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A semi-probabilistic assessment rule for the stability of block revetments under wave attack

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R.B. Jongejan
M. Klein Breteler

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Client	Project	Reference	Pages
Rijkswaterstaat	1220080-004	1220080-004-ZWS-0002	58

Key words
WTI2017, probabilistic assessment, semi-probabilistic assessment, partial safety factor, block revetments, residual strength

Abstract
This report presents the semi-probabilistic assessment rule (i.e. definitions of representative values and partial safety factors) for the stability of block revetments under wave attack, for use in the WTI2017. This rule has to be implemented in Ringtoets.

The semi-probabilistic assessment rule rests on detailed calibration studies that have been carried out for blocks on their side and columns, and a study about the effects of residual strength on the probability of levee failure.

When the probability of flooding given the failure of the revetment is small, i.e. when there is significant residual strength, the target reliability for the stability of the block revetment may be lowered. This leads to a lower partial safety factor. Such an integrated procedure is less conservative than the step-wise procedure from the WTI2006 and WTI2011, in which the stability of the revetment is evaluated independently from the residual strength, and vice versa.

Partial safety factors of 0.9, 0.1 or 1.1 should be applied to the block thickness, depending on the residual strength classification. These partial safety factors are broadly applicable: they are independent from the flood protection standard, the water system and the type of block revetment. The hydraulic loads should be obtained with the Q-variant in Hydra-Ring for an exceedance probability equal to the maximum allowable probability of flooding. The representative value of the block density is the 5th quantile value. The representative values of all other stochastic variables related to the block revetment are average values.

The partial factors presented herein should only be used in safety assessments of existing revetments, they should not be used for design purposes.

Version	Date	Author	Signature	Review	Signature	Approval	Signature
2	August 2015	R.B. Jongejan		R. 't Hart		G. Blom	
						b.a. A.M. de Leeuw	

This report rests on calibration studies that have also been discussed with Ton Vrouwenvelder (TNO) and Ed Calle (Deltares). The proposed partial safety factors have been discussed in an expert panel with Hans van der Sande (Scheldestromen), Mark Klein Breteler (Deltares), Ton Vrouwenvelder (TNO) and Ruben Jongejan (RMC).

State
final

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Trefwoorden
WTI2017, probabilistische beoordeling, semi-probabilistische beoordeling, partiële veiligheidsfactor, steenzetting, blokken op hun kant, toplaagstabiliteit onder golfaanval, reststerkte

Nederlandse samenvatting
In dit rapport worden de achtergronden beschreven van het semi-probabilistische voorschrift voor de beoordeling van de toplaagstabiliteit van steenzettingen onder golfaanval. Het voorschrift zal deel uitmaken van het WTI2017 en worden geïmplementeerd in Ringtoets.

Het semi-probabilistische voorschrift berust op een tweetal kalibratiestudies die zijn uitgevoerd voor zuilen en blokken op hun kant, evenals een studie naar het effect van reststerkte op de kans op een dijkdoorbraak.

Als de kans op een overstroming gegeven het falen van de toplaag klein is (grote reststerkte), dan mag de doelbetrouwbaarheid voor de toplaag worden verkleind. In dat geval volstaat een kleine partiële veiligheidsfactor voor de toplaagdikte. Een dergelijke geïntegreerde beoordeling van de toplaag en de aanwezige reststerkte is minder conservatief dan de stapsgewijze beoordelingsmethode uit het WTI2006 en het WTI2011 waarin de stabiliteit van de toplaag en de reststerkte onafhankelijk van elkaar worden beoordeeld.

Voor de dikte van de bekleding dient een partiële veiligheidsfactor van 0,9, 1,0 of 1,1 te worden aangehouden, afhankelijk van de reststerkteclassificatie. Deze partiële veiligheidsfactoren zijn breed toepasbaar: ze zijn onafhankelijk van de overstromingskansnorm, het watersysteem of het steenbekledingstype. De rekenwaarde van de belasting dient te worden bepaald met de Q-variant in HydraRing voor een overschrijdingskans die getalsmatig gelijk is aan de norm. De representatieve waarde van de dichtheid van de zetsteen is het 5% kwantiel. De overige representatieve waarden zijn gemiddelde waarden.

