- P_{max})$. This would result in a false positive, because the actual failure probability is about 30 times the requirement. It is not possible to prevent these false positives by using a safety factor, as they arise from the definition of the run-up and impact zone so it has to be solved by using boundary conditions with exceedance probabilities equal to the cross-sectional reliability requirement. This results in a safety format that is in line with the calculated influence coefficients.

From a physical point of view the findings in this section are not strange: If a revetment would be designed probabilistically with the failure mechanism model and the inputs used here, it would be very unlikely that a designer would choose to use grass in areas where wave impacts are likely to occur (i.e. with a probability higher than the required P_T). This is also shown in the report by Klein Breteler et al. (2016) on design of revetments. In this study wave impact and wave run-up were considered together in order to derive a design rule for the level from which a grass revetment is allowed. Figure 5.3 shows the result for the failure probability for wave impact along the slope (green line), as well as the exceedence probability of the water levels corresponding to it (dashed line). It can be seen that the lines are generally on top of each other, meaning that the influence coefficient of the load in the design point is very close to 1. This also follows from the other cases studied in this report.

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*probability. For other cases, such as Case 4 this value is 0.996*3.96=3.94 corresponding to a P_{exc} of $4*10^{-5}$, while P_f is $3.7*10^{-5}$. Thus for this type of failures the exceedance probability of the load is very close to the failure probability.*

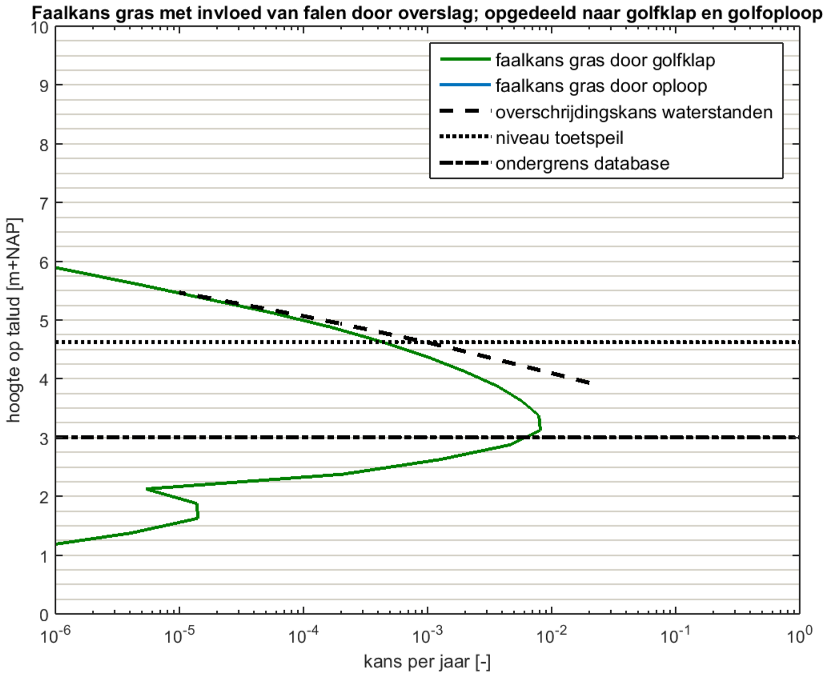

Figure 5.3 Probability of failure of grass revetments at different heights along the outer slope for wave impact failures for the case HA2 (Klein Breteler et al. 2016)

However, Klein Breteler et al. (2016) also concludes that not all locations at the coast and especially the lakes behave like this, as there are also locations at Lake IJssel that are lake level dominated. An example is given in Figure 5.4, for location Edam. This is a lake level dominated location, meaning that there is less correlation between water level and wind speed. Therefore, the distance between the black dotted and green line is significantly larger, as wave heights do not increase with water levels as rapidly as for locations with strong correlation. It has to be noted that, as the study of Klein Breteler et al. (2016) is a study on design rules, they have used load durations that are quite long and that are not valid for the safety assessment. With a shorter load duration the difference between the two lines will be larger than the difference of Figure 5.4.

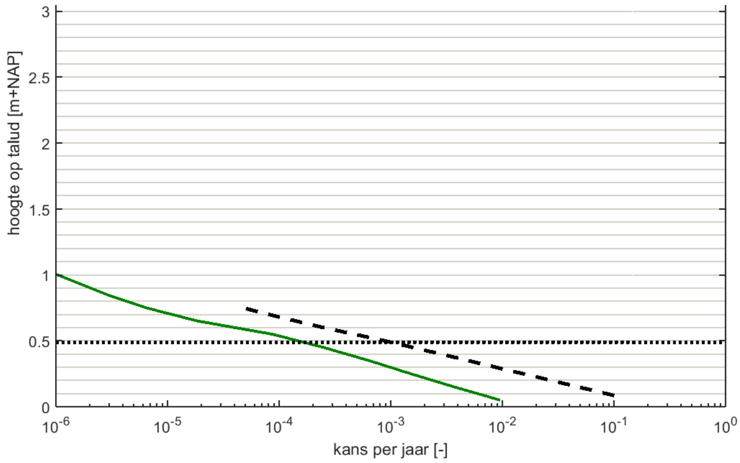

Figure 5.4 Probability of failure of grass revetments at different heights along the outer slope for wave impact failures for the case Edam1 (Klein Breteler et al. 2016)

This result indicates that the assessment for locations with little or no correlation between (high) water levels and waves can significantly differ from locations with strong correlation. Since the latter group has already been discussed in previous sections, the following section describes cases without correlation between (high) water levels and waves, specifically in the riverine area.

5.2 Assessment of grass impact in riverine areas

5.2.1 Characteristics of grass revetments in riverine areas

The assessment of grass revetments in riverine areas differs from those in coastal areas. The main reason is because the type of boundary conditions is completely different:

- Small or no correlation between water level and wave height.
- Much smaller waves, generally H_s < 1 meter.

Due to the fact that waves are much smaller, assessing grass revetments for wave impact failures in riverine areas makes sense, as, at most locations, grass revetments will be strong enough to withstand the ‘highest possible’ waves. The distinction between impact and run-up zone as defined for other areas is a bit confusing for riverine areas. Due to the low correlation between water level and wave height, the probability of high waves hitting the slope at high levels is very small. Therefore, multiple combinations of waves and water levels need to be considered in the assessment. This is schematically illustrated in Figure 5.5. Figure 5.6 show san example of combinations of water level and wave height with the same exceedence probability as calculated by the revetment load module in Hydra-Ring for location Lobith. This shows that wave heights decrease with increasing water levels.

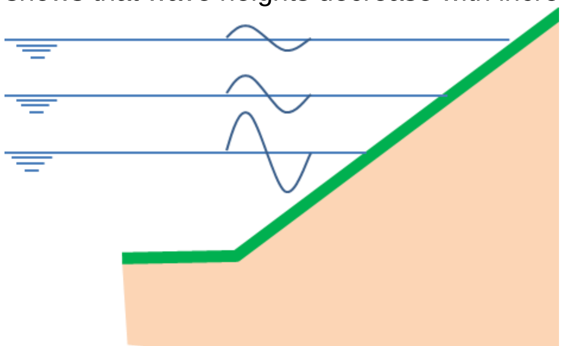

Figure 5.5 Assessment for riverine areas using the load module for revetments and assuming no correlation between water level and waves. The figure shows combinations of water levels and wave heights with the same joint probability density of exceedence. Higher up the slope, wave heights decrease as the water level consumes a larger part of the total probability of the load combination.

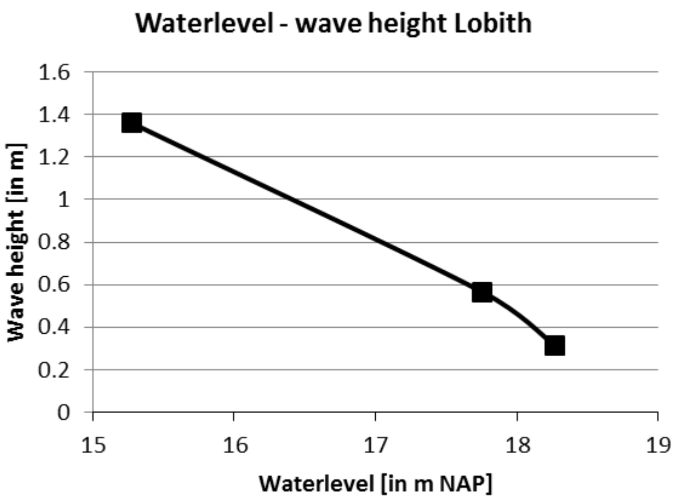

Figure 5.6 Relation between water level and wave height for Lobith at the Rhine. Values were derived using the Q-variant in Hydra-Ring.

5.2.2 Analysis of cases in the upper and lower Rhine River

5.2.2.1 Approach and selection of cases

In order to further investigate possible assessment formats for the riverine areas, a selection of cases has been analysed. The approach is as follows:

- For a selection of cases derive hydraulic boundary conditions using Hydra-Ring
  - Marginal distribution of wave height and water level for probabilistic calculation
  - Loads for revetments using Q-variant
- Carry out a probabilistic calculation using derived marginal distributions using the model set-up for the calibration
- Carry out a semi-probabilistic calculation for different combinations of water level and wave height using the derived loads for revetments and the 5% load-duration curve

By comparing the results for probabilistic and semi-probabilistic calculations the safety format can be evaluated. For the analysis the locations in Table 5.2 have been selected. A more thorough description of the selection of locations can be found in Appendix I.

Table 5.2 Selection of riverine locations

Location	Dike track	HCLDid	Deelsysteem	Safety standard	Max fetch Hydra-Zoet
Lobith	48-1	100563	Upper Rhine	1/30000	3860
Wageningen	45-1	105454	Upper Rhine	1/100000	2325
Doesburg	49-2	102604	Upper Rhine	1/1000	4114
Zutphen	50-2	102842	Upper Rhine	1/3000	1219
Tiel	43-6	107822	Upper Rhine	1/30000	3218
Cabauw	15-1	306305	Tidal Rhine	1/30000	993
Gorinchem	16-1	310211	Tidal Rhine	1/100000	2103
Moerdijk	34-2	304251	Tidal Rhine	1/1000	5889

5.2.2.2 Schematization of loads

Hydra-Ring offers 2 types of boundary conditions containing wave heights: the loads for revetments, and marginal distributions for wave height and water level. The first only gives design points (in Dutch: illustratiepunten) for combinations of loads, which is an insufficient representation of the entire distribution wave - water level combinations for a probabilistic calculation. Therefore in this case, as the assumption is that for the selected locations the loads can be represented by combining the marginal distributions for wave height and water level. However, for an uncorrelated case these cannot be simply multiplied as these are both annual probabilities and the probability that both occur at the same time has to be accounted for. Therefore the marginal distributions are corrected using a base duration for the water level of 4 days and for the wave height of 6 or 12 hours, as shown in Figure 5.7. This is done using the following formula:

$$P(h > \bar{h} \cap H_s > \bar{H}_s \cap together) = \mathrm{p}(h > \bar{h}) * \mathrm{p}(H_s > \bar{H}_s) * \frac{(t_h / t_{storm})}{365/2}$$

where h is the water level, H_s the wave height, t_h the duration of the high water level (i.e. 4 days), t_{storm} the storm duration (i.e. 6 or 12 hours). Additionally a factor 365/2 is used to correct for the fact that statistics of extremes are generally based on data during the winter half year. It is assumed that water level and waves are independent and that exceedence probabilities of $\bar{h}$ and $\bar{H}_s$ are smaller than 1/year. The latter is a reasonable assumption if the probability of two or more exceedences in a year is small.

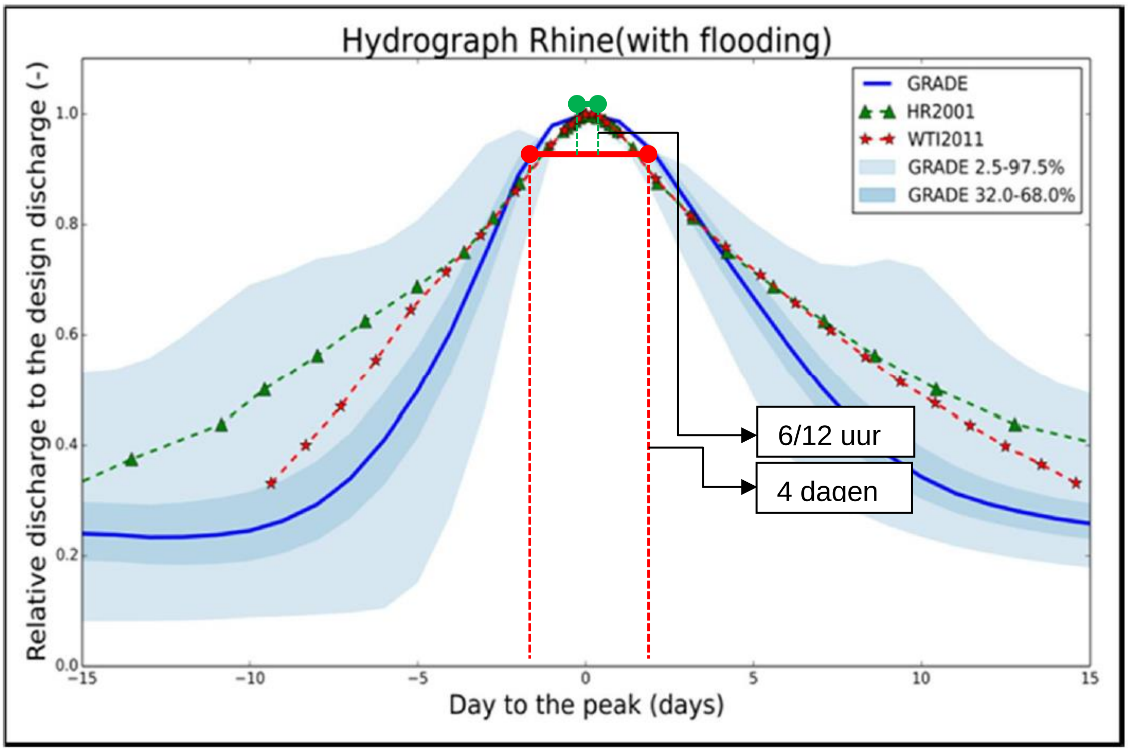

Figure 5.7 Sketch of the schematized hydrograph and storm duration for the calculations

5.2.2.3 Results

All locations have been evaluated using Monte Carlo with Importance Sampling with 20,000 samples. These were evaluated for two cases: an infinitely high revetment (A) (i.e. no lower boundary), and a revetment starting at a level corresponding to the 1/10 yr^{-1} water level, corresponding to the lower boundary of the wave impact zone. For locations in the Tidal

Rhine, cases with a correlation coefficient between water level and wave height of 0 and 1 have been evaluated, for locations in the Upper Rhine only a correlation coefficient of 0 has been considered.

For the semi-probabilistic analysis the loads derived with the Q-variant were used. These were derived for a probability corresponding to the cross-sectional reliability requirement and water levels from the marginal distribution for the following 3 exceedence probabilities:

- 1/10 yr^{-1}
- safety standard
- cross-sectional reliability requirement

Table 5.3 shows the results for a storm duration of 6 hours, Table 5.4 shows the results for a storm duration of 12 hours. From left to right it shows the following main headers: Location, Cross sectional reliability requirement (P_T), Correlation (0 or 1) and the results for the semi-probabilistic and probabilistic assessment. The latter two categories have subcolumns. For the semi-probabilistic assessment these indicate the exceedence probability of the water level in the load combination, for the probabilistic assessment the two columns correspond to the two considered revetments cases (infinite slope, Case A, or lower boundary at 1/10 yr^{-1} level, Case B). The colors distinguish cases that are approved (green), slightly disapproved (orange, $P_f<10*P_T$) and disapproved (red, $P_f>10*P_T$).
The results show that a semi-probabilistic assessment with a combination of 1/10 yr^{-1} water level and corresponding wave leads provides, for nearly all locations, a consistent result with the fully probabilistic assessment. As local circumstances might vary due to irregularities (e.g. due to foreshores or gullies) it is advised to consider different combinations of loads and water levels along the entire revetment. Assessments with higher water levels result in a reduction of the wave height, which makes the semi-probabilistic assessment too optimistic. Thus it can be concluded that, in order to obtain a reliable safety format for riverine locations, especially load combinations with lower water levels will have to be considered. .

Table 5.3 Results for a storm duration of 6 hours

Location	P_T	Correlation	Semi-probabilistic assessment			Probabilistic assessment	
			1/10	P_{norm}	P_T	*Case A*	*Case B*
Lobith	1.52E-06	0	NV	V	V	7.07E-05	3.67E-05
Wageningen	4.55E-07	0	V	V	V	5.33E-09	3.21E-09
Doesburg	4.55E-05	0	V	V	V	7.94E-05	5.33E-05
Zutphen	4.55E-06	0	V	V	V	3.70E-07	3.57E-07
Tiel	1.52E-06	0	NV	V	V	4.66E-06	3.07E-06
Terwolde	1.52E-05	0	V	V	V	1.66E-08	9.72E-09
Cabauw	1.52E-06	0	NV	V	V	6.45E-06	8.15E-06
	1.52E-06	1	NV	V	V	1.02E-04	8.27E-05
Gorinchem	4.55E-07	0	NV	V	V	1.22E-07	8.52E-08
	4.55E-07	1	NV	V	V	1.68E-06	1.07E-06
Moerdijk	2.27E-05	0	nb	nb	V	4.30E-08	4.63E-10
	2.27E-05	1	nb	nb	V	7.72E-07	9.73E-7

Table 5.4 Results for a storm duration of 12 hours

Location	P_T	Correlation	Semi-probabilistic assessment			Probabilistic assessment	
			1/10	P_{norm}	P_T	*Case A*	*Case B*
Lobith	1.52E-06	0	NV	V	V	1.15E-04	6.68E-05
Wageningen	4.55E-07	0	V	V	V	3.06E-08	9.29E-09
Doesburg	4.55E-05	0	NV	V	V	1.41E-04	9.70E-05
Zutphen	4.55E-06	0	V	V	V	1.96E-06	1.82E-06
Tiel	1.52E-06	0	NV	V	V	1.55E-05	6.82E-06
Terwolde	1.52E-05	0	V	V	V	5.49E-08	4.14E-08
Cabauw	1.52E-06	0	NV	V	V	2.27E-05	1.20E-05
	1.52E-06	1	NV	V	V	6.07E-04	3.73E-04
Gorinchem	4.55E-07	0	NV	V	V	2.51E-07	2.40E-07
	4.55E-07	1	NV	V	V	6.77E-06	6.03E-06
Moerdijk	2.27E-05	0	V	V	V	2.35E-07	2.02E-09
	2.27E-05	1	V	V	V	7.58E-06	4.85E-06

5.2.3 Conclusions on assessment of grass revetments in riverine areas

From the analysis of several cases in the riverine areas, it can be concluded that for a proper assessment of wave impact on grass revetments it is pivotal to do the assessment using various combinations of water levels and waves. Although in the studied cases in every occasion the combination with the lowest water level is most relevant, it is advised to carry out the assessment for a range of water levels as local circumstances (e.g. foreshores, gullies) might cause other levels to give the highest load.

In our analysis the peak storm duration was assumed to be equal to 6 or 12 hours. It is advised to derive a standard duration for the different water systems in the Netherlands where wind is not the dominant factor for high water levels (i.e. al river dominated water systems and the western part of the Lake area). For both upper and tidal Rhine, 12 hours is expected to be a good representative duration.

5.3 Choice of representative values

5.3.1 Representative values for the strength

For the assessment there are various possibilities for representative values in the semi-probabilistic assessment. From the cases in coastal areas it appears that the influence of strength uncertainty on failure probabilities is very small as influence coefficients of the loads are very large. For such areas assessing with a median (50%) resistance duration curve would be adequate. For riverine areas, the strength uncertainty plays a more significant role. Influence coefficients for strength parameter a are typically in the order of 0.3 to 0.5. Combining with the reliability indices this would result in 1 to 5% quantile values in the design point. Hence, a 5% quantile is chosen for the resistance duration curve. From the cases studied it was shown that this gives an adequately safe but not overly stringent assessment. It is advised to also use this value for coastal areas for the following reasons:

- for sheltered locations that might also be present along the coast and along lakes, a 5%-value is more adequate as these locations have more resemblance to the studied riverine cases

- for wind-dominated locations the choice between a 50 or 5%-value will rarely make a difference. Due to the height of the waves, grass revetments in the wave impact zone will fail regardless of the selected duration curve.

Thus for all areas the 5% resistance-duration curve can be used. For the sand fraction the 50%-value can be used as influence coefficients for this variable are very small in all cases (α≈-0.02).

5.3.2 Representative value for the load

For assessments at locations with (almost) full correlation between wind and water level in principle it is sufficient to carry out the assessment with a single combination of water level and waves: the combination with water level equal to the cross-sectional failure probability and corresponding wave conditions. However in practice the relation between wave height and water level can vary due to local differences (e.g. foreshores, gulleys). Therefore it is advised to assess the revetment for different combinations of water level and wave conditions for all locations. This is especially necessary for locations where there is no (near) full correlation between water level and waves as was shown from the cases in the riverine areas. For these locations, a load combination with a relatively low water level is usually decisive for the assessment result. Therefore, it is proposed to assess grass revetments in accordance with the example of Figure 5.8. For such cases, various upper and lower limits for the water levels at which to assess can be selected, namely:

- Revetment bounds: the upper and lower limit of the grass revetment[4]
- Limit at 0.01 m below $F_h^{-1}(1\text{-}P_T)$: limit for the highest water level that can be used as input for calculating a load combination corresponding to a probability of $1\text{-}P_T$ using the Q-variant.
- User defined range, in order to take location specific knowledge into account.

Combined with a step size for the water level this gives a set of water levels at which the load combination for the revetment can be calculated. All three options are facilitated in the assessment software; it is advised to use all three options to make sure the revetment is thoroughly assessed. Of course especially the free range has to be used with sufficient system knowledge.

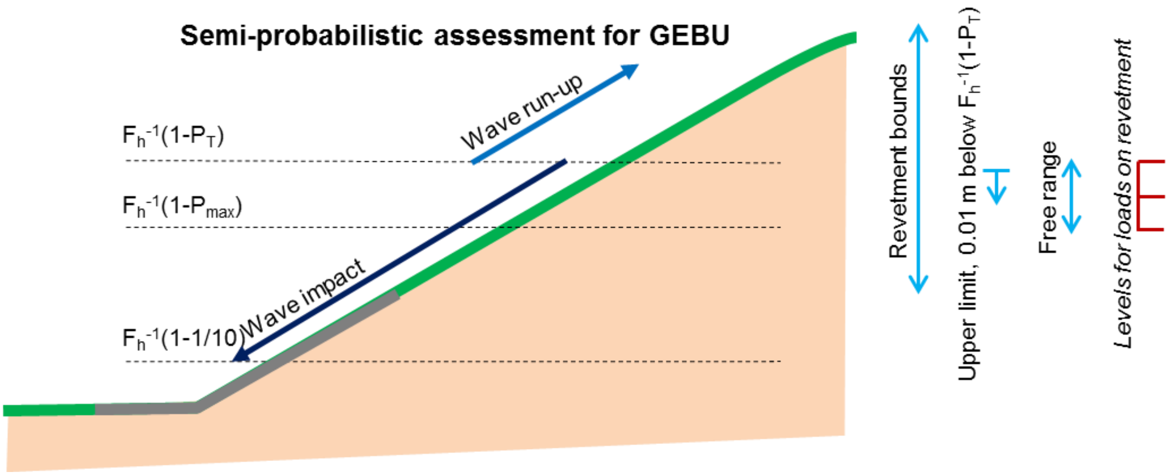

Figure 5.8 Proposed semi-probabilistic assessment including boundaries for water levels in the derived load combinations.

[4] *It has to be noted that these are only applicable for wave impact, as for wave run-up the lower bound of the revetment is above the level $F_h^{-1}(1\text{-}P_T)$. It is impossible to calculate a load combination for that level.*

It has to be noted that, similar to the safety format for wave run-up, the safety format implies a combination of α-values that is slightly more pessimistic than theoretically possible. This could be solved in two ways:

- Using a more optimistic percentile for the strength duration curve
- Using a more optimistic exceedance probability of the load

Both are not done as from an analysis for the influence coefficients of the strength it follows that a 5%-quantile is quite a good estimate for the representative value. Table 5.5 shows the results for 4 cases (no correlation between wind and water levels) in the riverine area. Here it is seen that a more optimistic percentile for the strength would not be an adequate solution given the design points from the probabilistic calculations. Based on the influence factor αa and the reliability index β the corresponding value of the standard normal distribution can be calculated. This value implies a quantile for that parameter in the design point. In the last column this quantile is given. Here it can be seen that these are in the range of the chosen 5% value.

Table 5.5 Relation between resulting influence coefficients for strength parameter a and the implied quantile.

Case	α_a	**β**	**u = α_a * β**	**Implied quantile**
Cabauw	-0.38	4.22	-1.608	0.05
Doesburg	-0.42	3.72	-1.548	0.06
Gorinchem	-0.45	4.99	-2.264	0.01
Wageningen	-0.33	5.45	-1.790	0.04

Reducing the load to be implemented is also not desirable as this could result in false positives for transitions that are close to, but just below the level $F_h^{-1}(1\text{-}P_T)$ in cases with high correlation between wind and water level.

As both solutions have clear disadvantages the proposed safety format is the closest overall approximation of the design point. Also, as was shown for run-up, generally refinements have a minor effect in terms of probabilities.

5.4 Summary of proposed semi-probabilistic assessment rule for wave impact on grass revetments

In the previous section it has been proposed to use 1 general assessment rule for wave impact for all locations. The proposed procedure is as follows for the semi-probabilistic assessment:

- Evaluate the revetment with boundary conditions corresponding to the cross-sectional target failure probability P_T. These boundary conditions are combinations of waves and water levels to be derived using the module for loads on revetments (Q-variant). These should be derived for a selection of water levels such as in shown in Figure 5.8.
- For the assessment the 5%-resistance duration curve should be used :

Table 5.6 Proposed parameters of the duration curve

Parameter	**Representative value (closed sod)**	**Representative value (open sod)**
F_{sand}	from schematization	from schematization
a	1	0.8
b	-0,035	-0.07
c	0,25	0.25
d_c	from schematization	from schematization

The representative values are described in detail in Appendix G. These are 5%-values of the resistance-duration curves based on experimental data. For the sand fraction use of the 50%-value is proposed.

6 Overview of the assessment for GEBU

In the previous chapters assessment procedures were derived for both wave run-up and wave impact. In this chapter these are combined in the form of a workflow for the semi-probabilistic assessment. It is assumed that all the requirements for applying the assessment are satisfied and that all necessary information on strength and geometry is already known.

Step 1: determine the dominant submechanism
Determine the level of $F_h^{-1}(1\text{-}P_T)$ from the marginal distribution of the water levels and determine whether the transition from hard to grass revetment is below or above this level. If the transition is below, wave impact should be assessed, if the transition is above this level, wave run-up should be assessed. See Figure 6.1.

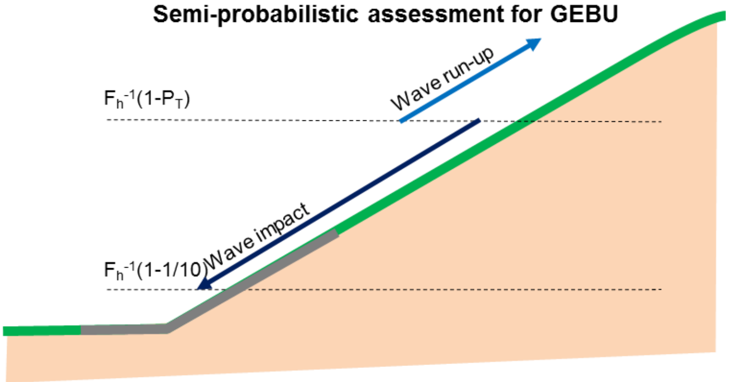

Figure 6.1 Proposed boundary between impact and run-up zone

Step 2: determine the loads to consider
For the assessment a set of load combinations will have to be considered. For wave impact this is specifically important for locations with little or no correlation between water level and wave conditions. For wave run-up this is also required as the load combination with the maximum water level doesn't necessarily result in the highest cumulative load. For this purpose, it is proposed to use 3 different ranges for water levels in the calculation(see also Figure 6.2).

- The bounds of the revetment: for grass impact these limit the load combinations to the relevant range. please note that for run-up this range cannot be used as it contradicts to the value for the upper limit.
- Upper limit, 0.01 m below $F_h^{-1}(1\text{-}P_T)$: this is a limit that is necessary when deriving boundary conditions using the Q-variant. If a higher level is used no wave height and/or water level can be calculated.
- Free range: this is a free range that can be defined by the user. For instance, in a riverine area in an assessment for wave impact only the lower part of the grass revetment could be considered. On the other hand, when assessing run-up in a wind-dominated location (e.g. with full correlation), only the part just below $F_h^{-1}(1\text{-}P_T)$ can be selected. This reduces calculation time.
- Step size: this is the step size within which the water level is increased within the defined range. The default value is 0.1 m, but this can be changed if desired.

It has to be noted that grass revetments located below the level $F_h^{-1}(1\text{-}1/10)$ should be covered in the daily maintenance and do not have to be assessed in the safety assessment. Therefore that level is always the lower limit of the safety assessment.

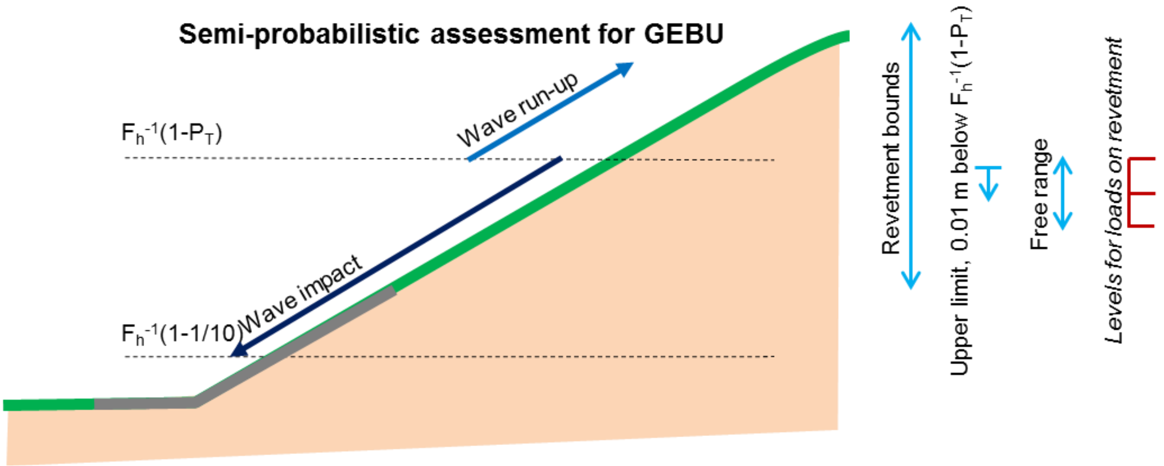

Figure 6.2 Proposed semi-probabilistic assessment including boundaries for water levels in the derived load combinations.

<u>Step 3: determine the load duration</u>
The load duration should be determined based on the water system and the values given in the Schematization Manual Grass Revetments (in Dutch: 'Schematiseringshandleiding Grasbekledingen'). The load duration is in accordance with the load durations used for HR2006 and for the assessment of block and asphalt revetments.

<u>Step 4: assess the mechanism</u>

Wave impact
If wave impact is assessed the previously derived boundary conditions and load duration will have to be entered in the assessment module for Wave Impact. For the assessment the 5%-resistance duration curve should be used:

Parameter	Representative value (closed sod)	Representative value (open sod)
F_{sand}	from schematization	from schematization
a	1	0.8
b	-0,035	-0.07
c	0,25	0.25
d_c	from schematization	from schematization

For the sand fraction the 50%-value should be used.

Wave run-up
If wave run-up is assessed with the previously derived boundary conditions and load duration, the input value for the critical run-up velocity U_c should be: $\frac{U_{c,5\%}}{\gamma_\beta}$ with γ_β = 1 (i.e. the 5%-quantile value divided by the beta-dependant partial safety factor). If the standard failure probability budget is <u>not</u> used, γ_β has to be calculated using one of the following formulas:

Water system	β_T-dependent safety factor
Western Scheldt	$\gamma_\beta = 0.191\left(\beta_{T,cs} + 2.894\right) - 0.124\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
Wadden Sea	$\gamma_\beta = 0.175\left(\beta_{T,cs} + 3.095\right) - 0.097\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
IJssel lake	$\gamma_\beta = 0.236\left(\beta_{T,cs} + 1.931\right) - 0.165\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$

If the cumulative overload D < 7000 m^2/s^2 the revetment can be approved.

7 Conclusions & Recommendations

This chapter gives conclusions and recommendations on the assessment of run-up (section 7.1), impact (section 7.2) as well as for the assessment of the GEBU-mechanism as a whole in section 7.3.

7.1 Conclusions and recommendations for the assessment of run-up

For wave run-up a β-dependent safety factor has been derived, which enables assessments with boundary conditions corresponding to an exceedance probability equal to the cross-sectional reliability requirement (P_T). For the default failure probability budget it is advised to use a safety factor of 1 for all cases, this safety factor is rather insensitive to changes in boundary conditions as well as other parameters. If a different choice is made for the failure probability budget a different safety factor has to be used for which the formulas have been provided. From a comparison for a set of cases it can be concluded that the proposed safety format is in accordance with findings from the Design Instruments (OI).

It has to be noted that there is some conservatism in the assumed distribution for strength parameter U_c. Experts assume that critical velocities for run-up are higher than for run-down, but the used distribution is based on run-down experiments. Also, the distribution of U_c is based on a regional test set, while a local test set based on field experiments would be more appropriate. Most likely this test set will have a much smaller variation, which would reduce the required safety factors. This will lead to a more precise assessment of the actual strength. This should be subject to further study. On the other hand, the values of the influence factors of transitions (α_m and α_s) are assumed to be 1, meaning that a transition has no negative influence on the strength. From practical experience and recent research this is a doubtful assumption, as transitions are clearly identified as weakest spots, thus this is likely an optimistic assumption. There is no clear model uncertainty defined, in this report an overview of possible contributors to the model uncertainty is given, which could be used as input for further study on the correctness of the model. It is advised to further look into the different assumptions for U_c, influence factors and other model uncertainties in order to derive a proper model uncertainty factor.

7.2 Conclusions and recommendations for the assessment of wave impact

Based on the results of the probabilistic calculations for wave impact it is advised to assess wave impact for the entire slope, up to a level corresponding with water levels with exceedance probability equal to the target failure probability for wave impact (P_T).
Otherwise there is a risk of false positives in the assessment. This assessment is to be done by an impact calculation in the detailed assessment with use of different combinations of water levels and wave parameters with probability of exceedence equal to the target failure probability. This results in a robust assessment without being extremely stringent.

Regarding the assessment for wave impact it has to be noted that the schematization of residual strength is fairly conservative, as the clay layer thickness is assumed to be limited to 1 meter. Especially for failures at lower parts of the slope this might be stringent, as it is unlikely that wave impact failures will lead to flooding here. This is especially relevant for riverine areas, where high waves generally only occur at lower parts of the slope.

7.3 General conclusions

From the probabilistic analyses, it follows that, due to the low strength of grass revetments to impact failures boundary conditions corresponding with an exceedence probability equal to the cross-sectional reliability requirement (P_T) have to be used. Therefore the boundary between impact and run-up zone is shifted towards this level (i.e. the water level with exceedence probability P_T). Consistent safety formats have been given that are efficient and appropriate for assessments with this shifted boundary.

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A Summary of semi-probabilistic assessment of impact and run-up

A.1 Assessment of GEBU - run-up

This section outlines the different steps to be taken when doing a semi-probabilistic assessment of wave run-up on a grass revetment of a dike section in a dike segment with overall safety standard P_{max}.

1. Determine the geometry and relevant input parameters for the run-up model using the Schematization Manual for Grass Revetments (“Schematiseringshandleiding Grasbekledingen”) (van Hoven 2016).
2. If the default failure probability is applied, use as input value for the critical run-up velocity U_c: $\frac{U_{c,5\%}}{\gamma_\beta}$ with partial safety factor γ_β = 1 (i.e. the 5%-quantile value divided by the beta-dependant partial safety factor). If the default failure probability budget is <u>not</u> used, calculate γ_β using one of the following formulas:

Water system	β_T-dependent safety factor
Western Scheldt	$\gamma_\beta = 0.191(\beta_{T,cs} + 2.894) - 0.124\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
Wadden Sea	$\gamma_\beta = 0.175(\beta_{T,cs} + 3.095) - 0.097\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$
IJssel lake	$\gamma_\beta = 0.236(\beta_{T,cs} + 1.931) - 0.165\beta_{\max}$ with $\beta_{\max} = -\Phi^{-1}(P_{\max})$

   Here $\beta_{T,cs}$ is the required cross-sectional reliability index, β_{max} the reliability index corresponding to the safety standard of the dike track P_{max}.

3. Use the above values as input for run-up calculations with different boundary condition combinations (see Chapter 6) that all correspond to a probability equal to P_T, to be calculated by Ringtoets. For other strength parameters, mean values are used.

4. The revetment complies to the safety standard if the resulting cumulative overload D < 7000 m^2/s^2

A.2 Assessment of GEBU - impact

The proposed procedure is as follows for the semi-probabilistic assessment:

- Evaluate the revetment with boundary conditions with probability equal to the cross-sectional target failure probability P_T. These boundary conditions are combinations of waves and water levels to be derived using the module for loads on revetments. These should be derived for a selection of water levels as depicted in Figure 5.8.
- For the assessment the 5%-resistance duration curve should be used :

Table 8.1 Proposed parameters of the duration curve

Parameter	**Representative value (closed sod)**	**Representative value (open sod)**
F_{sand}	from schematization	from schematization
a	1	0.8
b	-0,035	-0.07
c	0,25	0.25
d_c	from schematization	from schematization

The representative values are described in Appendix G, and are 5%-values of the resistance-duration curves based on experimental data. For the sand fraction the 50%-value can be used.

B Parameters in the wave run-up and wave impact models

B.1 Input parameters for the GrassRunup-kernel

Parameters for definition of the revetment:

Description	Variable	Unit
Duration of stationary time interval	ΔT	[h]
X-coordinates cross section (profile), (x_1,..., x_m)	X	[m]
Y-coordinates cross section (profile), (y_1,..., y_m)	Y	[m]
Roughness factor dike segments (r_1,..., r_{m-1})	r	[-]
factor between runup level and max front velocity	c_u	[-]
Factor for increased load at transitions and objects	α_M	[-]
Factor for decreased strength at transitions and objects	α_S	[-]
Level of interest on the outer slope	Z_{eval}	[m NAP]
Critical wave runup front velocity along the slope	U_c	[m/s]
Critical value of cumulative overload	D_{crit}	[m^2/s^2]

Parameters for definition of the load:

Description	Variable	Unit
Constant, ratio of T_m and $T_{m-1,0}$	$c_{Tm\_Tm-1,0}$	[-]
Duration of the storm [in h]	t_{storm}	[h]
Tidal amplitude [in m]	h_{Tide}	[m]
Wave steepness	sop	[-]
Water system: determines relation between waves and water level in accordance with (Kaste & Klein Breteler 2012)	watersystem	[-]

The direction of the revetment is assumed perpendicular to the direction of the incoming waves.

B.2 Input parameters for the GrassImpact-kernel

Parameters for definition of the revetment:

Description	Variable	Unit
Constant in relation between wave height and strength duration	a	m
Constant in relation between wave height and strength duration	b	1/hr
Constant in relation between wave height and strength duration	c	m
Thickness of the combination of the top layer and the sub layer (clay)	d_c	m
Sand fraction in the clay	F_{sand}	-
Upper limit of grass section	$z_{grass,max}$	m NAP
Lower limit of grass section	$z_{grass,min}$	mNAP
Duration of stationary time interval	ΔT	hr
Vertical stepsize between evaluation levels	Δz	m

Parameters for definition of the load:

Description	**Variable**	Unit
Constant, ratio of T_m and $T_{m-1,0}$	$c_{Tm\_Tm-1,0}$	[-]
Duration of the storm [in h]	t_{storm}	[h]
Tidal amplitude [in m]	h_{Tide}	[m]
Vave steepness	sop	[-]
Water system: determines relation between waves and water level in accordance with (Kaste & Klein Breteler 2012)	watersystem	[-]

C Definition of the test set for wave run-up

This appendix presents considerations on the definition of a proper test set to be used in the calibration of safety factors for wave run up on grass revetments on outer slopes. This test set should be representative for grass revetments in the Netherlands.

C.1 Set up of the assessment

The assessment of grass revetments on outer slops consists of an assessment of either wave impact (if the grass revetment is located below the design water level) or wave runup (in other cases). For wave runup a level Z_{eval} is defined at which the revetment is assessed. This level is typically the lower edge of the grass revetment, where it is adjacent to another type of revetment (e.g. block or asphalt). The grass cover will be normative at this point. For this level the cumulative overload of a design storm is determined and it is assessed whether this satisfies the safety requirement.

C.2 Estimates and considerations of the most important parameters

C.2.1 Load parameters

For previous calibrations a simple model has been used for the boundary conditions (Jongejan 2014; Kanning & Den Hengst 2013). It specifies 3 different water systems: IJsselmeer, Waddenzee and Westerschelde. The model relates a water level to a maximum wave height which is subsequently translated into a storm event based on a basic schematization of a storm.

As we are dealing with a cumulative load model the number of waves is very important for the result, and this is largely determined by the wave steepness and storm duration. However, it is not clear if the calibration is dependent on the number of waves, as a design is made for the limit state after which a failure probability is calculated. An analysis was conducted on whether the wave steepness influences the safety factors. If this is not the case a standard wave steepness of 0.04 will be used. Otherwise cases with different wave steepnesses between e.g. 0.02 and 0.05 will have to be used in the test set.

There is also variation in wave height per subsystem. To take that into account, a multiplication factor is used to scale the wave height.

Approach:

Derive separate safety factors for 3 different water systems (Westerschelde, Waddenzee and Ijsselmeer). If safety factors are roughly the same there is no need for regional variation of safety factors, otherwise the β-γ-relations should be specific for different (classes of) hydraulic conditions. A factor(F_{WH}) is used to scale the wave height to either 75%, 100% or 125% of the original wave height.