De bovengenoemde partiële veiligheidsfactoren zijn uitsluitend bedoeld voor toetsingen. Ze zijn niet geschikt voor ontwerpdoeleinden.

Contents

Appendices

Symbols

Symbol	Definition	Unit
a, b, c	Constants	-
b_f	Thickness of the filter layer	m
$\cot\alpha$	Outer slope	-
D	Block thickness	m
D_{f15}	Grain size of the filter material	m
d_i	Correlation distance	m
f	Maximum allowable contribution of revetment failure to the probability of flooding	-
f_O	Maximum allowable contribution of overtopping failure to the probability of flooding (f_O=0.24)	-
F_{BR}	Failure of block revetment	-
F_{RS}	Exceedance of the residual strength (failure of residual strength)	-
h	Water level at a particular moment during the storm relative to NAP	m
$h_{\max}$	Water level at the top of the storm relative to NAP	m
H_s	Significant wave height at a particular moment during the storm, at water level h	m
$H_{s,crit}$	Critical significant wave height	m
$H_{s,\max}$	Significant wave height at the top of the storm	m
N	Length-effect factor for revetment failure under wave attack: number of independent, equivalent block revetments	-
N_O	Length-effect factor for overtopping	-
$P_{cross,avg}$	Average cross-sectional probability of failure	yr^{-1}
P_f	Probability of failure	yr^{-1}
P_{norm}	Maximum allowable probability of failure (flood protection standard)	yr^{-1}
P_T	Target failure probability: maximum allowable probability of flooding due to the series of events triggered by the instability of a block revetment under wave attack that lead to flooding	yr^{-1}
$P_{T,cross}$	Cross-sectional target failure probability; the average cross-sectional probability of failure may not exceed $P_{T,cross}$	yr^{-1}
$P_{T,cross,corr}$	Cross-sectional target failure probability after correcting for correlations with overtopping	yr^{-1}
$P_{T,cross,O}$	Cross-sectional target failure probability for overtopping	yr^{-1}
R	Resistance	-*
R_d	Design value of stochastic resistance variable R	-*
R_{rep}	Representative value of R	-*
S	Load	-*
S_d	Design load	-*
T_{storm}	Storm duration	s
T_{BR}	Time to failure of block revetment	s
T_{RS}	Time to failure of base and filter layers (and possibly a geotextile) and the remainder of the levee (time to failure of residual strength)	s
u	Standard normally distributed variable (mean μ=0 and standard deviation σ=1)	-
X_i	Stochastic variable	-*
Z	Limit state function (Z=R-S)	-
Z_1	A cross-section's limit state function for overtopping failure	-
Z_2	A cross-section's limit state function for block revetment failure under	-

Symbol	Definition	Unit
	wave attack	
z_{Bottom}	Bed level relative to NAP	m
Z_{II}	Linearized and normalized limit state function	-
α_i	Influence coefficient for stochastic variable X_i ($\sum\alpha_i^2$=1)	-
α_m	Influence coefficient for model uncertainty parameter	-
α_{Si}	Influence coefficient of the hydraulic load in limit state function i	-
β	Reliability index	-
β_{norm}	Reliability index that corresponds to the flood protection standard	-
$\beta_{T,cross}$	Cross sectional reliability requirement (reliability index)	-
γ	Overall safety factor	-
γ_m	β_T-invariant model factor	-
γ_R	Partial safety factor for stochastic resistance variable R	-
γ_S	Partial safety factor for stochastic load variable S	-
γ_β	β_T-dependent safety factor	-
Δ	Relative density of the blocks	-
Δl	Length of independent, equivalent stretches	m
Δx	Distance between two cross-sections	m
λ_1	Maximum allowable contribution of block revetments to the probability of flooding due to revetment failures (all types)	-
λ_2	Maximum allowable contribution of failures of block revetments and subsequent erosion to the probability of flooding due to block revetment failure	-
λ_3	Maximum allowable contribution of failures of block revetments caused by wave attack to the overall probability of failure of a block revetment	-
μ	Expected value	-*
$\Phi(\cdot)$	Standard normal distribution function	-
$\rho_{i,\infty}$	Lower limit of the autocorrelation function (‘residual correlation’) for variable X_i	-
ρ_S	Block density	kg/m^3
$\rho_{Z1,Z2}$	correlation coefficient between limit state functions Z_1 and Z_2	-
σ	Standard deviation	-*

* The unit depends on the variable concerned

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. Nowadays, policymakers are contemplating a move towards flood protection standards defined 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 WTI2017.