For the wave steepness a small test will be executed to determine whether varying it is necessary.

C.2.2 c_u

This dimensionless parameter relates the wave run-up velocity to the run-up height. For individual waves it is found to be distributed with mean = 1 and a standard deviation of 0.25. However, in the calibration we model storm events so this distribution cannot be used, as over a storm the variation will be averaged out due to the large number of waves. It has been proposed to use 1.1 as a representative, deterministic value. From the experimental data it appears also that the value of c_u differs per revetment. It is assumed that using a representative value of 1.1 is sufficiently safe, so a deterministic value of 1.1 is used in the calibration. However, it should be noted that from a probabilistic point of view this is not the preferred way to do it, as the uncertainty on c_u should be a part of the safety format.

C.2.3 Strength parameters

For the cumulative overload model various strength parameters are of importance. These parameters including estimates for their value are discussed below. It has to be noted that while the uncertainties are considered, some parameters might have small (assumed) uncertainties, yet their values can have large implications for the assessment result.

$\underline{\alpha_m}$

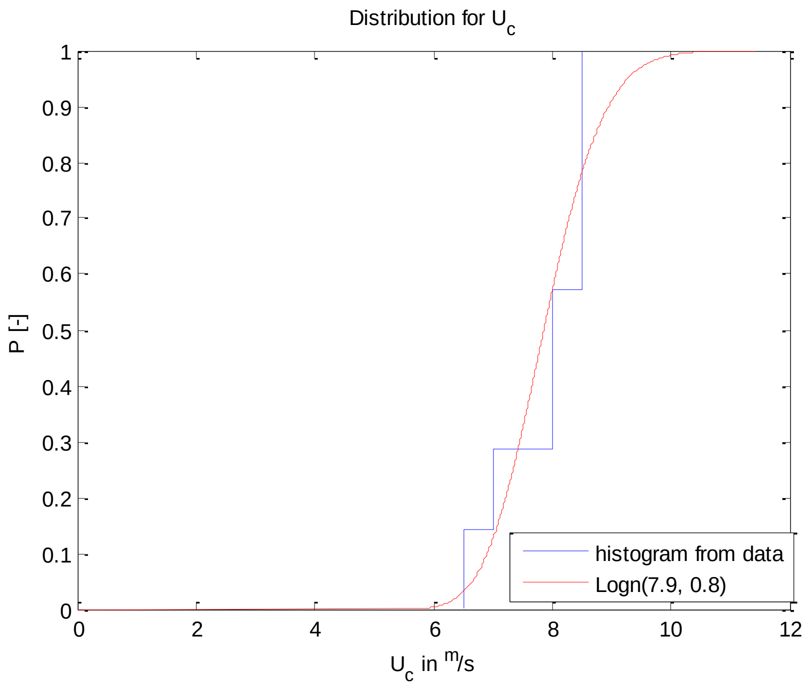
This parameter is a factor to model the increase in load at a transition between 2 types of revetments. It is highly uncertain and still under discussion with experts. Theoretically the value should be between 1 and 2. Theoretically a transition from relatively smooth asphalt to relatively rough grass would result in a factor of approximately 1.8; there has however been argument to use a value of 1.2 for a well-maintained transition between asphalt and grass, which signifies the ambiguity of this parameter. A practical starting point would be an average value of 1.6 with a coefficient of variation of 0.2, with use of the theoretical upper and lower bound.
Approach: as there is still a lack of knowledge on this factor it was decided to use a deterministic value of 1.

$\underline{\alpha_s}$

This is a similar parameter but for reduced strength at transitions. This value has limited uncertainty and for a transition from hard (block/asphalt) to grass it is between 0.8 and 1. A sound stochastic estimate would be an average of 0.9 and a coefficient of variation of 0.05.
Approach: as it was decided not to use the still uncertain knowledge on this factor it was decided to use a deterministic value of 1.

$\underline{U_c}$

U_c is the critical velocity of the wave front of an uprunning wave. It has been derived based on overflow experiments. It models the sods resistance against erosion. For a closed sod, after correcting the experimental data of overflow experiments, the experiments result in a mean of 7.9 m/s with a standard deviation of 0.8 m/s. For a grass sod on a clay substrate the minimum value found is 6.5 m/s, the maximum is 8.5 m/s. It has to be noted that experts agree on the fact that for runup the critical velocity is likely higher than for overflow. This hasn't been proven with experiments yet. Also it is likely that the distribution for U_c based on the different experiments is a representative distribution for the mean, but the standard deviation is not necessarily representative for a dike section (i.e. It is a regional test set, while a local test set would be more appropriate for the assessment). It is likely that, when doing multiple experiments at 1 section the standard deviation found will be lower. There is no physical basis to adapt the distribution for this reason.
For open sods the mean is assumed to be 5.5 m/s, with the same standard deviation of 0.8 m/s. The latter might not be the best estimate but enables assessing with the same safety factors. However, it has to be noted that in coastal areas open sod revetments are most likely not acceptable, due to the large waves. For open sods a 10% lower average has been assumed in the past, without further experimental validation.
Approach: U_c is used as a design variable. This means that the mean value will be used to design a revetment at exactly the limit state. As an initial distribution, a lognormal distribution with mean 7.9 m/s and standard deviation of 0.8 m/s will be used.



$\underline{D_{crit}}$

This is the critical cumulative overload. For failure it is found to be 7000 m^2/s^2 based on overflow experiments. As for experimental validation experiments for overflow have been used, there might be a slight discrepancy of the failure mechanism model and the used experimental verification. Thus it is suggested to use a coefficient of variation of 0.1 to cover this uncertainty.

Approach: D_{crit} will be modelled using a lognormal distribution with mean 7000 and a coefficient of variation of 700.

C.2.4 Dike geometry

Slope

The slope of the revetment is an important factor for the runup velocities and runup height. Slopes typically vary between 1:3 and 1:6, although 1:6 is rarely seen for grass revetments. Occasionally slopes of 1:2.5 can also be found. From the test calibration it appeared that the slope did not have influence on the safety factor. Therefore a fixed slope of 1:4 was assumed, with a sensitivity analysis for different slopes for 1 case.

$\underline{Z_{eval}}$

This is a very important parameter in the assessment. It is usually the lower bound of the grass revetment which is, if an assessment of runup is done is above design water level. This parameter was varied in the calibration, between 0 and 1 meter above the design water level.

C.2.5 Safety standard

For deriving the β -γ-relations the safety standards should be varied. For the calibration 3 safety standards are used: 1/1000, 1/10000 and 1/100000 years.

C.3 Summary of the test set

For the calibration the following parameter variations have been applied:

Water system:

- Westerschelde;
- Waddenzee;
- IJsselmeer.

Wave steepness: 0.04
c_u: 1.1
α_m & α_s: 1
Z_{eval}: between 0 and 1 meter above assessment water level in 4 steps
F_{WH}: 0.75, 1 or 1.25
Slope: 1:4, sensitivity analysis with different slopes

Safety standard:
Three safety standards: 1/1000, 1/10000, 1/100000

This results in a total number of calculations of:
3 water systems x 1 slope x 3 safety standards x 4 evaluation heights x 3 wave heights = 108 cases to be calculated for each safety factor.

Summarizing the parameters and distributions:

Parameter	*Distribution*	*Parameters*	*Variation*
Hydraulic load	conditional Weibull	dependent on water system	no
c_u	deterministic	1.1	no
α_m	deterministic	1	no
α_s	deterministic	1	no
U_c	lognormal	mu = design variable σ = 0.8	design variable
D_{crit}	lognormal	mu = 7000 CoV = 0.1	no
Z_{eval}	deterministic	mu = design water level + 0.2 meters	no
F_{WH}	deterministic	1	0.75, 1, 1.25

Depending on the results of the test calibration some varied parameters might be found to have no influence on the resulting safety factors. In that case it is not necessary to vary them in the final calibration.

D Background on parameter c_u in run-up calculations

Figure D.1 from the memorandum by van Hoven (2015), shows that per slope the mean of c_u can vary, as the points with different colors seem to have different means. This means that there is some uncertainty in the assumption that c_u = 1.1 is a sufficiently safe estimate, as the mean of c_u can vary per location. Therefore it is advised to use different scenarios in the test set to represent this uncertainty in c_u. The memorandum by van Hoven (2015) gives further explanation on the background of Figure D.1.

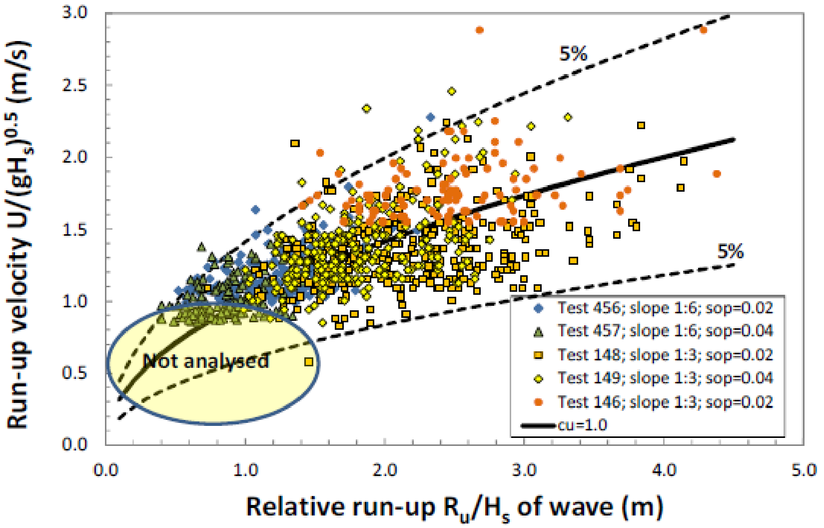
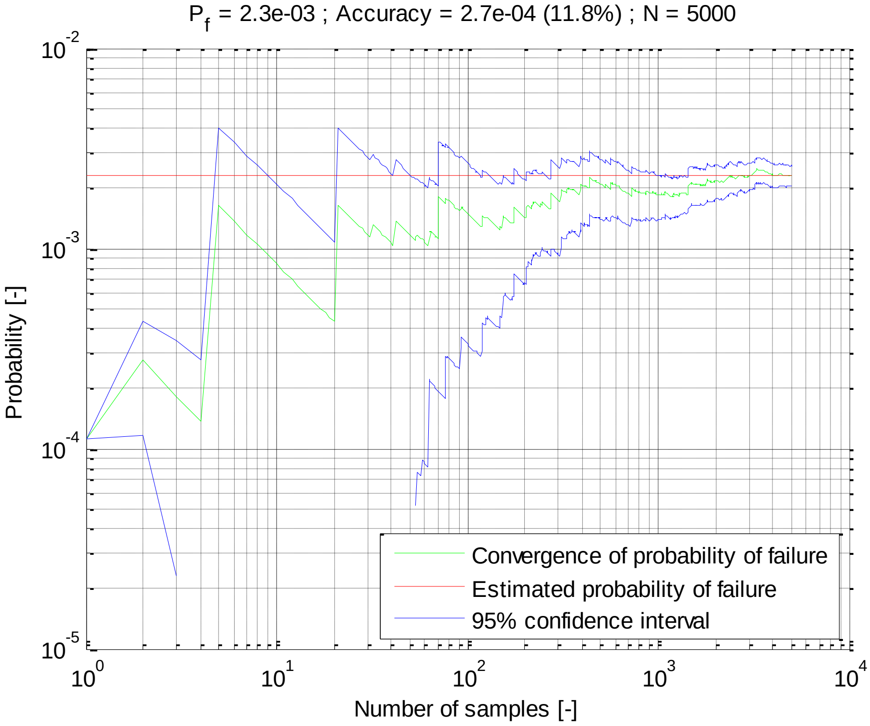


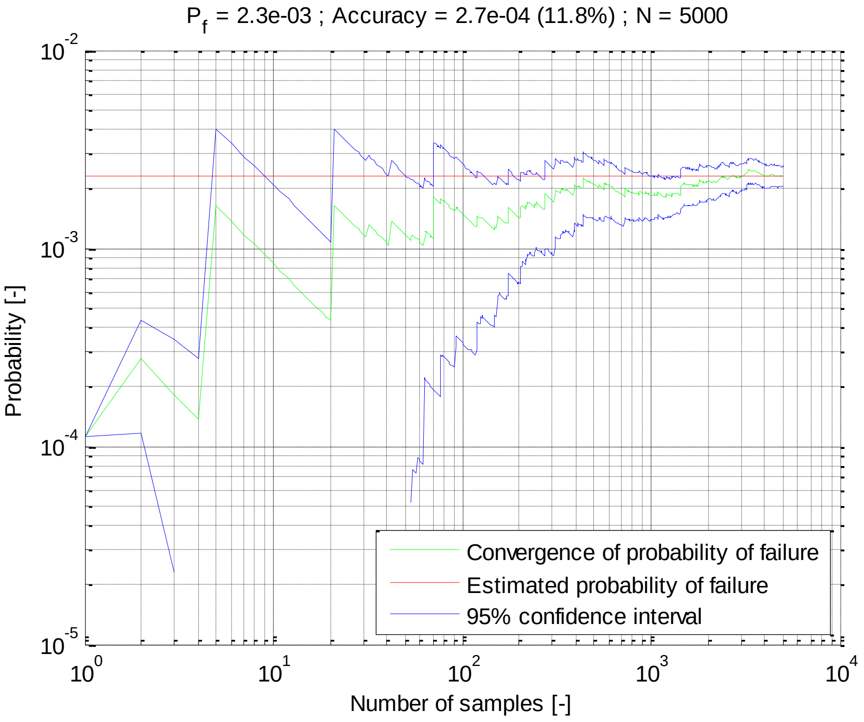
Figure D.1 Relation from which c_u can be determined per wave from *van Hoven (2015)*

E Convergence plots for wave impact calculations using Monte Carlo with Importance Sampling

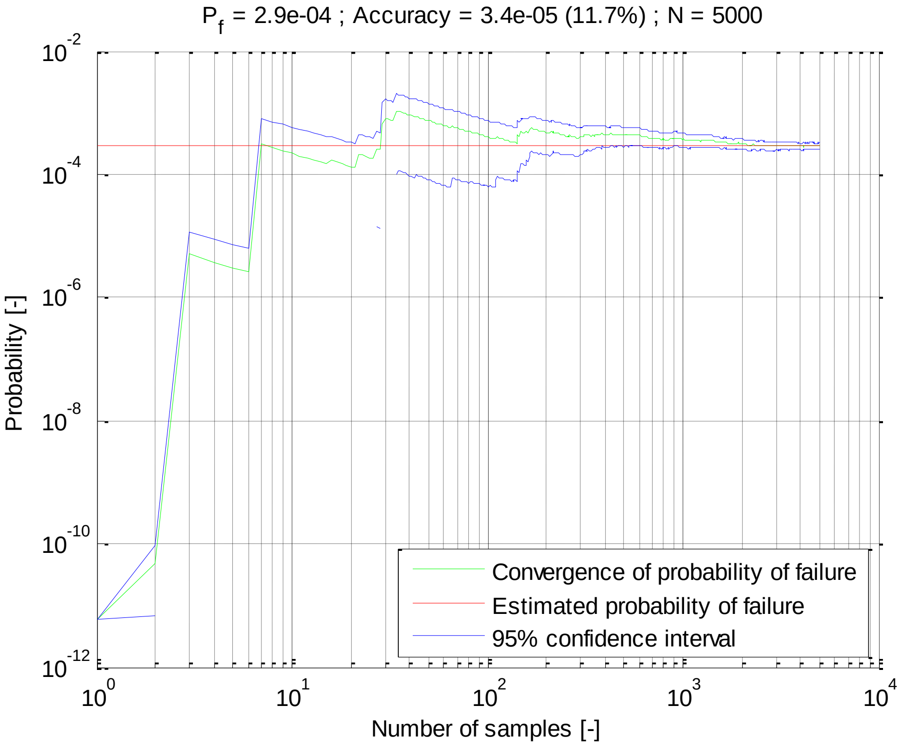
Case 1:



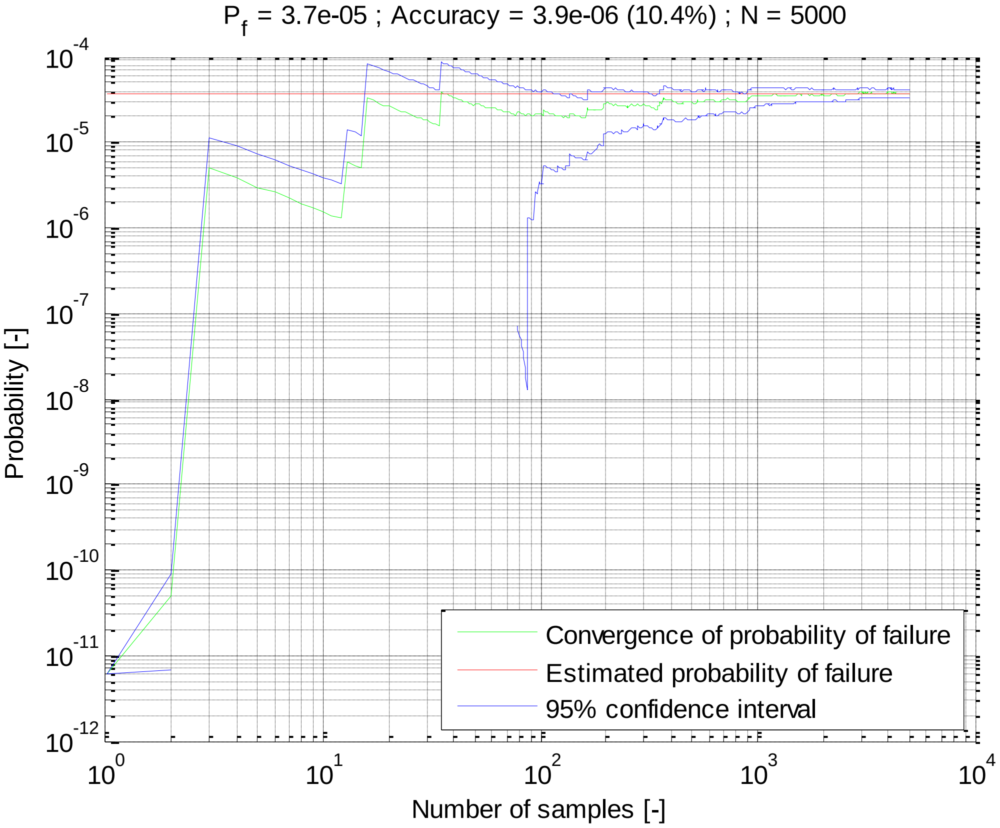
Case 2:



Case 3:


P_f = 2.9e-04 ; Accuracy = 3.4e-05 (11.7%) ; N = 5000
Probability [-]
10^{-2}
10^{-4}
10^{-6}
10^{-8}
10^{-10}
10^{-12}
10^{0}
10^{1}
10^{2}
10^{3}
10^{4}
Number of samples [-]
Convergence of probability of failure
Estimated probability of failure
95% confidence interval

Case 4:


P_f = 3.7e-05 ; Accuracy = 3.9e-06 (10.4%) ; N = 5000
Probability [-]
10^{-4}
10^{-5}
10^{-6}
10^{-7}
10^{-8}
10^{-9}
10^{-10}
10^{-11}
10^{-12}
10^{0}
10^{1}
10^{2}
10^{3}
10^{4}
Number of samples [-]
Convergence of probability of failure
Estimated probability of failure
95% confidence interval

F Fitting method for β-γ relations

Method of least squares

Translating the results from the calibration to linear β-γ relations was done by using a least squares fit. However, a least squares fit can lead to optimistic estimates in some points, as underestimations and overestimations are valued in the same way. To ensure a conservative fit therefore a penalty was applied to the values for which the fitted line resulted in lower safety factors than the calibration result.

In this case a vertical least squares fit was applied, which is based on the following formula:

$$R^2 = \sum [y_i - f(x_i, a_1, a_2, ..., a_n)]^2$$

with y_i the realizations from the calibration and f the resulting values for the fitted line.
In this case for each subarea a separate fit was made using the following standard formula:

$$\gamma_\beta = a\left(\beta_{T,cross,corr} - b\right) - c\beta_{Norm}$$

With a, b and c the parameters used for optimizing the fitted line.

Penalty function

In order to obtain conservative estimates for relations between β and γ a penalty function was applied for values where the fitted line overestimated β.

$$R^2 = R_1^2 + R_2^2$$

with R_1 consisting of all points where $y_i > f(x_i, a_1, a_2, ..., a_n)$ and R_2 consisting of all points where $y_i < f(x_i, a_1, a_2, ..., a_n)$

After that a penalty function P is applied to R_2, which gives for R_1 and R_2:

$$R_1^2 = \sum [y_i - f(x_i, a_1, a_2, ..., a_n)]^2$$

$$R_2^2 = P * \sum [y_i - f(x_i, a_1, a_2, ..., a_n)]^2$$

for which $P = 1 + 1000 * | y_i - f(x_i, a_1, a_2, ..., a_n) |$

The factor 1000 in this function is based on visual inspection of the fitting results to provide a slightly conservative fit.
As an optimization method the standard GRG2 method as implemented in the Microsoft Excel Solver was used.

G Uncertainties erosion model grass wave impact zone

Note: this is a copy of an internal memorandum written by Andre van Hoven.

G.1 Preface

Erosion of the grass revetment in the wave impact zone is modelled by lines describing the time to failure against the wave height H_{m0} (m). A grass revetment endures wave impacts if the revetment is 0 to $H_{m0}/2$ below the water surface.

In the prolonged 3rd safety assessment round, the assessment was carried out by the ‘Handleiding’ (RWS 2012), holding the erosion model which was presumed to be sufficiently safe by means of expert judgement. There was no probabilistic analysis to whether or not the chosen amount of safety was sound, insufficient, or over the top.

The model from RWS 2012 was based on the research report by Gerard Kruse (Deltares 2010), where a less conservative model is given together with the observed failure points and physics description of the erosion process supporting the given model. From Deltares 2010 to RWS 2012 an unknown amount of extra safety was incorporated.

In the following sections the models or resistance lines from Deltares 2010 and RWS 2012 are given. Based on these models, an estimate is given of the best guess model describing a 50% probability of failure and the model for lower probabilities of failure. These models c.q. lines describing the grass failure time against H_{m0} are to be implemented in a kernel and linked to Ringtoets for facilitation of the detailed assessment.

G.2 Resistance lines Deltares 2010

The data points of the grass revetment from tests and field observations are summarized in Table G.1.

Table G.1 Summary observations grass revetments in tests and field observations (Deltares 2010)

Case	Slope 1:..	H_{m0} (m)	Soil (Stevig/ schraal)	Sod (Closed/ open)	t_{fail} (hour)
Deltagoot gras 1993	4	1,35	stevig	closed	17
Deltagoot gras 1993	4	0,76	stevig	closed	20+
Gras scheldegoot	3	0,31	stevig	closed	60+
Gras scheldegoot	3	0,31	schraal	closed	60+
Gras scheldegoot	3	0,31	schraal	open	gat
Gras TUD 1989	3	0,25-0,29	stevig	closed	264+
Gras TUD 1989	1,5	0,25	stevig	closed	264+
Gras TUD 1989	1,5	0,25	schraal	closed	168+
Gras TUD 1989	1,5	0,35	schraal	open	7
Gras TUD 1989	1,5	0,22	stevig	closed	168+
Deltagoot groene dijk	8	1,57	schraal	closed	8+
Emmapolder 1962	4	1-1,5	schraal	open	2?
Duitsland 1962	3	1-1,5	schraal	open	2?

In Deltares 2010 the significant wave height H_s is used, for practical reason in this report H_{m0} is used. It is assumed that for this purpose $H_s = H_{m0}$ is satisfactory. Note that in most cases the failure time t_{fail} (hour) is given with a '+', which means the test was stopped for other reasons than failure of the grass sod. The resistance time is thus an unknown amount of time higher than the given time.

Based on the observations and the physics of the erosion process a subdivision is made in:

slope	1V:2:5H – 1V:4H and 1V:5H – 1V:8H
sod	closed – open
soil	stevig – schraal (firm, plastic – low plasticity, sand like)

The erosion model for slopes 1V:2:5H – 1V:4H given by Deltares 2010 is represented in Figure G.4 and for 1V:5H – 1V:8H in Figure G.2. For practical reasons rather than physics the lines are truncated at 36 hours.

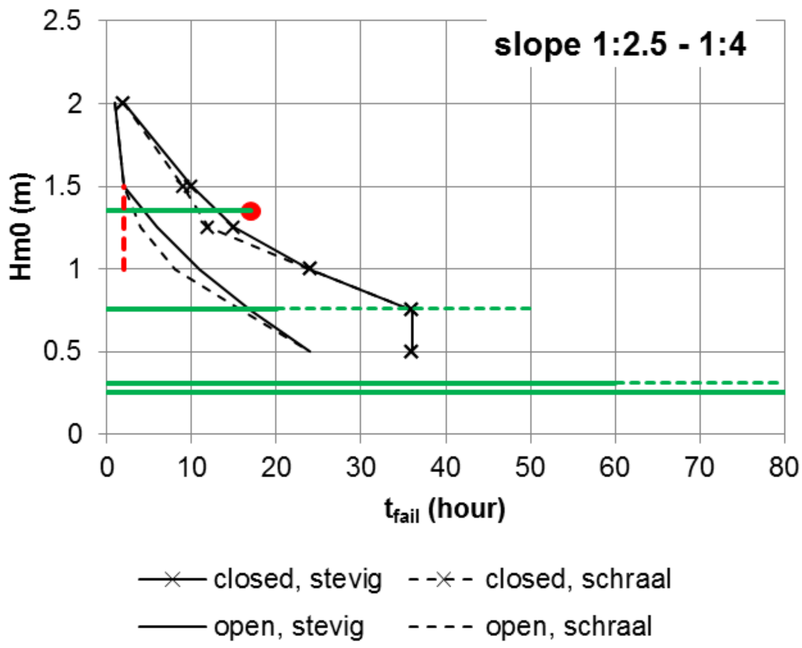


Figure G.1 Resistance lines grass revetment from Deltares 2010 (Table 9.1 page 98), together with flume-test results and field observations for slopes 1:2.5-1:4.

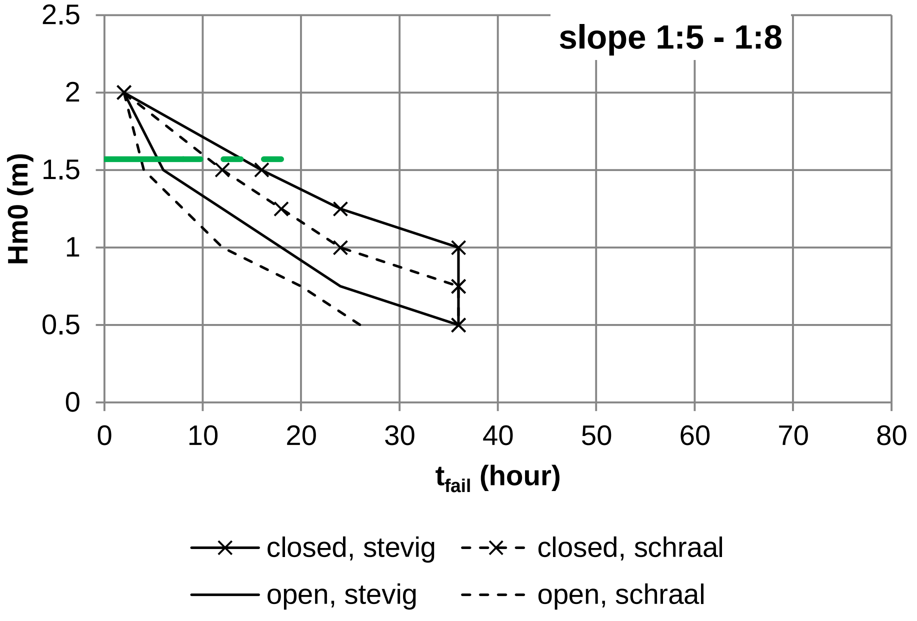


Figure G.2 Resistance lines grass revetment from Deltares 2010 (Table 9.1 page 98), together with flume-test result for slopes 1:5-1:8

For comparison the lines for steep (S – black) and gentle (G – red) slopes are combined in Figure G.3. Obviously the resistance lines for gentle slopes are more favourable than the ones for steep slopes.

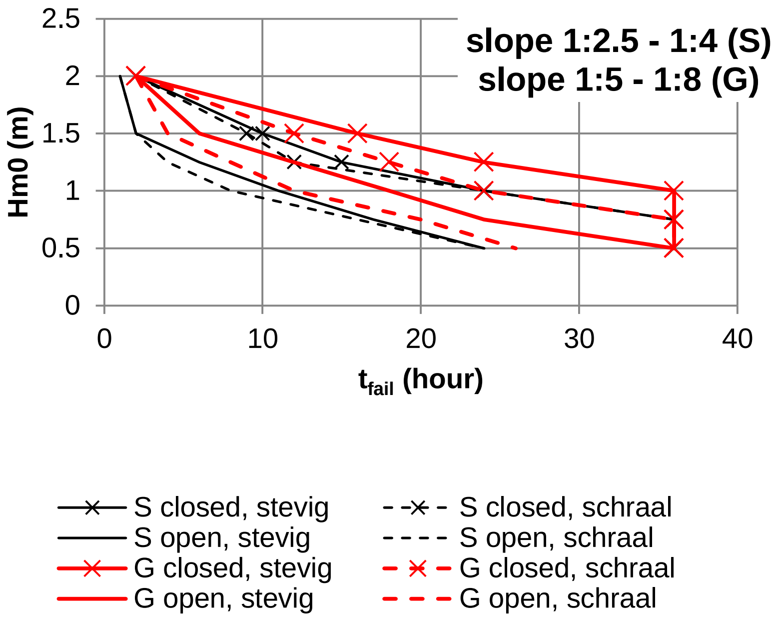


Figure G.3 Resistance lines grass revetment from Deltares 2010 (Table 9.1 page 98), both Steep slopes (S) in black and gentle slopes (G) in red combined

For the RWS 2012 report the lines for gentle slopes were abandoned. These slopes are rare and the observations supporting the better performance of gentle slopes is thin: one Delta flume test.

Support for the safety assessment of gentle slopes (less than 1:5) can be given in the advanced assessment.The extra resistance time for gentle slopes is very plausible from a physics point of view as elaborated in Deltares 2010.

Together whit the author of the Deltares 2010 report Gerard Kruse the lines were extended for longer periods of time at low H_{m0} in order to facilitate the lines for RWS 2012. The extension resulted in de lines in Figure G.4. It is noted that the most interesting part of the graph is the left side, because the common loading time is in the order of 5-24 hours for most extreme conditions.

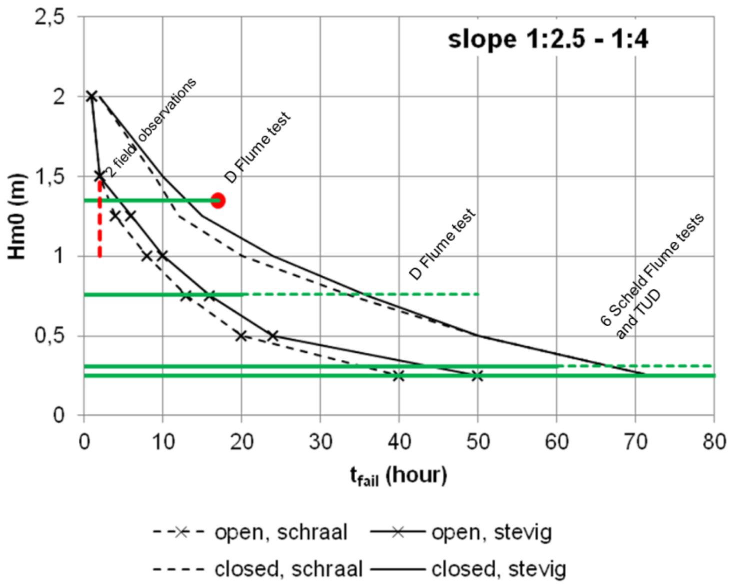


Figure G.4 Resistance time grass sod slope 1:2.5 – 1:4 for open and closed sod on substrate ‘stevig’ and ‘schraal’, taken from Deltares 2010, with representation of tests and field observations

The field observations from Emmapolder and Duitsland (Table 2.1 and dashed vertical red line in Figure G.4) stand out from the lines and the Delta-flume test. It is uncertain in what condition the grass sod was and what the hydraulic condition was (estimated H_{m0} 1-1.5 m). Although the sod grew in a zone where a closed sod was possible, photos of the damages show open spots and it is noted in Deltares 2010 that the sod was not in good condition. If the sod was ‘fragmented’ then only the soil would resist and the resistance time of the sod would be negligible (as observed).

Unfortunately most wave flume tests did not lead to failure at all (marked by a drawn line to the end of the test and a dashed line onward) and most tests were performed with relatively small wave heights (0,25-0,3 m).

Overall the experimental evidence for the lines is limited, but the base of the resistance lines are both tests and observations and a description of the physics involved in the erosion process.

G.3 Resistance lines in Handreiking Toetsen Grasbekledingen (RWS 2012)

The graph presented in Figure G.4 was discussed with the review team levees (review team dijken) and ENW-T. The lines from Deltares 2010 were thought to have too little safety margin. The discussion led to the adjustment to the safe side of the lines given in Figure G.5 .

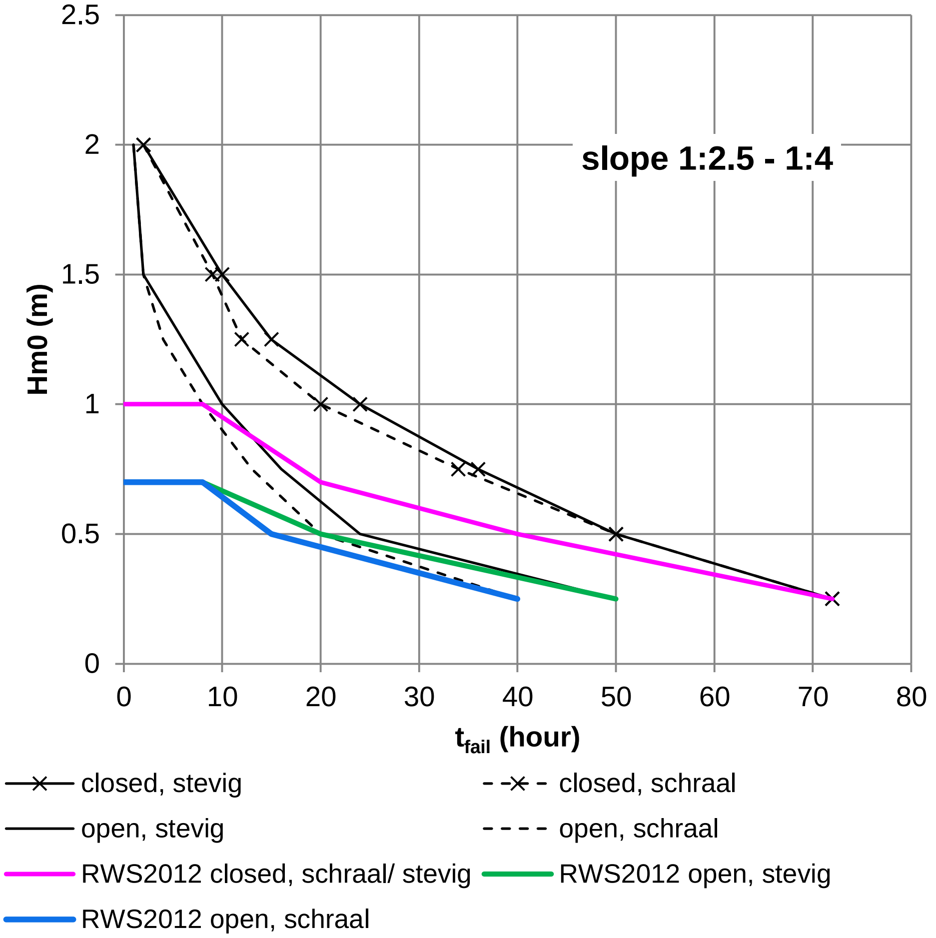


Figure G.5 Resistance lines adjusted Deltares 2010 and RWS 2012

To facilitate the calibration of safety factors the lines are simplified to an analytical formula:

$$H_{m0} = a \cdot e^{b \cdot t_{fail}} + c$$

Further the lines are condensed to two categories 'open' and 'closed' independent of the soil type 'stevig' or 'schraal' and slope angle category. The approximation of the closed and open sod resistance lines with a cumulative probability of exceedance of 50% is given in Figure G.6. The lines are chosen close to the ones given in Deltares 2010.

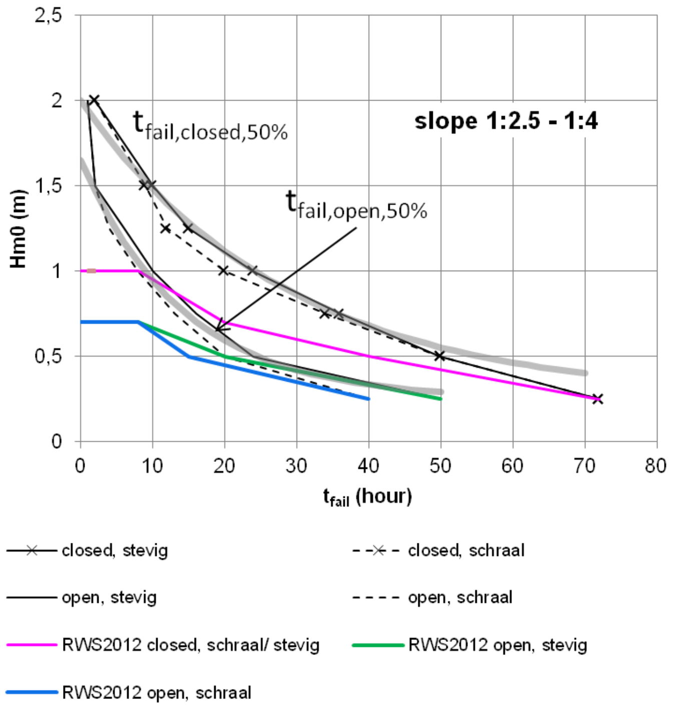


Figure G.6 Approximations of Deltares 2010 resistance lines

The lines given in RWS 2012 are probably close to the 5% cumulative exceedence probability as is assumed to be often the case when a safe estimate is given based on expert judgement. A lower limit where the probability of failure is thought to be very close to zero was estimated (Van Hoven and Klein Breteler). The lines are given in Figure G.7 and Figure G.8. The values of a, b and c are given in Table G.2.

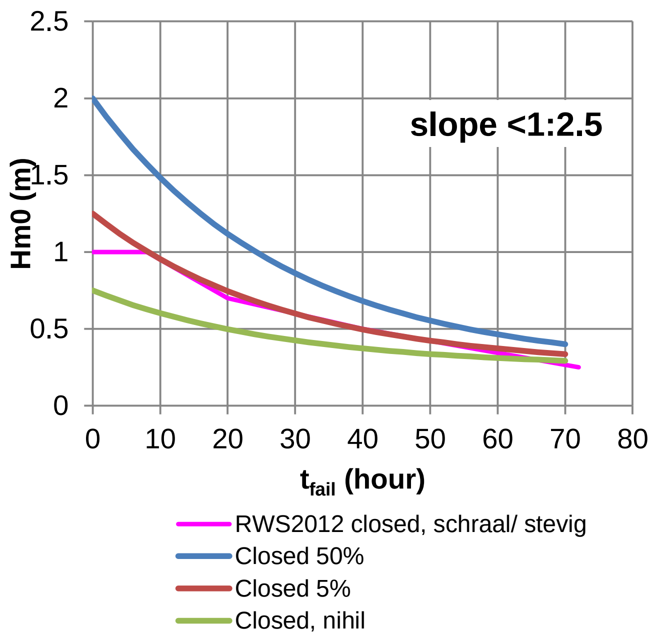


Figure G.7 Resistance lines closed sod rws 2012, and estimates of 50%, 5% and +/-0 prob of failure

Table G.2 Coefficients a, b and c

	closed			open		
	50%	5%	+/- 0%	50%	5%	+/- 0%
a	1,82	1	0,5	1,4	0,8	0,4
b	-0,035	-0,035	-0,035	-0,07	-0,07	-0,07
c	0,25	0,25	0,25	0,25	0,25	0,25

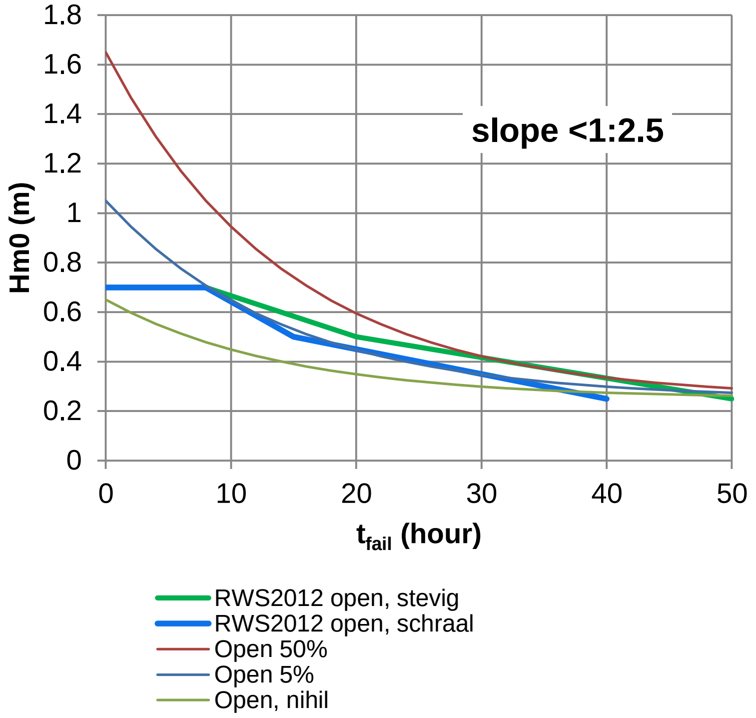


Figure G.8 Resistance lines open sod, RWS 2012 and estimates of 50%, 5% and +/-0 prob of failure

The lines have the following meaning: given a H_{m0} (m), a grass sod in a dike stretch (dijkvak) given a certain quality (open or closed sod) will have a +/-0 %, 5% or 50% probability of failure at the time given by the lines in the figures above, defined by the coefficients in table Table G.2.