The WTI2017 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 WTI2017 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 WTI2017 concerns the semi-probabilistic assessment rule for the stability of block revetments under wave attack (in Dutch: “Toplaaginstabiliteit onder golfaanval”, ZTG). The residual strength of e.g. base and filter layers is an integral part of this rule. The semi-probabilistic assessment rule presented herein rests on the results of the following studies:

1. A calibration study for the stability of columns under wave attack (Jongejan et al., 2015a)
2. A calibration study for the stability of blocks on their side under wave attack (Jongejan et al., 2015b)
3. A study into the differences between assessments of the stability of block revetments, and assessments of the stability of block revetments *and subsequent erosion* (Kaste and Klein Breteler, 2014).

This report presents a summary the abovementioned studies and synthesizes their results .

The report is organized as follows. Chapter 2 introduces several basic concepts in reliability engineering. Chapter 3 then provides an overview of the numerous types of block revetments and shows for which types of block revetments partial safety factors have been calibrated. The failure mechanism models that have been used in the calibration exercises and residual strength analyses are discussed in chapter 4. An overview of the calibration procedure is presented in chapter 5. Each step in this procedure is discussed in greater detail in chapters 6 to 11. Conclusions and recommendations are given in chapter 12.

A summary (fact sheet) of the semi-probabilistic assessment rule is given in Appendix H.

2 Basic concepts

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 Steentoets, 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):

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

or

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

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:

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

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:

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

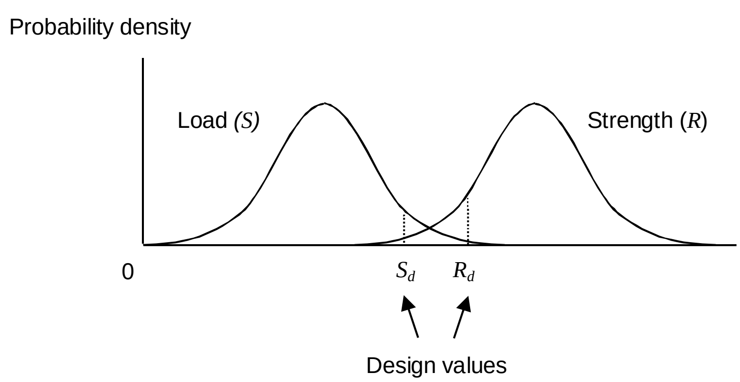
where
$\Phi(.)$ standard normal distribution function
β reliability index

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.



Fully probabilistic assessment: evaluate whether $P(R<S) \leq P_T$
Semi-probabilistic assessment: evaluate whether $S_d \leq R_d$

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:

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

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

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 The need to generalize the results for columns and blocks on their side

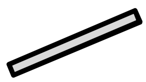
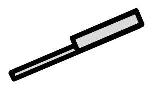
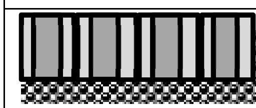
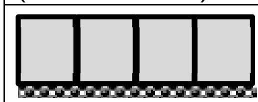
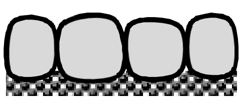
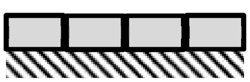
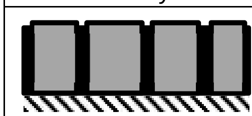
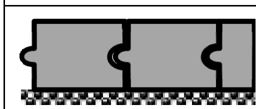
Calibration exercises have been carried out for columns (Jongejan et al., 2015a) and blocks on their side (Jongejan et al., 2015b). Concrete columns with strong interaction (in Dutch: 'zuilen met klemming', Figure 3.1) are typical for levees along the Western Scheldt, the Eastern Scheldt, Lake IJssel, the Markermeer and some other locations in the Netherlands. Blocks on their side (in Dutch: 'blokken op hun kant', Figure 3.2) can be found in Zeeland, along the Eastern Scheldt and the Western Scheldt.