G.4 References

Deltares 2010, Studie voor richtlijnen klei op dijktaluds in het rivierengebied, drs. G.A.M. Kruse, Deltares kenmerk 1202512-000-GEO-0002

RWS 2012 Handreiking Toetsen Grasbekledingen op Dijken t.b.v. het opstellen van het beheerdersoordeel (BO) in de verlengde derde toetsronde, Rijkswaterstaat 2012

H Eerste aanzet rekenwaarde tijd tot falen door erosie grasbekleding in golfimpactzone

Note: this appendix is a part of an internal memorandum by Ruben Jongejan. It describes the determination of a distribution for the parameters in the resistance-duration curve.

Inleiding
Dit document vat de belangrijkste resultaten samen van een eerste verkenning naar geschikte rekenwaarden voor een semi-probabilistische toetsing. Het is bedoeld als basis voor discussie over aanpak en uitgangspunten.

De volgende stappen zijn gezet:

1. Beschrijven onzekerheid t.a.v. relatie tussen de tijd tot falen en Hm0.
2. Bepaling betrouwbaarheidseis op doorsnedeniveau.
3. Bepaling rekenwaarde voor onzekerheid t.a.v. de relatie tussen de tijd tot falen en Hm0.
4. Vergelijking met huidige relatie tussen tijd tot falen en de relatie tussen de tijd tot falen en Hm0.
5. Gevoeligheidsanalyse.

Ten slotte worden enkele conclusies een aanbevelingen gepresenteerd.

Stap 1 Beschrijven onzekerheid t.a.v. relatie tussen de tijd tot falen en Hm0
Volgens de notitie van van Hoven (2015) (zie vorige bijlage) kan de relatie tussen de tijd tot falen (t_{fail}) en H_{m0} worden beschreven door:

$$H_{m0} = a \cdot e^{b \cdot t_{fail}} + c$$

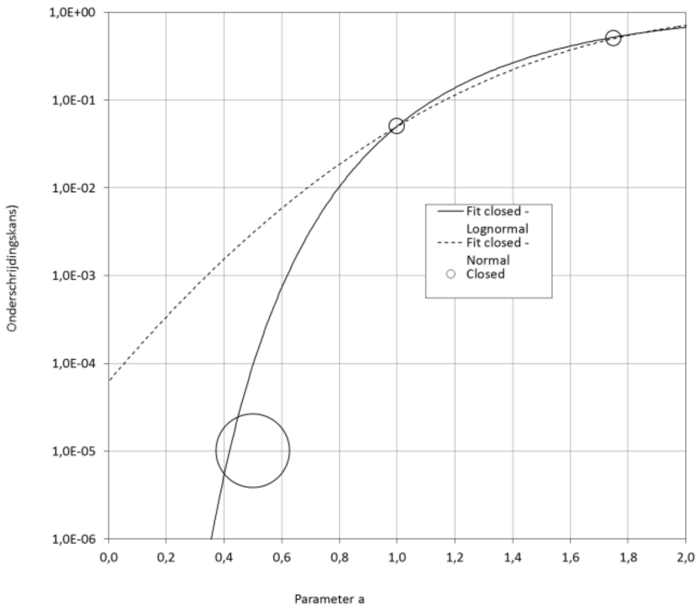
Volgens de tabel 2.2 uit de bewuste notitie is a een stochastische variabele en zijn b en c constanten. In tabel 2.2 zijn voor a de 50%- en 5%-waarden gegeven, voor zowel een open als een gesloten zode. Ook zijn waarden voor a gegeven met een kans op onderschreiding die “nihil” is. Aangenomen is dat een dergelijke beschrijving correspondeert met een onderschrijdingskans van maximaal 10^{-5}.

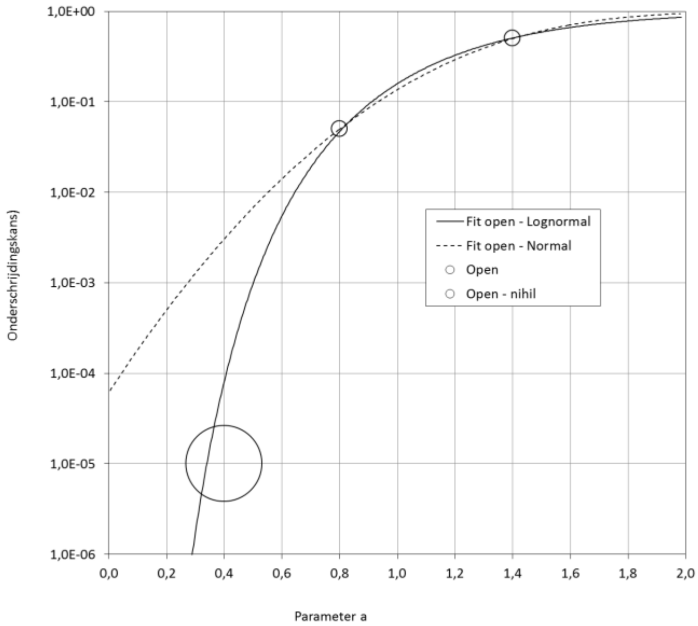
Of de in tabel 2.2 gegeven waarden als 50%- en 5%-waarden beschouwd moeten worden, is van grote betekenis voor de vraag welke standtijden als voldoende veilig aangemerkt kunnen worden. Daarom zijn bij stap 3 ook alternatieven bekeken, waarbij de waarden uit tabel 2.2 als resp. 50%- en 1%-waarden worden geïnterpreteerd, en als resp. 10%- en 0,1%-waarden.

Welke interpretatie de voorkeur verdient, zou op basis van expert judgement bepaald moeten worden, ondersteund door proeven en praktijkervaringen. Een kans moet daarbij worden opgevat als een maat voor de waarschijnlijkheid dat een bepaalde propositie de juiste is. Dat er niet voldoende gegevens voorhanden zijn voor een statistisch onderbouwing van de kansverdelingen is dus geen onoverkomelijk bezwaar.

Er zijn visueel lognormale en normale verdelingen gefit voor parameter a, voor zowel een open als een gesloten zode. De resultaten zijn hieronder weergegeven.

De getoonde lognormale verdelingen stroken het beste met de gegeven onderschrijdingskwantielen. Overigens kunnen ook andere verdelingsparameters en/of verdelingstypes onderzocht worden.





Parameter (voor de lognormaal verdeelde variabele)	*Parameterwaarde– gesloten zode*	*Parameterwaarde – open zode*
Gemiddelde	1,82	1,4
Standaarddeviatie	0,62	0,50
Ondergrens	0,00	0,00

I Memo aanvullende analyses GEBU & GEKB

In this Appendix a memorandum is added which discusses some additional analysis for the GEBU and GEKB mechanisms. This concerns both the simple assessment as well as the detailed assessment. Parts of the memorandum on the detailed assessment for GEBU are very relevant for this report.

Memo

Aan
Betrokkenen WBI-2017 Grasbekledingen

Datum
30 september 2016

Aantal pagina's
28

Van
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Onderwerp
Analyses eenvoudige en gedetailleerde toets GEBU & GEKB

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1 Aanleiding

Binnen WBI-2017 wordt bij het mechanisme GEBU (GrasErosie Buitentalud) onderdeel golfklap in de gedetailleerde toets gebruik gemaakt van randvoorwaarden met overschrijdingskans gelijk aan doorsnede-eis. Bij mechanisme GEKB (GrasErosie Kruin Binnentalud) wordt probabilistisch getoetst met een doorsnede-eis die een stuk groter is dan de trajecteis. Omdat in laag 1 randvoorwaarden worden afgeleid bij trajecteis brengt dit het risico met zich mee dat de eenvoudige toets tot goedkeuren kan leiden waar in laag 2a/b afgekeurd wordt.

Om te verkennen of dit mogelijkerwijs het geval zou kunnen zijn is een groot aantal cases doorgerekend, waarbij voor het mechanisme GEBU laag 1, 2a en een probabilistische berekening worden vergeleken en waar voor het mechanisme GEKB laag 1 en 2b worden vergeleken. Daarnaast wordt een aantal sommen uitgevoerd voor mechanisme GEBU om zo een toetsmethode vast te kunnen stellen voor het rivierengebied/locaties waar wind en waterstand niet of nauwelijks gecorreleerd zijn. Dit memo behandelt dus twee onderwerpen:

- De houdbaarheid van de eenvoudige toets bij zowel GEBU als GEKB
- Het toetsen van grasbekledingen buitentalud in het rivierengebied, inclusief randvoorwaardenschematisatie

Voor het eerste wordt voor een selectie van locaties de eenvoudige toets vergeleken met de gedetailleerde toets. Voor het tweede wordt voor een set locaties in het rivierengebied probabilsitische berekeningen uitgevoerd voor GrasErosie BuitenTalud, submechanisme Golfklap.

In hoofdstuk 2 wordt de schematisatie besproken, in hoofdstuk 3 de resultaten voor GEBU en in hoofdstuk 4 de resultaten voor GEKB. Hoofdstuk 5 bespreekt de conclusies uit deze analyses en hoofdstuk 6 hoe de bevindingen voor GEBU te implementeren zijn in toetslaag 2a.

2 Schematisatie

2.1 Belangrijke termen/kansbegrippen

In dit memo wordt een aantal kansbenamingen en notities gehanteerd. Deze worden hieronder puntsgewijs toegelicht:

- P_{norm}: faalkans bij de norm (-/jaar)
- P_T: doelfaalkans van het submechanisme (-/jaar)
- $F_x^{-1}(1\text{-}p)$: waarde van stochast x met overschrijdingskans p (-/jaar)
- h_{PT}: waterstand met overschrijdingskans P_T (dus eigenlijk ($F_h^{-1}(1\text{-}P_T)$))

2.2 Typologie randvoorwaarden/locaties

Omdat grasbekledingen doorgaans vanaf een bepaald niveau (boven een harde bekleding) liggen speelt in de toetsing het type van het watersysteem een grote rol. In deze studie wordt uitgegaan van 2 hoofdtypen systemen:

- Windgedomineerde locaties: hoofdzakelijk langs de kust en meren. Hier zijn waterstand en golfhoogte sterk gecorreleerd waardoor de belasting (golfhoogte) hoger op het talud zal toenemen. Dit zijn typisch locaties waar golven relatief hoog zijn en laag op het talud een steenbekleding ligt. Voor dergelijke locaties moet ook op golfoploop worden getoetst.
- Waterstandsgedomineerde locaties: hoofdzakelijk in het bovenrivierengebied. Hier zijn waterstand en golfhoogte volledig onafhankelijk, waardoor bij toenemende waterstand de golfhoogte zal afnemen. Dit zijn typisch locaties waar vaak geen steenbekleding zal liggen in verband met relatief lage golfhoogtes. Wanneer er zich toch steenbekledingen bevinden zal dit vaak zijn ter bescherming tegen scheepsgolven.

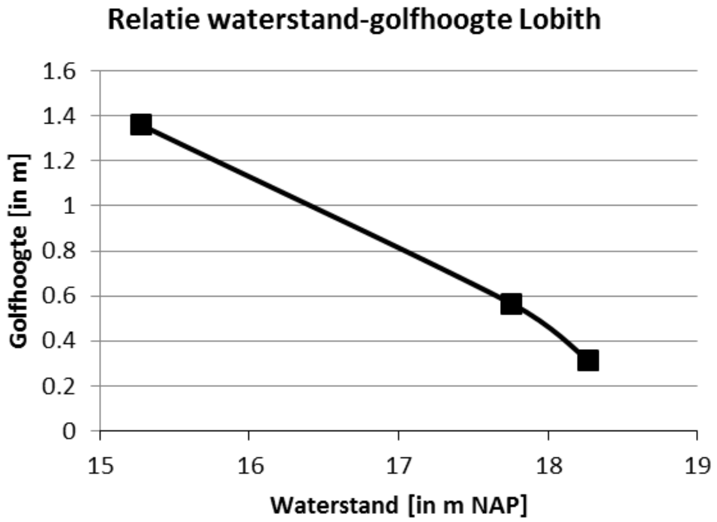
2.3 Randvoorwaarden GEBU

Voor de afleiding van de randvoorwaarden zijn verschillende methoden beschikbaar. In de eenvoudige toets wordt voor alle mechanismen een waterstand en golfhoogte afgeleid voor overschrijdingskans gelijk aan de norm (P_{norm}) met behulp van de marginale statistiek van beide parameters. Voor windgedomineerde locaties, waar de correlatie sterk is is dit een redelijke benadering, voor locaties zonder correlatie tussen wind en waterstand is dit een tamelijk conservatieve benadering.

2.3.1 Randvoorwaarden bekledingen in de gedetailleerde toets GEBU

Voor het afleiden van randvoorwaarden voor bekledingen in de gedetailleerde toets wordt gebruik gemaakt van de Q-variant. Deze bepaalt voor gegeven waterstand en overschrijdingskans het illustratiepunt van de belasting, met als resultaat een significante golfhoogte H_s, golfrichting θ en golfperiode T_p. Wanneer de Q-variant gebruikt wordt voor windgedomineerde locaties wordt de waterstand met gewenste overschrijdingskans als invoer gegeven en worden de bijbehorende golfparameters bepaald, wat goed werkt en eenduidig is.

Voor waterstandsgedomineerde locaties ligt e.e.a. ingewikkelder omdat hierbij, voor de waterstand $F_h^{-1}(1\text{-}P)$ een zeer lage golf volgt omdat alle kansruimte wordt ingenomen door de waterstand. Dit is het best uit te leggen aan de hand van Figuur 2.1, waar de resultaten voor Lobith zijn weergegeven. Hier is te zien dat bij toenemende waterstand de golfhoogte afneemt voor een gelijkblijvende kans van de combinatie van waterstand-wind. De punten op de lijn komen overeen met overschrijdingskansen van de waterstand van respectievelijk 1/10, 1/30000 (norm) en 1/660000 (faalkanseis GEBU). Wanneer voor deze locatie wordt getoetst met de golfcondities bij de hoogste waterstand, dan wordt de bekleding ruimschoots goedgekeurd, wanneer dit wordt gedaan met de golfcondities met de laagste waterstand dan wordt de bekleding afgekeurd. Derhalve moet bij het toetsen met randvoorwaarden afgeleid met de Q-variant altijd goed worden gekeken naar de gebruikte golfhoogte.



Figuur 2.1 Relatie tussen waterstand en golfhoogte in de Q-variant voor locatie Lobith

2.3.2 Randvoorwaarden bekledingen in probabilistische berekeningen GEBU

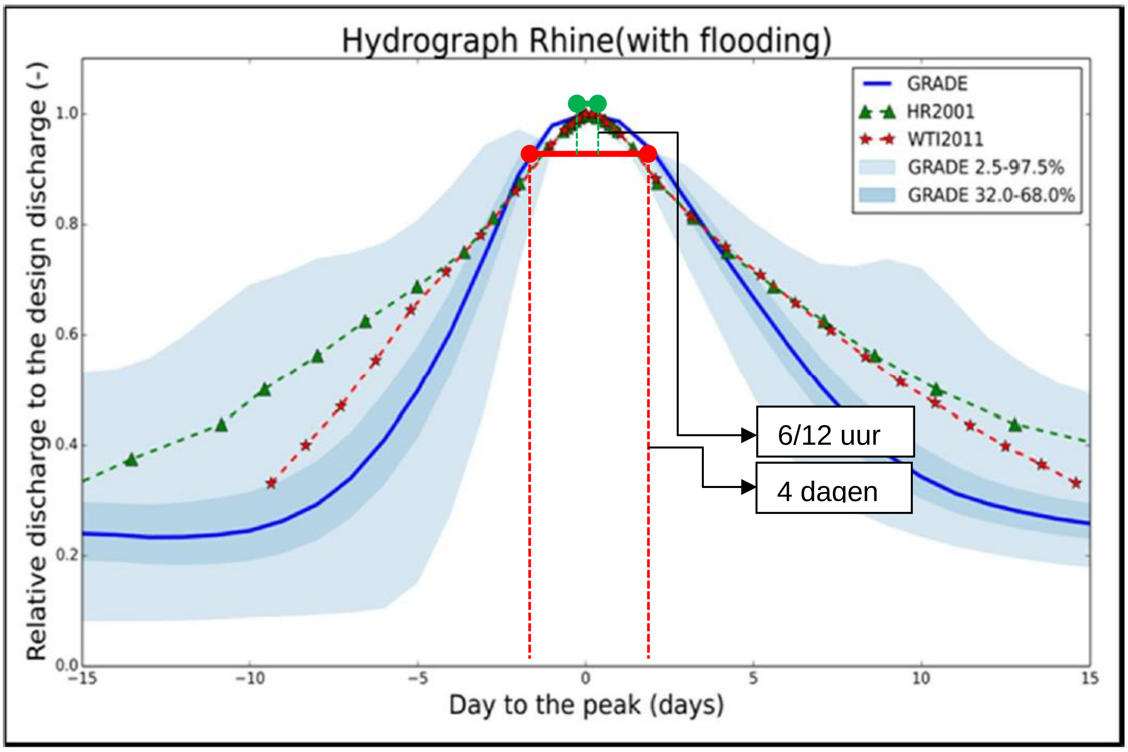
Voor het maken van probabilistische berekeningen kan gebruik gemaakt worden van resultaten uit de Q-variant. De verwachting is dat dit voor windgedomineerde locaties redelijk zal werken, belangrijke kanttekening is echter dat de Q-variant illustratiepunten geeft en geen kansverdelingen. Hierdoor valt in feite een belangrijk deel van de informatie uit de onderliggende randvoorwaardenstatistiek weg. Bij windgedomineerde locaties zal doorgaans 1 dominante windrichting zijn en is het illustratiepunt een goede weergave van de onderliggende statistiek. Voor complexere locaties, bijvoorbeeld locaties waar stochasten niet of nauwelijks gecorreleerd zijn, is een groter aantal randvoorwaardencombinaties met eenzelfde overschrijdingskans. Hierdoor een illustratiepunt niet per se een goede weergave.

Een nettere omgang met randvoorwaarden in probabilistische berekeningen is het gebruik van de marginale statistiek. Hierbij wordt de marginale statistiek van zowel waterstand als golfhoogte afgeleid met Hydra-Ring en wordt uit beiden een trekking gedaan. Deze methode werkt vooral goed voor locaties met volledige of geen correlatie omdat de trekking dan ook volledig of niet gecorreleerd kan worden gedaan. Wanneer de correlatiestructuur complexer wordt moet deze ook meegenomen worden wat extra complexiteit toevoegt. Omdat van de meeste locaties waarvoor probabilistische berekeningen worden gemaakt voor GEBU het uitgangspunt is dat er (vrijwel) geen correlatie is, is dit een goede benadering. Opgemerkt dient te worden dat bij het niet-gecorreleerd trekken de marginale statistiek niet zomaar vermenigvuldigd kan worden omdat dan in feite de kans dat beide stochasten in een jaar hoog zijn wordt uitgerekend, en niet de kans dat beide stochasten op hetzelfde moment hoog zijn. Hiervoor moet een factor in rekening worden gebracht die afhankelijk is van de aangenomen belastingduur van beide stochasten.

In deze studie is voor locaties in het benedenrivierengebied naast een som met ongecorreleerde wind en waterstand ook een som met gecorreleerde wind en waterstand gedaan, wat een boven-/ondergrens benadering geeft.

2.3.3 Schematisering belastingen in het rivierengebied bij berekeningen GEBU

Voor de berekeningen van GEBU en met name ook de correctie van de marginale statistiek in de berekeningen is de belastingduur van waterstand en wind van belang. Normaliter wordt voor de waterstand in afvoergedomineerde gebieden gerekend met een afvoergolf van ongeveer 30 dagen. Omdat belasting door golfklappen met name interessant is op 1 en hetzelfde waterstandsniveau is voor deze studie de aanname gedaan dat de belastingduur van de waterstand 4 dagen is. Opgemerkt dient te worden dat deze belastingduur met name van belang is voor de kans van samenvallen van beiden stochasten. Voor de duur van de storm is met zowel een stormduur van 6 uur als 12 uur gerekend. Dit correspondeert met de duur die in het verleden werd gebruikt voor wind in het rivierengebied. Figuur 2.2 geeft schematisch de belastings-schematisatie weer. Voor locaties in het Benedenrivierengebied wordt dezelfde schematisatie aangehouden.

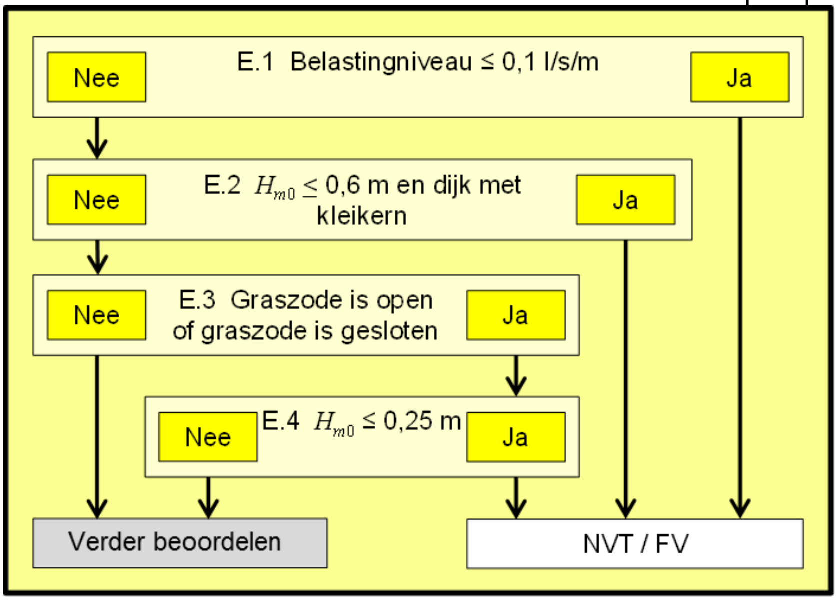


Figuur 2.2 Belastingsschematisatie voor het rivierengebied voor berekeningen voor golfklap op gras.

2.4 Toetsschema's

2.4.1 GEBU

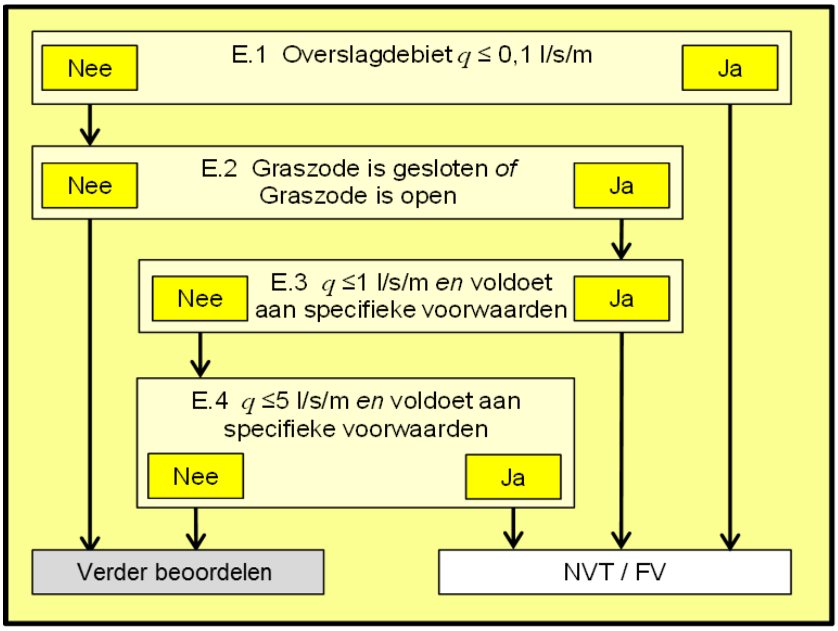
De eenvoudige toets voor GEBU bestaat uit 4 stappen, zie Figuur 2.3. De eerste is een check op het belastingniveau, dit is een zeer conservatieve formule voor de formule gebruikt voor het mechanisme GEKB. De rest van de stappen heeft te maken met de golfhoogte. Wanneer deze voldoende klein is en/of er voldoende reststerkte is vanuit de kleikern mag de bekleding worden goedgekeurd. Opgemerkt dient te worden dat de eenvoudige toets gevolgd wordt door 2 sporen in de gedetailleerde toets: een toets op klap en op oploop. Voor de in dit memo beschreven rivierenlocaties wordt geen toets op oploop gedaan omdat de aanname is dat er zich geen harde bekleding bevindt in de golfklapzone en het gras in de golfklapzone maatgevend is boven gras in de golfoploopzone. Wanneer de grasbekleding onvoldoende sterk is komt dit dan voldoende naar voren vanuit de toets op klap.



Figuur 2.3 Toetsschema eenvoudige toets GEBU

2.4.2 GEKB

De eenvoudige toets voor het toetsspoor erosie kruin en binnentalud verloopt volgens het schema in Figuur 13.1



Figuur 13.1 Schema eenvoudige toets van grasbekleding erosie kruin en binnentalud (GEKB)

<u>Stap E.1: Overslagdebiet ≤ 0,1 l/s/m.</u>
Bij een overslagdebiet kleiner of gelijk aan 0,1 l/s/m, wordt aangenomen dat de bekleding de belasting zonder ontoelaatbare schade kan weerstaan: de faalkans is verwaarloosbaar klein. Als de belasting groter is dan 0,1 l/s/m wordt de beoordeling voortgezet met Stap E.2.

Het overslagdebiet wordt met de volgende formule bepaald:

$$q = 200 \cdot \sqrt{g \cdot H_{m0}^3} \cdot e^{\frac{-2.6(h_k - h)}{H_{m0}}} \qquad \text{Vgl 2.1}$$

Waarin:

h_k	Kruinhoogte ten opzichte van NAP [m].
h	Waterstand ten opzichte van NAP [m].
H_{m0}	Significante golfhoogte, gebaseerd op spectrum [m].
q	Gemiddeld overslagdebiet [l/s/m].
g	Zwaartekrachtversnelling [m/s^2].

Voor deze eenvoudige toets dienen conform Bijlage II Hydraulische belastingen de volgende parameters te worden bepaald:

- Waterstand h ten opzichte van NAP [m].
- Golfhoogte H_{m0} [m].

Deze parameters worden met behulp van de WBI-software berekend voor verschillende overschrijdingskansen [1/jaar]. Voor deze eenvoudige toets moeten deze parameters worden bepaald behorende bij de norm.

<u>Stap E.2: Kwaliteit graszode.</u>
Stap E.2 bestaat uit een beoordeling van de kwaliteit van de graszode op de kruin en het binnentalud van de dijk. Indien de graszode *open* of *gesloten* is, wordt de toets voortgezet met Stap E.3. Indien de graszode *fragmentarisch* is, dan kan op grond van deze eenvoudige toets geen oordeel worden geveld. Hoe de grasbekleding moet worden ingedeeld in een van de drie categorieën (*gesloten* zode, *open* zode of *fragmentarische* zode) is toegelicht in de Schematiseringshandleiding grasbekleding.

<u>Stap E.3: Sterkte criterium gegeven q≤1 l/s/m.</u>
Bij een overslagdebiet kleiner of gelijk aan 1 l/s/m is de faalkans van de grasbekleding verwaarloosbaar klein in één van de volgende situaties:

- Graskwaliteit is *gesloten* zode.
- Graskwaliteit is *open* zode op een kleilaagdikte van minimaal 0,4 m.

Indien niet aan deze voorwaarden wordt voldaan wordt de toets voortgezet met Stap E.4.

<u>Stap E.4: Sterkte criterium gegeven q≤5 l/s/m.</u>
Bij een overslagdebiet kleiner of gelijk aan 5 l/s/m is de faalkans van de grasbekleding verwaarloosbaar klein wanneer alle vijf volgende voorwaarden van toepassing zijn:

1. Graskwaliteit is *gesloten* zode.

2. Golfhoogte $H_{m0} \leq 3$ m.
3. Kleilaagdikte $d_{klei} \geq 0{,}4$ m òf taludhelling flauwer dan 1V:4H.
4. De snede door de zode van objecten (NWO's) is kleiner dan 0,15 m. Met snede door de zode wordt bedoeld de grootste afmeting van het object (NWO) gemeten in het vlak van de bekleding.
5. Geen wegen of (fiets)paden op de dijk.

Indien niet aan al deze vijf voorwaarden wordt voldaan, of het overslagdebiet is groter dan 5 l/s/m, dan kan op grond van deze eenvoudige toets nog geen oordeel worden geveld.

2.5 Keuze locaties

Voor beide mechanismen geldt de vraag of de eenvoudige toets voldoende veilig is zowel voor windgedomineerde als voor waterstandsgedomineerde locaties. Derhalve moet bovengenoemde exercitie voor beide typen locaties worden uitgevoerd. Voor GEBU wordt hiervoor gebruik gemaakt van de kustlocaties uit de studie van Klein Breteler et al. (2016), waarvoor de randvoorwaarden in Figuur 2.4 zijn weergegeven[1]. Voor de rivieren (waterstandsgedomineerde locaties) moeten apart randvoorwaarden worden afgeleid, dit wordt gedaan voor de locaties in Tabel 2.1 . Voor GEKB worden langs de kust andere locaties gebruikt dan voor GEBU, deze zijn weergegeven in Tabel 2.2. Toelichting over de keuze van de locaties is gegeven in Bijlage A, hier wordt ook een aantal van de kolommen in de tabellen nader verklaard. De tabel geeft van links naar rechts weer: signaalkans (P_S) en voor maximaal toelaatbare faalkans (P_{max}), doorsnede-eis (P_T) en faalkanseis HBN (P_{HBN}) telkens waterstand ($h_{P...}$), golfhoogte (H_{m0}) en piekgolfperiode (T_P).

Lo-ca-tie	Norm	Randvoorwaarden bij P_{max}				Randvoorwaarden bij P_T				Randvoorwaarden bij P_{HBN}			
	P_S (/jaar)	P_{Max} (/jaar)	h_{Pmax} (m)	H_{m0} (m)	T_p (s)	P_T (/jaar)	h_{PT} (m)	H_{m0} (m)	T_p (s)	P_{HBN} (/jaar)	h_{HBN} (m)	H_{m0} (m)	T_p (s)
ED 1	3,3E-4	1,0E-3	5,51	0,88	2,83	1,50E-5	6,61	1,13	3,05	8,00E-5	5,96	1,29	3,55
ED 2	3,3E-4	1,0E-3	5,51	1,05	3,10	1,50E-5	6,61	1,36	3,66	8,00E-5	5,96	1,55	3,89
ED 3	3,3E-4	1,0E-3	5,51	1,23	3,35	1,50E-5	6,61	1,58	4,27	8,00E-5	5,96	1,81	4,20
ED 4	3,3E-4	1,0E-3	5,51	1,40	3,58	1,50E-5	6,61	1,81	4,88	8,00E-5	5,96	2,07	4,49
ED 5	3,3E-4	1,0E-3	5,51	1,58	3,80	1,50E-5	6,61	2,03	5,49	8,00E-5	5,96	2,32	4,77
HA 1	3,3E-4	1,0E-3	4,62	2,45	4,81	1,50E-5	5,40	2,97	5,17	8,00E-5	5,07	2,81	5,06
HA 2	3,3E-4	1,0E-3	4,62	1,47	3,73	1,50E-5	5,40	1,78	3,10	8,00E-5	5,07	1,69	3,92
HA 3	3,3E-4	1,0E-3	4,62	1,72	4,03	1,50E-5	5,40	2,08	3,62	8,00E-5	5,07	1,97	4,23
HA 4	3,3E-4	1,0E-3	4,62	1,96	4,30	1,50E-5	5,40	2,37	4,14	8,00E-5	5,07	2,25	4,52
HA 5	3,3E-4	1,0E-3	4,62	2,21	4,57	1,50E-5	5,40	2,67	4,66	8,00E-5	5,07	2,53	4,80
LM	3,3E-4	1,0E-3	4,74	2,14	4,00	1,50E-5	5,62	2,66	4,32	8,00E-5	5,26	2,48	4,29
OS	3,3E-4	1,0E-3	5,64	1,94	4,40	1,50E-5	6,80	2,56	4,89	1,25E-4	5,96	2,49	4,98
VL 1	3,3E-4	1,0E-3	4,93	2,55	6,75	1,50E-5	5,90	3,04	7,19	1,25E-4	5,14	3,42	6,99
VL 2	1,0E-4	3,3E-4	5,18	2,70	6,88	5,00E-6	6,14	3,19	7,29	4,00E-5	5,36	3,56	7,09

Figuur 2.4 Cases uit Klein Breteler et al. (2016) *met randvoorwaarden bij P_{max},P_T en P_{HBN}*

---

[1] De afkortingen van de locaties staan voor:
ED = Eemshaven-Delfzijl
HA = Harlingen
LM = Lauwersmeer
OS = Ossenisse (Westerschelde)
VL = Vlissingen
De nummers duiden aan dat golfhoogtes zijn verhoogd, om zo het bereik van de cases te vergroten.

Tabel 2.1 Locaties in het rivierengebied, gebruikt voor de analyses van GEBU[2]

Locatie	Dijk-traject	HCLDid	Deelsysteem	Overslaghoogte RBSO	Norm	Max strijklengte Hydra-Zoet
Lobith	48-1	100563	Bovenrivieren	>1 m	1/30000	3860
Wageningen	45-1	105454	Bovenrivieren	0.5-1 m	1/100000	2325
Doesburg	49-2	102604	Bovenrivieren	≈1 m	1/1000	4114
Zutphen	50-2	102842	Bovenrivieren	<0.5 m	1/3000	1219
Tiel	43-6	107822	Bovenrivieren	0.5-1 m	1/30000	3218
Cabauw	15-1	306305	Benedenrivieren	0.5 m	1/30000	993
Gorinchem	16-1	310211	Benedenrivieren	≈0.5 m	1/100000	2103
Moerdijk	34-2	304251	Benedenrivieren /Haringvliet	nb	1/1000	5889

Tabel 2.2 Extra locaties langs kust en meren, gebruikt voor de analyses voor GEKB

Locatie	HCLDid	Deelsysteem	Norm	Factor referentie-0.1	Naam in Hoffmans(2015)
Breskens	1500681	Westerschelde	1/1000	40	hm 68.800 - 69.600
Maeslantkering	1700175	Benedenrivieren	1/10000	2.7	D161.3-D173.0_rd06
Kloosterzande	1500454	Westerschelde	1/1000	1	hm 19.610 - 20.090
Harlingen	1000501	Waddenzee West	1/3000	9.79	DV_064_0.15_1.85
Oosterend Texel	1000151	Waddenzee West	1/1000	-	-
Rotterdamse Hoek	700056	IJsselmeer	1/3000	-	-
Cornwerd	700003	IJsselmeer	1/1000	-	-

2.6 Overige aandachtspunten t.a.v. de berekeningen

Overige aandachtspunten zijn hieronder toegevoegd:

- Probabilistische berekeningen voor GEBU zijn doorgerekend met MCIS (Monte Carlo met Importance Sampling). Hierbij is H_s gesampled, en in gevallen waar de ondergrens van het gras op het niveau $F_h^{-1}(1\text{-}1/10)$ lag ook de waterstand. Voor alle cases is gerekend met 20.000 samples wat leidde tot, afhankelijk van de faalkans, orde 1000-10000 faalsamples per case.
- GEBU is alleen probabilistisch doorgerekend voor de rivieren, niet voor de kust, daar is 2a toets voldoende (zie ook kalibratierapport voor achterliggende probabilistische berekeningen).
- Er is uitgegaan van een gesloten zode en afwezigheid van een kleikern, tenzij anders vermeld. De dikte van de kleilaag is aangenomen op 0.8 meter en de zandfractie op 0.35.

[2] NB: in een eerdere versie van dit memo werd ook locatie Hellevoetsluis gebruikt. Deze is verwijderd omdat hier geen betrouwbare randvoorwaarden konden worden uitgerekend, bovendien bleek uit de afgeleide randvoorwaarden dat er hier sprake is van sterke wind-waterstandscorrelatie. De gedane analyse voor enkel golfklap, met gras op het hele talud, snijdt dan ook geen hout. Dit wordt bevestigd door het feit dat in werkelijkheid op deze locatie tot op redelijke hoogte een steenbekleding is aangelegd. De locatie bij Moerdijk is twijfelachtig qua resultaat, maar hier is wel een redelijk logische marginale verdeling gevonden voor waterstand en golfhoogte. Daarom is deze locatie toch gehandhaafd, dit betreft een andere oeverlocatie dan in een eerdere versie van dit memo.

- De golfperiode is een determinist en is afgeleid op basis van de resulterende golfsteilheden in de Q-variant. Deze is voor alle locaties in het rivierengebied, voor alle waterstands-golfcombinaties ongeveer 0.04.

3 Resultaten GEBU

3.1 Vergelijking eenvoudige en gedetailleerde toets op kustlocaties

3.1.1 Aanpak en resultaten

Allereerst is voor de cases uit Klein Breteler et al. (2016) het resultaat voor de eenvoudige toets bepaald. Hieruit volgt dat, doordat golfhoogtes allemaal hoger zijn dan 0.6 meter en het belastingniveau ver boven het niveau h(P_T) ligt geen goedkeuren in de eenvoudige toets plaatsvindt, zie Tabel 3.1.

Tabel 3.1 Resultaten eenvoudige toetsing voor mechanisme GEBU

Locatie	**Niveau gras** *(in m NAP)*	**Belastingniveau** *(in m NAP)*	**E.1** *Belastingniveau <0.1 l/s/m*	**E.2** *Hm0<0.6 en kleikern*	**E.3** *Open of Gesloten*	**E.4** *Hm0<0.25*
ED1	6	8.40	naar E.2	naar E.3	naar E.4	naar laag 2a
ED2	6.3	9.07	naar E.2	naar E.3	naar E.4	naar laag 2a
ED3	6.55	9.79	naar E.2	naar E.3	naar E.4	naar laag 2a
ED4	6.4	10.49	naar E.2	naar E.3	naar E.4	naar laag 2a
ED5	6.4	11.24	naar E.2	naar E.3	naar E.4	naar laag 2a
HA1	6	14.12	naar E.2	naar E.3	naar E.4	naar laag 2a
HA2	5.3	9.89	naar E.2	naar E.3	naar E.4	naar laag 2a
HA3	5.39	10.94	naar E.2	naar E.3	naar E.4	naar laag 2a
HA4	6	11.97	naar E.2	naar E.3	naar E.4	naar laag 2a
HA5	6	13.06	naar E.2	naar E.3	naar E.4	naar laag 2a
LM	6	12.88	naar E.2	naar E.3	naar E.4	naar laag 2a
OS	6.6	12.91	naar E.2	naar E.3	naar E.4	naar laag 2a
VL1	6	14.88	naar E.2	naar E.3	naar E.4	naar laag 2a
VL2	6	15.81	naar E.2	naar E.3	naar E.4	naar laag 2a

Wanneer we vervolgens kijken naar de gedetailleerde toets op oploop geeft dat de resultaten uit

Tabel 3.2, voor de gedetailleerde toets op klap zijn de resultaten te zien in Tabel 3.3. Voor gevallen waarbij geldt dat het niveau van de overgang hoger ligt dan het niveau horend bij de doorsnede-eis (dus h_{gras}>h_{PT}) zijn golfklapberekeningen uitgevoerd. Op basis van deze resultaten is er geen reden om aan te nemen dat de eenvoudige toets bij windgedomineerde locaties tot foutief goedkeuren kan leiden. De toetsing op het belastingniveau (stap E.1) is zeer conservatief en golfhoogtes zijn zonder uitzondering veel hoger dan de vereiste 0,25 m bij een dijk zonder kleikern.

Tabel 3.2 Resultaten gedetailleerde toets op oploop

Locatie	D (m2/s2)	Goed?
ED1	0	Voldoet
ED2	0	Voldoet
ED3	0.37	Voldoet
ED4	10	Voldoet
ED5	64	Voldoet
HA1	3911	Voldoet
HA2	29	Voldoet
HA3	220	Voldoet
HA4	752	Voldoet
HA5	1978	Voldoet
LM	660	Voldoet
OS	878	Voldoet
VL1	19200	naar laag 3
VL2	24317	naar laag 3

Tabel 3.3 Resultaten gedetailleerde toets op golfklap

Locatie	Gedetailleerde toets totaal	h_{PT}	Belasting-niveau>h_{PT}?	h_{gras}>h_{PT}	FofS	Gedet. toets klap
ED1	Voldoet	6.61	Ja	som maken	1.695	Voldoet
ED2	naar laag 3	6.61	Ja	som maken	0.45597	Naar laag 3
ED3	naar laag 3	6.61	Ja	som maken	0.94575	Naar laag 3
ED4	naar laag 3	6.61	Ja	som maken	0.369	Naar laag 3
ED5	naar laag 3	6.61	Ja	som maken	0.315	Naar laag 3
HA1	Voldoet	5.4	Ja	Voldoet		Voldoet
HA2	naar laag 3	5.4	Ja	som maken	0.622	Naar laag 3
HA3	Voldoet	5.4	Ja	som maken	2.245	Voldoet
HA4	Voldoet	5.4	Ja	Voldoet		Voldoet
HA5	Voldoet	5.4	Ja	Voldoet		Voldoet
LM	Voldoet	5.62	Ja	Voldoet		Voldoet
OS	naar laag 3	6.8	Ja	som maken	0.247	Naar laag 3
VL1	naar laag 3	5.9	Ja	Voldoet		Voldoet
VL2	naar laag 3	6.14	Ja	som maken	0.22	Naar laag 3

3.1.2 Conclusie eenvoudige toets GEBU kust

Voor kustlocaties is de eenvoudige toets voor GEBU voldoende veilig. Het is zelfs de vraag hoe relevant met name toetsstap E.1 is, aangezien deze op alle locaties waarden geeft die ver boven het niveau h_{PT} liggen, vooral voor locaties waar golfklap dominant is.