Figure 3.1 Concrete columns. The photo also shows the granular filter and a geotextile.

Figure 3.2. Blocks on their side.

While columns and blocks on their side are most common, there are numerous other types of block revetments, each with different characteristics. In theory, partial safety factors should be calibrated for each individual case/limit state function. Since this is practically impossible within the limited time available for the development of the WTI2017, the results of calibration exercises for a subset of cases had to be extrapolated to other types of block revetments, based on expert judgment. This is illustrated in Table 3.1.

Table 3.1. Types of block revetments.

Type of revetment		Continuous, single revetment 	Transitions 	Berms 		Low crested breakwaters 	
				Berm	Slope above berm	Crest	Inner slope
 Columns on granular layer (concrete / basalt)		(calibration study by Jongejan et al., 2015a)					
 Blocks on granular layer	Without clamping	(calibration study by Jongejan et al., 2015b)					
	With clamping						
 Nordic stones on granular layer							
 Blocks on clay or sand							
 Penetrated stones							
 Interlock blocks etc.							

Roughly six categories of placed block revetments can be identified (Table 3.1, first column):

1 The category ‘columns on a granular layer’ includes common concrete columns like Basalton, Hydroblocks, Ronaton or Pit-polygoon with granular material between the columns. Basalt is a special case within this category. Natural stone blocks may be

categorized as columns if there is clamping due to granular material in the voids between the blocks.

2 Blocks on a granular layer are usually well-sized concrete blocks that are placed in full contact with each other. Despite the method of placement, there is often little interaction between the elements (no clamping). The blocks are sometimes kept apart by spacers, with the voids being filled by granular material. When there is significant clamping, the blocks behave differently. The presence of significant clamping has to be proven by in situ testing.

3 Nordic stones are irregularly shaped elements of natural rock that are more or less rounded. They are boulders shaped by glacial activity. The same assessment rules are used for Lessinische and Doornikse stones.

4 Blocks without a granular layer might be placed directly on the soil (clay or sand) or on a geotextile that covers the soil.

5 The category ‘grouted stones’ covers all block revetments in which a bituminous or concrete penetration ensures a durable interaction between the elements. These elements come in various shapes and sizes, including columns. A related type is the superficially grouted block revetment. For these types of revetments, the grouting is only superficially present in the voids between the elements, because the voids were filled with granular material before the penetration mortar was applied.

6 The last category in Table 3.1 stands for all cases in which the interaction between the elements is realised by other means than a granular fill (although there may be granular material between the elements as well). The interaction could be the result of interlocking, or steel cables that keep the elements together or a geotextile to which the blocks are attached.

There are several factors that determine how the reliability of a block revetment should be evaluated (Table 3.1, columns 2-5):

1 Almost all model tests concerned single, continuous revetments. The models for single, continuous revetments are therefore most refined and best documented. These could be seen as a reference for the other cases.

2 Transitions from one type of revetment to another may reduce the strength of the revetment or change the way in which the revetment is loaded (depending on the type and/or change in the revetment).

3 Berms will strongly influence the loading conditions on the berm and the slope above the berm.

The failure mechanism models for breakwaters differ from the ones for levees. The assessment rules for revetments on breakwaters are relatively simple rules.

Partial safety factors have only been calibrated for columns and blocks on their side. These are very different types of block revetments. Columns have a relatively short leakage length. This means that the most important loads are wave impacts. Their resistance is not only formed by their weight. Clamping/interaction also contributes strongly to their resistance. By contrast, blocks on their side have a relatively long leakage length so that relatively high loads occur during wave-rundown. Their resistance is entirely due to their weight because there is no clamping. The calibration studies for these very different types of blocks have given similar results (see also chapter 8), for a wide range of different loading conditions (see sensitivity studies in Appendix C).

The failure mechanism models for other types of block revetments are based on the theory that accurately captures the behaviour of columns and blocks on their side, experiments and practical experience. Conservative (safe) assumptions have always been made when data was scarce or unavailable. This, combined with the similarities between the results of the calibration studies for two very different types of block revetments, gives confidence in the broader applicability of the calibrated partial safety factors presented in this report.

4 Modelling the failure of a block revetment and subsequent erosion

4.1 The stability of a block revetment under wave attack

A block revetment may become unstable (fail) when breaking waves generate uplift pressures that are higher than the blocks can resist. The pressures are transmitted through the filter layer of the revetment to the region next to the impact zone (see Figure 4.1). When the upward pressure exceeds the downward pressure due to the block weight, the block might be pushed out.

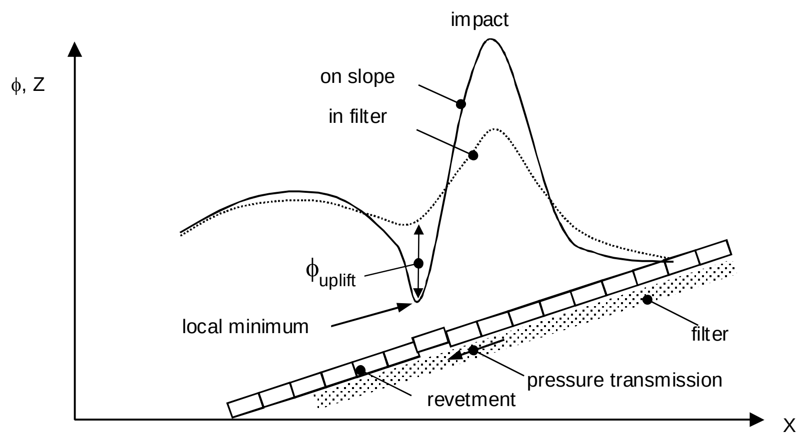


Figure 4.1 Pressure head ϕ on the slope and under the revetment due to wave impact

The probability of flooding given the failure of a revetment is smaller than one. This is due to residual strength, i.e. the part of the total resistance that is ignored when only the stability of the revetment is considered. The residual strength is formed by the time to failure of base and filter layers (and possibly a geotextile) and the remainder of the levee. Figure 4.2 shows how the failure of a block revetment and subsequent erosion may lead to flooding. The events that are relevant for levee failures due to the failure of block revetments under wave attack (‘toetsspoor ZTG’) have been highlighted.