3.2 Vergelijking eenvoudige en gedetailleerde toets in het rivierengebied

3.2.1 Aanpak

Voor het rivierengebied is een vergelijkbare analyse uitgevoerd. Ook voor het rivierengebied zijn probabilistische berekeningen uitgevoerd. Deze komen in een volgende paragraaf ter sprake. Hier wordt gefocust op de consistentie tussen de eenvoudige en gedetailleerde toets.
Zoals eerder in de paragraaf over de Q-variant (2.3.2) is opgemerkt is ook van belang welke waterstand als invoer wordt gegeven voor de randvoorwaardencombinatie die gebruikt wordt in de gedetailleerde toets. Hiervoor zijn hier de volgende 3 overschrijdingskansen van de waterstand gekozen, welke vervolgens zijn afgeleid uit de marginale statistiek:

- 1/10
- P_{norm}
- P_T

Opgemerkt dient te worden dat ten tijde van het uitvoeren van deze berekeningen de Q-variant nog niet volledig uitontwikkeld was. Derhalve kunnen in de uiteindelijke variant resultaten per locatie verschillen. In het algemeen is er echter een dekkend beeld van verschillende combinaties van randvoorwaarden ontstaan, wat voor deze studie voldoende is.

3.2.2 Resultaten eenvoudige toets

Allereerst is gekeken naar de resultaten van de eenvoudige toets. De resultaten zijn weergegeven in Tabel 3.4. Voor toetsstap E.1 wordt in geen geval goedgekeurd. Toetsstap E.2 is niet relevant, want er is aangenomen dat er nergens een kleikern aanwezig is. Wederom is de hoogte van het belastingniveau voor alle locaties zeer hoog. Dit ligt in alle gevallen ruim boven de kruinhoogte van de dijk.

Als tweede stap wordt de gedetailleerde toets uitgevoerd. Dit wordt gedaan voor 3 combinaties van waterstand en golfhoogte en voor stormduren van 6 en 12 uur. Resultaten zijn weergegeven in Tabel 3.5 voor een stormduur van 12 uur. In Bijlage A staat dezelfde tabel voor een stormduur van 24 uur, maar hiervoor zijn de toetsoordelen hetzelfde. Uit de tabel is te zien dat in geen enkel geval de eenvoudige toets positiever is dan de gedetailleerde toets. De 3 ‘toetsingen’ in de kolommen naast elkaar zijn uitgevoerd met de waterstands-golfcombinatie bij overschrijdingskans van de waterstand als weergegeven in de bovenste rij (respectievelijk 1/10, P_{norm} en P_T). De verschillen tussen de 3 toetsingen zijn voor het uiteindelijke toetsvoorschrift voor GEBU interessant, hier wordt in een volgende paragraaf op ingegaan.

Tabel 3.4 Resultaten voor eenvoudige toets GEBU voor locaties in boven- en benedenrivierengebied

Locatie	**Lobith**	**Wageningen**	**Doesburg**	**Zutphen**	**Tiel**	**Terwolde**	**Cabauw**	**Gorinchem**	**Moerdijk**
Waterstand bij norm ***(in m NAP)***	17.75	11.87	11.04	9.05	11.29	6.87	4.85	5.75	2.7
Hm0 ***(in m)***	1.88	0.97	1.49	1.09	1.37	0.8	1.4	1.31	0.83
Niveau gras ***(in m NAP)***	15.265	8.915	10.145	7.799	8.962	7.4	3.188	3.569	3
Kleikern ***(Ja/Nee)***	Nee	Nee	Nee	Nee	Nee	Nee	Nee	Nee	Nee

Zode *(Open/Gesl)*	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl
Belastingniveau *(in m NAP)*	24.77	15.12	16.38	12.78	16.16	9.46	9.82	10.34	4.74
E.1 *Belasting-niveau <0.1 l/s/m*	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2
E.2 *Hm0<0.6 en kleikern*	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3
E.3 *Open of Gesloten*	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4
E.4 *Hm0<0.25*	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a

Tabel 3.5 Resultaten gedetailleerde toets GEBU - golfklap voor verschillende randvoorwaardencombinaties uit de Q-variant, voor een stormduur van 6 uur[3] bij een constant peil

t = 6 uur	**1/10**				**P_{norm}**				**P_T**			
Locatie	h	H_s	FofS	V/NV	h	H_s	FofS	V/NV	h	H_s	FofS	V/NV
Lobith	15.3	1.4	0.27	NV	17.8	0.6	8.92	V	18.3	0.3	10	V
Wageningen	8.9	0.9	2.56	V	11.9	0.6	6.65	V	12.2	0.4	10	V
Doesburg	10.2	1.1	1.45	V	11	0.7	5.45	V	11.7	0.3	10	V
Zutphen	7.8	0.8	3.26	V	9.1	0.5	10	V	9.6	0.3	10	V
Tiel	9	1.1	0.83	NV	11.3	0.6	8.25	V	11.8	0.3	10	V
Terwolde	5.8	0.6	6.7	V	6.9	0.4	10	V	7.5	0.2	10	V
Cabauw	3.2	1.5	0.23	NV	4.9	0.8	3.73	V	5.3	0.4	10	V
Gorinchem	3.6	1.2	0.41	NV	5.8	0.7	5.24	V	6.1	0.4	10	V
Moerdijk	nb	nb	nb	nb	nb	nb	nb	nb	4.2	0.0	nb	V

Tabel 3.6 Resultaten gedetailleerde toets GEBU - golfklap voor verschillende randvoorwaardencombinaties uit de Q-variant, voor een stormduur van 12 uur bij een constant peil

t = 12 uur	**1/10**				**P_{norm}**				**P_T**			
Locatie	h	H_s	FofS	V/NV	h	H_s	FofS	V/NV	h	H_s	FofS	V/NV
Lobith	15.3	1.4	0.14	NV	17.8	0.6	4.49	V	18.3	0.3	10	V
Wageningen	8.9	0.9	1.29	V	11.9	0.6	3.35	V	12.2	0.4	10	V
Doesburg	10.2	1.1	0.34	NV	11.0	0.7	2.75	V	11.7	0.3	10	V
Zutphen	7.8	0.8	1.64	V	9.1	0.5	10	V	9.6	0.3	10	V
Tiel	9.0	1.1	0.27	NV	11.3	0.6	4.16	V	11.8	0.3	10	V
Terwolde	5.8	0.6	3.40	V	6.9	0.4	10	V	7.5	0.2	10	V
Cabauw	3.2	1.5	0.12	NV	4.9	0.8	1.88	V	5.3	0.4	10	V
Gorinchem	3.6	1.2	0.19	NV	5.8	0.7	2.64	V	6.1	0.4	10	V
Moerdijk	nb	nb	nb	nb	nb	nb	nb	nb	4.2	0.0	nb	V

[3] Voor locatie Moerdijk geeft de Q-variant niet alle waterstanden resultaten.

3.2.3 Conclusie eenvoudige toets GEBU rivieren

Net als voor kustlocaties is voor rivierenlocatiesde eenvoudige toets voor GEBU voldoende veilig bij alle onderzochte varianten van de gedetailleerde toets. Ook hier geldt dat het de vraag is hoe relevant toetsstap E.1 is, aangezien deze op alle locaties waarden geeft die vrijwel zeker ver boven het kruinniveau liggen. In het algemeen is er echter geen reden om de eenvoudige toets aan te passen omdat deze minder veilig zou zijn dan de gedetailleerde toets.

3.3 Analyses t.b.v. gedetailleerde toets GEBU in het rivierengebied

3.3.1 Opzet berekeningen

De resultaten voor verschillende randvoorwaardencombinaties uit de Q-variant in

Locatie	**Lobith**	**Wageningen**	**Doesburg**	**Zutphen**	**Tiel**	**Terwolde**	**Cabauw**	**Gorinchem**	**Moerdijk**
Waterstand bij norm ***(in m NAP)***	17.75	11.87	11.04	9.05	11.29	6.87	4.85	5.75	2.7
Hm0 ***(in m)***	1.88	0.97	1.49	1.09	1.37	0.8	1.4	1.31	0.83
Niveau gras ***(in m NAP)***	15.265	8.915	10.145	7.799	8.962	7.4	3.188	3.569	3
Kleikern ***(Ja/Nee)***	Nee	Nee	Nee	Nee	Nee	Nee	Nee	Nee	Nee
Zode ***(Open/Gesl)***	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl	Gesl
Belastingniveau ***(in m NAP)***	24.77	15.12	16.38	12.78	16.16	9.46	9.82	10.34	4.74
E.1 ***Belasting-niveau <0.1 l/s/m***	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2	naar E.2
E.2 ***Hm0<0.6 en kleikern***	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3	naar E.3
E.3 ***Open of Gesloten***	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4	naar E.4
E.4 ***Hm0<0.25***	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a	naar laag 2a

Tabel 3.5 maken duidelijk dat de gekozen waterstand bij de randvoorwaardenberekening in de Q-variant zeer veel invloed heeft. Daarom is voor alle cases een aantal probabilistische berekeningen gemaakt om de faalkansen te berekenen voor de verschillende locaties. Zo kan worden bepaald hoe conservatief de toetsing met verschillende randvoorwaardencombinaties is. In de probabilistische berekeningen is een aantal zaken gevarieerd, namelijk:

- Stormduur is aangenomen als ofwel 6 ofwel 12 uur. Dit zijn realistische stormduren voor boven- en benedenrivierengebied die in het verleden ook zijn gehanteerd in Hydra-Zoet.
- Het onderste niveau van de grasbekleding is aangenomen op 0 meter NAP, dus in feite een oneindig talud met gras, dit is ‘Talud A’. Er is ook een ‘Talud B’ waarvoor geldt dat het talud begint op het niveau F_h^{-1}(1-1/10). Dit is in de toetsvoorschriften aangemerkt als het niveau waaronder de veiligheidstoetsing niet meer van toepassing is.

- Voor de locaties in het benedenrivierengebied zijn cases met correlatiecoëfficiënt 0 en 1 doorgerekend. Dit is een boven- ondergrensbenadering en in werkelijkheid zal de waarheid ergens daartussen liggen.

3.3.2 Resultaten

In

Tabel 3.8 zijn de resultaten weergegeven voor een stormduur van 6 uur. In groen is aangegeven waar is goedgekeurd, in rood waar is afgekeurd. Bij de faalkansberekeningen is ook aangegeven waar is afgekeurd maar waar de faalkans maximaal een factor 10 groter is dan de doelkans. Te zien is dat de faalkansen, zoals verwacht iets hoger zijn voor Talud A dan voor Talud B[4]. Wanneer een randvoorwaardencombinatie gekozen zou moeten worden die leidt tot een consistent resultaat tussen probabilistische berekening en gedetailleerde toets zou de keuze in principe op de combinatie komen met een waterstand op het niveau $F_h^{-1}(1-1/10)$. Deze is voor beide taluds licht conservatief, met name bij Gorinchem.

NB: in een eerdere conceptversie van dit memo was ook een locatie bij Hellevoetsluis opgenomen. Dit is echter een case met behoorlijke correlatie waardoor de gedane analyse geen hout snijdt en zeer hoge faalkansen oplevert. Omdat er in de praktijk ook een steenbekleding op het talud ligt zou ook op oploop moeten worden getoetst. Daarom is de gedane analyse niet correct en is deze case uit de dataset verwijderd.

Tabel 3.7 Probabilistische berekeningen GEBU - golfklap met een stormduur van 6 uur

			Gedetailleerde toets			**Faalkansberekening**	
Locatie	**Pt**	**Correlatie**	***1/10***	***P_{norm}***	***P_T***	***Talud A***	***Talud B***
Lobith	1.52E-06	0	NV	V	V	7.07E-05	3.67E-05
Wageningen	4.55E-07	0	V	V	V	5.33E-09	3.21E-09
Doesburg	4.55E-05	0	V	V	V	7.94E-05	5.33E-05
Zutphen	4.55E-06	0	V	V	V	3.70E-07	3.57E-07
Tiel	1.52E-06	0	NV	V	V	4.66E-06	3.07E-06
Terwolde	1.52E-05	0	V	V	V	1.66E-08	9.72E-09
Cabauw	1.52E-06	0	NV	V	V	6.45E-06	8.15E-06
	1.52E-06	1	NV	V	V	1.02E-04	8.27E-05
Gorinchem	4.55E-07	0	NV	V	V	1.22E-07	8.52E-08
	4.55E-07	1	NV	V	V	1.68E-06	1.07E-06
Moerdijk	2.27E-05	0	nb	nb	V	4.30E-08	4.63E-10
	2.27E-05	1	nb	nb	V	7.72E-07	9.73E-7

[4] NB: in enkele gevallen is dit niet het geval, hier is het verschil tussen beiden echter erg klein en is komt door de nauwkeurigheid van de probabilistische berekeningen de faalkans voor talud A iets lager uit dan voor talud B. Dit valt echter in alle gevallen ruimschoots binnen de te verwachten nauwkeurigheid van probabilistische berekeningen met betrouwbaarheidsindices in de orde van 5.

Tabel 3.8 Probabilistische berekeningen GEBU - golfklap met een stormduur van 12 uur

Locatie	Pt	Correlatie	Gedetailleerde toets			Faalkansberekening	
			1/10	P_{norm}	P_T	*Talud A*	*Talud B*
Lobith	1.52E-06	0	NV	V	V	1.15E-04	6.68E-05
Wageningen	4.55E-07	0	V	V	V	3.06E-08	9.29E-09
Doesburg	4.55E-05	0	NV	V	V	1.41E-04	9.70E-05
Zutphen	4.55E-06	0	V	V	V	1.96E-06	1.82E-06
Tiel	1.52E-06	0	NV	V	V	1.55E-05	6.82E-06
Terwolde	1.52E-05	0	V	V	V	5.49E-08	4.14E-08
Cabauw	1.52E-06	0	NV	V	V	2.27E-05	1.20E-05
	1.52E-06	1	NV	V	V	6.07E-04	3.73E-04
Gorinchem	4.55E-07	0	NV	V	V	2.51E-07	2.40E-07
	4.55E-07	1	NV	V	V	6.77E-06	6.03E-06
Moerdijk	2.27E-05	0	V	V	V	2.35E-07	2.02E-09
	2.27E-05	1	V	V	V	7.58E-06	4.85E-06

Eenzelfde analyse is gedaan voor een stormduur van 12 uur, resultaten zijn weergegeven in

Tabel 3.8. Ook hier geldt dat de toets met randvoorwaarden met waterstand $F_h^{-1}(1\text{-}1/10)$ in de Q-variant het meest passend is.

Uit bovenstaande tabellen blijkt dat met het standtijdlijnenmodel Golfklap op het buitentalud van grasbekledingen op behoorlijk veel van de gekozen locaties leidt tot afkeuren, met de kanttekening dat deze locaties zijn uitgekozen op het vermoeden van een relevante golfbelasting.

3.3.3 Conclusie t.a.v. gedetailleerde toets GEBU in het rivierengebied

Op basis van de uitgevoerde berekeningen lijkt golfklap in het rivierengebied op redelijk grote schaal tot afkeuren te kunnen leiden, hoewel daaraan toegevoegd moet worden dat er een relatief ‘ongunstige’ set locaties is gekozen. Hoe dan ook is de vraag hoe realistisch dit afkeuren is.
Uit de sommen blijkt dat de ondergrens van het beschouwde talud behoorlijk wat invloed heeft. Vraag is of deze nog op het juiste niveau ligt (1/10), zeker gezien de toegenomen normkansen in het rivierengebied zou dat aanleiding kunnen zijn deze verder omhoog te schuiven, naar bijvoorbeeld een 1/100 niveau of een niveau afhankelijk van de norm. Daarnaast zou gekeken kunnen worden of de huidige reststerkte aanname zo laag op het talud niet erg conservatief is, aangezien het onwaarschijnlijk lijkt dat golfklappen op gras op dergelijke lage niveaus langs het talud tot doorgaand falen leidt.

4 Beoordeling eenvoudige toets GEKB

Voor de beoordeling van de eenvoudige toets van GEKB wordt dezelfde aanpak gehanteerd. Hierbij wordt de kruinhoogte zo aangenomen dat in de gedetailleerde toets precies afgekeurd wordt. Wanneer dit gecombineerd is met een positief toetsoordeel in de eenvoudige toets is de eenvoudige toets niet voldoende veilig.

De locaties wijken iets af van het eerdere voorstel. Tabel 4.1 geeft invoerparameters,

Tabel 4.2 geeft resultaten voor de eenvoudige toets. De benodigde kruinhoogtes in tabel 4.2 zijn dusdanig bepaald dat deze op het laagste niveau liggen waarop volgens de probabilistische berekeningen afkeuren kan plaatsvinden. In sommige gevallen zijn 2 cases doorgerekend, wanneer de verwachting was dat deze problemen op konden leveren (d.w.z. onveilige eenvoudige toets). In zulke gevallen is de kruin zo gekozen dat in de ene case precies wordt goedgekeurd en de andere precies wordt afgekeurd. Als invoerparameters voor de laag 2b berekeningen is uitgegaan van profielen uit Hydra-Zoet en verder default verdelingen voor sterkte parameters en modelfactoren zoals gedefinieerd in de technische documentatie van Hydra-Ring. Opgemerkt moet worden dat talud en kleilaagdikte in de eenvoudige toetsstap zo zijn gekozen dat de kans dat goedgekeurd wordt maximaal is.

Tabel 4.1 Gebruikte invoerparameters.

Locatie	**Talud**	**Norm** *(-/jaar)*	**N**	**Waterstand bij norm** *(in m NAP)*	**Hm0** *(in m)*	**Kruin-hoogte** *(in m NAP)*	**Kleilaag-dikte**	**Zode**
Doesburg	4	1000	1	11.04	1.49	11.6	0.4	Gesloten
Lobith	4	30000	1	17.75	1.88	18.3	0.4	Gesloten
Tiel	4	30000	1	11.29	1.37	11.9	0.4	Gesloten
Wageningen	4	100000	1	11.87	0.97	12.3	0.4	Gesloten
Zutphen	4	10000	1	9.05	1.09	9.6	0.4	Gesloten
Cabauw	4	30000	1	4.85	1.40	5.3	0.4	Gesloten
Gorinchem	4	100000	1	5.75	1.31	6.4	0.4	Gesloten
Hellevoetsluis	4	1000	2	2.44	2.10	3.6	0.4	Gesloten
Breskens	4	1000	2	4.96	3.3	8.4	0.4	Gesloten
Maeslantkering_1	2	1000	2	5.091	0.75	6.7	0.4	Gesloten
Maeslantkering_2	2	1000	2	5.091	0.75	6.8	0.4	Gesloten
Kloosterzande	4	1000	2	5.886	1.6	6.7	0.4	Gesloten
Harlingen	4	3000	3	4.904	3.3	12	0.4	Gesloten
Oosterend Texel	4	1000	2	4.06	2	5	0.4	Gesloten
Rotterdamse Hoek	4	3000	3	1.15	3.3	7.6	0.4	Gesloten
Cornwerd_1	2	1000	3	1.565	1.30	4.6	0.4	Gesloten
Cornwerd_2	2	1000	3	1.565	1.30	5	0.4	Gesloten
Hondsbossche noord	4	3000	3	4.495	5.75	16.7	0.4	Gesloten
Hondsbossche zuid	4	3000	3	4.52	5	16.8	0.4	Gesloten
Vlissingen	4	100000	2	6.149	*3.35*	9.5	0.4	Gesloten

Tabel 4.2 Resultaten voor de eenvoudige toets voor GEKB

Locatie	**Overslag-debiet**	**E.1 *Overslagdebiet <0.1 l/s/m***	**E.2 *Kwaliteit zode***	**E.3 *Sterktecrit q<1 l/s/m***	**E.4 Sterktecrit q<5 l/s/m**
Doesburg	427.12	naar E.2	naar E.3	naar E.4	naar laag 2
Lobith	756.72	naar E.2	naar E.3	naar E.4	naar laag 2
Tiel	317.23	naar E.2	naar E.3	naar E.4	naar laag 2
Wageningen	189.29	naar E.2	naar E.3	naar E.4	naar laag 2
Zutphen	192.86	naar E.2	naar E.3	naar E.4	naar laag 2
Cabauw	448.44	naar E.2	naar E.3	naar E.4	naar laag 2
Gorinchem	255.97	naar E.2	naar E.3	naar E.4	naar laag 2
Hellevoetsluis	453.38	naar E.2	naar E.3	naar E.4	naar laag 2
Breskens	249.78	naar E.2	naar E.3	naar E.4	naar laag 2
Maeslantkering_1	1.54	naar E.2	naar E.3	naar E.4	Voldoet
Maeslantkering_2	1.09	naar E.2	naar E.3	naar E.4	Voldoet
Kloosterzande	337.74	naar E.2	naar E.3	naar E.4	naar laag 2
Harlingen	14.01	naar E.2	naar E.3	naar E.4	naar laag 2
Oosterend Texel	522.04	naar E.2	naar E.3	naar E.4	naar laag 2
Rotterdamse Hoek	23.31	naar E.2	naar E.3	naar E.4	naar laag 2
Cornwerd_1	2.15	naar E.2	naar E.3	Voldoet	Voldoet
Cornwerd_2	0.96	naar E.2	naar E.3	naar E.4	Voldoet
Hondsbossche noord	34.64	naar E.2	naar E.3	naar E.4	naar laag 2
Hondsbossche zuid	11.81	naar E.2	naar E.3	naar E.4	naar laag 2
Vlissingen	285.06	naar E.2	naar E.3	naar E.4	naar laag 2

Uit de resultaten blijkt dat de eenvoudige toets meestal tot toetslaag 2leidt, omdat de berekende overslagdebieten hoog zijn. Tabel 4.3 geeft de vergelijking met de gedetailleerde toets.

Tabel 4.3 Vergelijking Eenvoudige en gedetailleerde toets

Locatie	**Eenvoudige toets**	**Doorsnede-eis (-/jaar)**	**Golfklasse**	**Faalkans laag 2**	**Goed in laag 2?**
Doesburg	naar 2b	2.40E-04	1-2m	2.30E-04	Voldoet
Lobith	naar 2b	8.00E-06	1-2m	6.89E-06	Voldoet
Tiel	naar 2b	8.00E-06	1-2m	5.99E-06	Voldoet
Wageningen	naar 2b	2.40E-06	0-1m	2.11E-06	Voldoet
Zutphen	naar 2b	2.40E-05	1-2m	1.81E-05	Voldoet
Cabauw	naar 2b	8.00E-06	1-2m	6.61E-06	Voldoet
Gorinchem	naar 2b	2.40E-06	1-2m	1.39E-06	Voldoet
Hellevoetsluis	naar 2b	1.20E-04	2-3m	8.13E-05	Voldoet
Breskens	naar 2b	1.20E-04	Golf te hoog	6.84E-05	naar laag 3
Maeslantkering_1	Voldoet	1.20E-04	0-1m	1.34E-04	naar laag 3

Maeslantkering_2	Voldoet	1.20E-04	0-1m	9.90E-05	Voldoet
Kloosterzande	naar 2b	1.20E-04	1-2m	7.25E-05	Voldoet
Harlingen	naar 2b	2.67E-05	Golf te hoog	2.63E-05	naar laag 3
Oosterend Texel	naar 2b	1.20E-04	2-3m	5.57E-05	Voldoet
Rotterdamse Hoek	naar 2b	2.67E-05	Golf te hoog	2.42E-05	naar laag 3
Cornwerd_1	Voldoet	8.00E-05	1-2m	1.49E-04	naar laag 3
Cornwerd_2	Voldoet	8.00E-05	1-2m	7.84E-05	Voldoet
Hondsbossche noord	naar 2b	2.67E-05	Golf te hoog	6.04E-05	naar laag 3
Hondsbossche zuid	naar 2b	2.67E-05	Golf te hoog	2.39E-05	naar laag 3
Vlissingen	naar 2b	1.20E-06	Golf te hoog	1.16E-06	naar laag 3

Te zien is dat er voor meerdere locaties, namelijk Cornwerd en Maeslantkering inconsistenties zijn tussen eenvoudige en gedetailleerde toets. Omdat een eenvoudige toets gegarandeerd veilig moet zijn is een dergelijk resultaat niet acceptabel.

5 Conclusie

In dit memo zijn de resultaten van de eenvoudige toets voor de mechanismen GEBU en GEKB vergeleken met de resultaten van de gedetailleerde toets.

Voor GEBU blijkt met name de eerste stap uit de eenvoudige toets dermate conservatief (door de zeer conservatieve berekening van het overslagdebiet), dat deze op geen enkele manier tot goedkeuren leidt. Dit betekent overigens wel dat deze stap consistent is met de gedetailleerde toets. Inhoudelijk gezien verdient een variant waarbij bijvoorbeeld marginale statistiek wordt afgeleid voor de doorsnede-eis, of gebruik gemaakt wordt van het HBN de voorkeur voor deze stap, dit zijn echter berekeningen die niet gefaciliteerd worden binnen de eenvoudige toets. Inhoudelijk is er dus geen bezwaar om de eenvoudige toets te handhaven, zeker aangezien de andere toetsstappen mogelijk wel tot goedkeuren kunnen leiden (H_{m0}<0,6 m en kleikern).

De eenvoudige toets voor GEKB is bij locaties Cornwerd en Maeslantkering een inconsistentie gevonden. Aangezien de eenvoudige toets ‘zeker veilig’ moet zijn is dit niet acceptabel. Daarom wordt geadviseerd de eenvoudige toets voor GEKB te schrappen.

6 Aanbevelingen t.a.v. gedetailleerde toets GEBU in WBI

6.1 Inleiding

In de afgelopen maanden is een aantal onderzoeken en berekeningen gedaan die bijdragen aan het totaalbeeld van de toetsing (en ontwerp) van grasbekledingen. In dit hoofdstuk worden deze afwegingen op een rij gezet en wordt aan de hand hiervan een advies gegeven voor verdere implementatie in het WBI-2017. De aspecten die hierbij een rol spelen zijn:

- Uitkomsten uit OI
- Uitkomsten verschillende WBI (kalibratie)studies
- Implementatie in de software

Hierbij zijn de belangrijkste aspecten:

- Op welk niveau wordt getoetst?
- Welke randvoorwaarden worden gehanteerd in de semi-probabilistische toets?

Er zijn twee niveaus die zeer belangrijk zijn in deze paragraaf: het niveau h_{PT}, ofwel het niveau van de waterstand met overschrijdingskans gelijk aan de doorsnede-eis (dus $F_h^{-1}(1-P_T)$), en het niveau h_{Pmax}, het niveau van de waterstand met overschrijdingskans gelijk aan de trajectkans (dus $F_h^{-1}(1-P_{max})$).

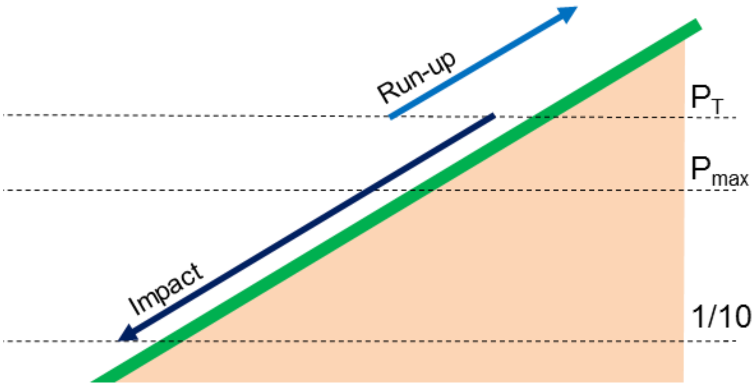
6.2 Op welk niveau wordt getoetst?

Voorheen gold toetspeil als scheiding tussen golfklap- en golfoploopzone. Uit de eerste versies van de kalibratiestudie bleek het door de geringe sterkte van gras tegen golfklappen onmogelijk om dit te handhaven en aan de doorsnede-eisen te voldoen. Inhoudelijk ideaal zou zijn om cumulatieve oploop en klapbelastingen op te tellen tot 1 som, voor deze optelling is echter geen inhoudelijke onderbouwing wat deze optie onmogelijk maakt. Beide mechanismen zullen dus apart moeten worden beschouwd.

In voorgaande WTI-versies werd aangenomen dat klap ten allen tijde dominant is boven golfoploop. Dit houdt in dat gegarandeerd moet worden dat wanneer voor een gegeven talud goedgekeurd wordt op golfklap, ook voor golfoploop goedgekeurd wordt. In alle sommen van WBI en OI is dit het geval, in enkele cases in de OI-studie van Klein Breteler et al. (2016) worden wel significant grotere faalkansen gevonden voor oploop dan voor klap, maar dit betreft cases met een zeer lange belastingduur, die voor ontwerp interessant kunnen zijn (i.v.m. voorkomen van grootschalig terugkerend onderhoud) maar voor de veiligheidstoetsing niet. De dominantie van golfklap boven golfoploop geldt dus nog steeds. Een toetsing op klap geeft dan voldoende zekerheid dat de doorsnede ook op oploop goedgekeurd kan worden.

Omdat zoals gezegd de grens tussen golfklap en golfoploopzone niet op h_{Pmax} gehandhaafd kan worden moet deze verschoven worden. Uit de analyses uit OI, WBI en dit memo blijkt dat het niveau h_{PT} een geschikte grens is. Hierbij moet opgemerkt worden dat deze grens een grens is die volgt uit de randvoorwaarden van de toetsing: namelijk dat met toetsen van 1 mechanisme voldoende veiligheid kan worden gegarandeerd voor beiden. Deze grens betekent niet dat fysisch gezien golfklap boven deze grens en golfoploop onder deze grens geen rol spelen. Figuur 6.1 geeft de opzet van de toetsing zoals hierboven beschreven weer.
Hierbij is een ondergrens aangegeven corresponderend met het waterstandsniveau met overschrijdingskans 1/10. Bekledingen die hieronder liggen vallen onder dagelijks beheer, met name omdat hier vaak andere belastingen, bijvoorbeeld scheepsgolven, maatgevend zijn. Bovendien is de kans dat erosie op dit niveau leidt tot doorgaand falen en een overstroming

nihil. Wellicht dat dit niveau zelfs nog naar boven kan worden bijgesteld omdat de hoeveelheid reststerkte op dit niveau veel groter is dan meegenomen[5].



Figuur 6.1 Golfklap en -oploopzone in de gedetailleerde toets.

6.3 Welke randvoorwaarden worden gehanteerd in de semi-probabilistische toets?

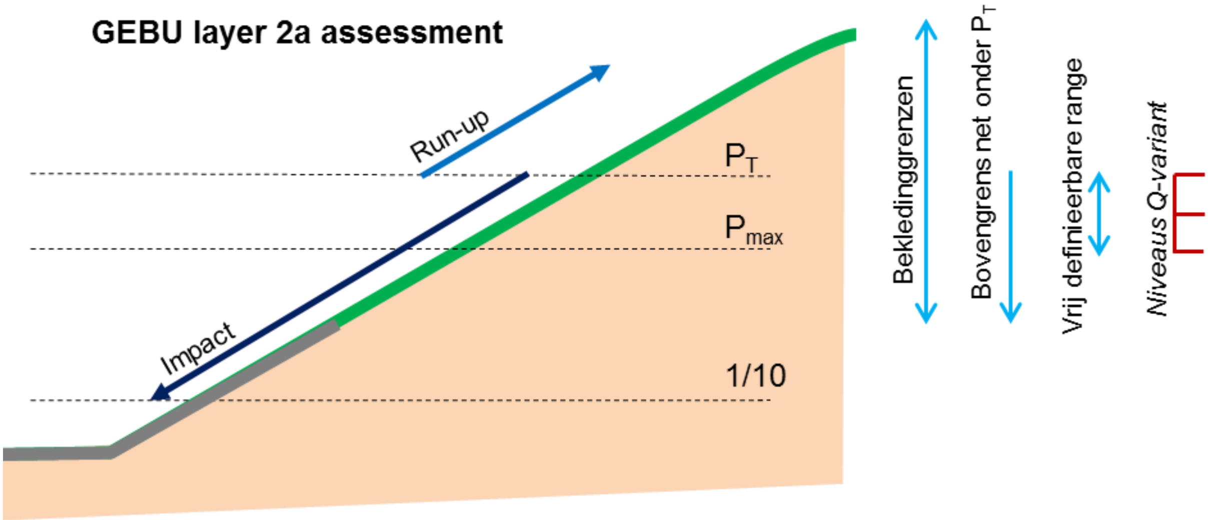
In het veiligheidsformat van oploop is in de versie van december 2015 opgenomen dat wordt getoetst met randvoorwaarden met overschrijdingskans gelijk aan de trajectnorm P_{max}. Hiervoor is ook een kalibratie uitgevoerd. Voor golfklap was deze kalibratie niet mogelijk omdat toetsen met randvoorwaarden P_{max} kan leiden tot ‘false positives’, namelijk wanneer de grasbekleding begint boven P_{max}, maar onder P_T. Door de geringe sterkte van gras is de faalkans voor golfklap in windgedomineerde gebieden namelijk goed te benaderen met de overschrijdingskans van de waterstand. Deze is in zulke gevallen groter dan P_T en dus te groot. Om dit te voorkomen moeten dus randvoorwaarden met overschrijdingskans P_T worden gebruikt.

Om consistent te zijn is in september 2016 voorgesteld om ook voor golfoploop te toetsen met randvoorwaarden met overschrijdingskans P_T, dit is ook voor het gebruik van de software handiger omdat niet twee sets bekledingsrandvoorwaarden hoeven te worden afgeleid met de Q-variant (voor overschrijdingskans P_{max} en P_T). De kalibratie hiervoor wordt in het nog op te leveren kalibratierapport meegenomen.

In dit memo en in de OI-studie van Klein Breteler (2016) is uitgebreid ingegaan op het toetsen van grasbekledingen op niet windgedomineerde locaties. Dit betreft dan locaties waar meerpeil of rivierafvoer belangrijke stochasten zijn die niet gecorreleerd zijn met de wind. Concreet betekent dit dat de golfhoogte afneemt hoger op het talud, en dus de lagere taluddelen in termen van golfbelasting maatgevend zijn. Daarom is het niet voldoende om enkel de golfbelasting bij waterstand met overschrijdingskans P_T door te rekenen, maar moet gekeken worden naar een set van belastingcombinaties met overschrijdingskans P_T. In Ringtoets is dit ondervangen als in Figuur 6.2. Er zijn 5 grenzen, van de bovengrenzen wordt de laagste genomen, van de ondergrenzen de bovenste.
De eerste set zijn de grenzen van de bekleding. Hiervan zal de ondergrens corresponderen met het laagste punt van de grasbekleding, de bovengrens in principe met de kruin. Deze laatste is dus zelden relevant. De tweede bovengrens is het niveau net onder P_T. De derde set is een vrij definieerbare range, waarbij het zoekgebied op basis van gebiedskennis kan worden

[5] NB: in de reststerktemodule van de Golfklap-kernel wordt een maximale kleilaagdikte van 1 meter aangenomen. Zeker voor plaatsen lager op het talud is dit waarschijnlijk tamelijk conservatief.

verkleind. Vervolgens bepaalt Ringtoets, aan de hand van een opgegeven stapgrootte, default 0,1 m, de combinaties van waterstand en golfparameters op de verschillende niveaus, binnen de aangegeven range. Voor al deze niveaus wordt een berekening uitgevoerd, en alle niveaus dienen te voldoen aan de eisen.



Figuur 6.2 Opzet golfklap en -oploopzone in de gedetailleerde toets met op te geven grenzen voor de Q-variant in Ringtoets.

6.3.1 Belastingduren in het benedenrivierengebied

In dit memo is gekeken naar locaties in het rivierengebied. Omdat hier ‘stormverlopen’ afvoergedreven zijn leidt het gebruik van de standaard afvoergolven tot zeer hoge belastingduren. In dit geval is het realistischer om uit te gaan van een constante waterstand met een duur van 6 of 12 uur, omdat anders de belasting van een snelle stochast (wind) gekoppeld wordt aan de duur van een trage stochast (afvoer). Dat leidt tot onrealistische resultaten. Het gebruik van een constant waterstandsniveau is acceptabel omdat de stijging van de waterstand tijdens een afvoergolf langzaam verloopt, en het verschil over 12 uur beperkt zal zijn.

Bijlage A: keuze locaties beide mechanismen

Aanleiding
Voor de studie naar de invloed van overschrijdingskanskeuzes in de toetsing van mechanismen GEBU en GEKB wordt een aantal sommen uitgevoerd waarbij resultaten voor laag 1 en 2b worden vergeleken voor GEKB en waarbij voor GEBU laag 1 en 2a worden vergeleken en wordt gekeken naar de vorm van de gedetailleerde toets voor golfklap in het rivierengebied. Dit laatste middels een vergelijking met probabilistische berekeningen. Dit memo behandelt de keuze van de locaties voor deze studies. Dit betreft een eerste locatieset, aan de hand van de resultaten kan het nodig blijken om nog enkele locaties toe te voegen.

Locatiekeuze studie overschrijdingskansen GEBU/GEKB
Voor de locaties wordt een selectie gemaakt op basis van de volgende punten:
- golven voldoende hoog om mogelijk tot falen te kunnen leiden, maar wel met enige variatie
- verschillende normen
- verschillende riviertakken

Als bronnen voor de keuzes van de locaties worden gebruikt:
- RBSO Kaartenatlas - kaart overslaghoogtes
- Resultaten Effect van Invloedsfactoren op Toetsing/Ontwerp (Hoffmans, 2015). Met name ter bepaling van dominantie van waterstand of golfhoogte bij overslag. Deze is af te leiden uit de quotient van faalkansen voor verschillende invloedsfactoren van dijkmeubilair. Wanneer deze quotient 1 is is overslag volledig waterstands-gedreven. Een hogere quotient betekent meer golfdominantie in het ontwerppunt.
- Strijklengtes uit Hydra-Zoet.

Locatie	Dijk-traject	HCLDid	Deelsysteem	x	y	Overslaghoogte RBSO	Norm	Max strijklengte Hydra-Zoet
Lobith	48-1	100563	Bovenrivieren			>1 m	1/30000	3860
Wageningen	45-1	105454	Bovenrivieren			0.5-1 m	1/100000	2325
Doesburg	49-2	102604	Bovenrivieren			≈1 m	1/1000	4114
Zutphen	50-2	102842	Bovenrivieren			<0.5 m	1/3000	1219
Tiel	43-6	107822	Bovenrivieren			0.5-1 m	1/30000	3218
Cabauw	15-1	306305	Benedenrivieren			0.5 m	1/30000	993
Gorinchem	16-1	310211	Benedenrivieren			≈0.5 m	1/100000	2103
Hellevoetsluis	20-4	305613	Benedenrivieren/Haringvliet			nb	1/1000	5740
Moerdijk	34-2	304251	Benedenrivieren/ Haringvliet			nb	1/1000	5889

Voor het Bovenrivierengebied zijn locaties langs alle Rijntakken gekozen met verschillende strijklengtes, normen en overslaghoogtes. De casus Zutphen is waarschijnlijk voor GEBU niet interessant, maar wel voor GEKB. Met de combinatie van locaties bij verschillende riviertakken, verschillende normen is de verwachting dat verschillend gedrag van belastingen voldoende is afgedekt voor GEBU.

Voor de locaties langs de bovenrivieren worden de verdelingen van waterstand en golfhoogte als onafhankelijk aangenomen. Voor het benedenrivierengebied wordt een boven-/ondergrensbenadering gehanteerd omdat er hier wel (enige) correlatie is tussen golven en waterstand.

Voor GEKB worden aanvullend nog een aantal analyses uitgevoerd voor locaties langs de kust en meren. Deze worden geografisch gespreid gekozen. Daarnaast wordt extra gecheckt aan de hand van de quotiënten uit de studie van Hoffmans(2015)[6], zo kan nog enig onderscheid gemaakt worden tussen cases met hoge en minder hoge golven (te zien is dat bij Breskens en Harlingen de factor hoger is dan bij bijvoorbeeld Kloosterzande (wat verder in de Westerschelde ligt dan Breskens) en de Maeslantkering. Niet voor alle locaties is deze factor gegeven, dit heeft te maken met het feit dat niet alle locaties overgenomen zijn of corresponderen met een locatie uit de studie van Hoffmans (2015).

Locatie	HCLDid	Deelsysteem	x	y	Norm	Factor referentie-0.1	Naam in Hoffmans(2015)
Breskens	1500681	Westerschelde			1/1000	40	hm 68.800 - 69.600
Maeslantkering	1700175	Benedenriviere n			1/10000	2.7	D161.3-D173.0_rd06
Kloosterzande	1500454	Westerschelde			1/1000	1	hm 19.610 - 20.090
Harlingen	1000501	Waddenzee West			1/3000	9.79	DV_064_0.15_1.85
Oosterend Texel	1000151	Waddenzee West			1/1000	-	-
Rotterdamse Hoek	700056	IJsselmeer			1/3000	-	-
Cornwerd	700003	IJsselmeer			1/1000	-	-

[6] In deze studie is gekeken naar de invloed van factoren voor dijkmeubilair op faalkansen. De gehanteerde factoren zijn 1 (referentie), 0.5 en 0.1, waarbij geldt dat bij bijvoorbeeld factor 0.5 en kritiek overslagdebiet van 1 l/m/s het werkelijke overslagedebiet waaraan moet worden voldaan 1*0.5 = 0.5 l/m/s is. Gevolg is dat wanneer faalkansen sterk beïnvloedt worden door deze invloedsfactoren (dus hoge quotient), dit betekent dat in het ontwerppunt golfbelasting een relatief grote invloed heeft. Wanneer er weinig verschil is in faalkans tussen de cases duidt dit op waterstandsgedomineerde overslag (dus lage quotient, langs rivieren en op locaties in de luwte).

Bijlage B: Resultaten gedetailleerde toets GEBU voor rivierengebied, stormduur 24 uur

	P = 1/10				**Pnorm**				**PT**			
Locatie	h	Hs	FofS	V/NV	h	Hs	FofS	V/NV	h	Hs	FofS	V/NV
Lobith	15.3	1.4	0.07	NV	17.8	0.6	2.27	V	18.3	0.3	10	V
Wageningen	8.9	0.9	0.24	NV	11.9	0.6	1.69	V	12.2	0.4	10	V
Doesburg	10.2	1.1	0.13	NV	11.0	0.7	1.38	V	11.7	0.3	10	V
Zutphen	7.8	0.8	0.36	NV	9.1	0.5	10	V	9.6	0.3	10	V
Tiel	9.0	1.1	0.12	NV	11.3	0.6	2.10	V	11.8	0.3	10	V
Cabauw	3.2	1.5	0.06	NV	4.9	0.8	0.49	NV	5.3	0.4	10	V
Gorinchem	3.6	1.2	0.09	NV	5.8	0.7	1.33	V	6.1	0.4	10	V
Hellevoetsluis	2.0	2.4	0.03	NV	2.4	2.3	0.03	NV	2.7	2.0	0.04	NV
Moerdijk	nb	nb	*10*	V	3.6	0.5	2.77	V	4.2	0.6	1.51	V