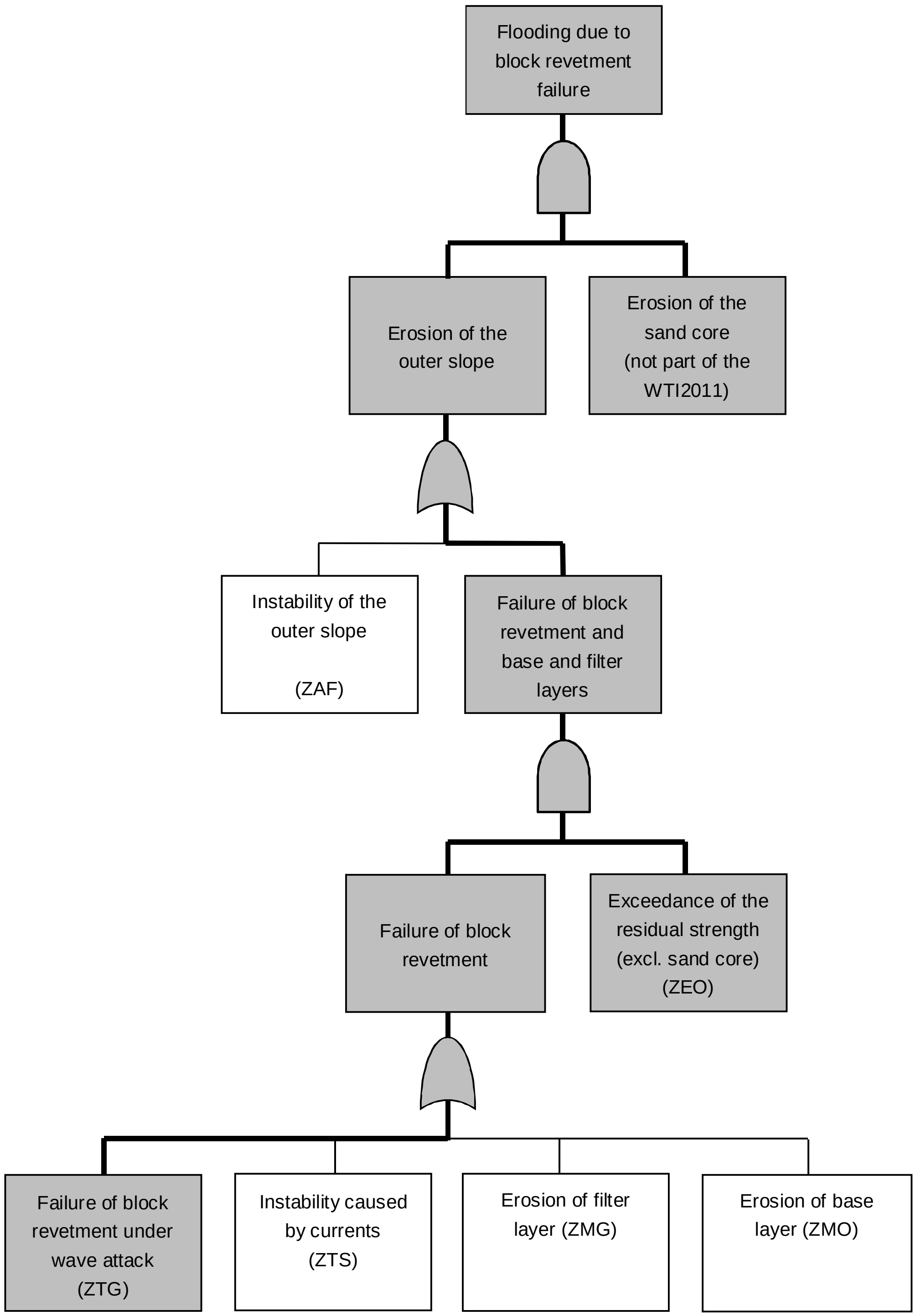


Figure 4.2. A fault tree for flooding due to revetment failure. The parts of the fault tree that this study is concerned with are highlighted. The other mechanisms are covered by other rules. The names of the 'toetssporen' in the WTI2011 are placed between brackets.

4.2 Modelling the stability of block revetments

A detailed discussion of the available failure mechanism models for the stability of block revetments under wave attack is given by Jongejan et al. (2013). The C#-version of Steentoets would be the preferred basis for the development of probabilistic assessment procedures for the stability of block revetments. A probabilistic prototype with this Steentoets version has recently been developed but it is not yet sufficiently robust or efficient for use in full-scale calibration exercises. This is why response surfaces (also named proxy functions) were used.

Response surfaces are available for three different types of block revetments: columns, blocks on their side, and koperslakblokken (but data for defining model uncertainty factors for koperslakblokken are not available). They are based on thousands of Steentoets computations for three different types of block revetments (Klein Breteler & Mourik, 2014). While each response surface closely mimics Steentoets, the correspondence between the response surfaces and Steentoets is imperfect and variable. This means that calculations with a response surface yield uncertain estimates of the outcomes of Steentoets calculations. Normally, this uncertainty should be taken into account in probabilistic analyses via a stochastic error term. In the calibration exercises, this uncertainty has been ignored. The reason for doing so is that we are interested in the failure probabilities of block revetments that would just pass a semi-probabilistic assessment. These semi-probabilistic assessments have been carried out with the same response surfaces that were used in the probabilistic assessments. Hence, the introduction of an error term would only distort the comparison between probabilistic and semi-probabilistic assessments.

Although the relation between partial safety factors and failure probabilities is hardly affected by the use of response surfaces (see also Appendix E), the “real-life” dimensions of blocks may differ slightly from the ones computed here. The fact that the set-up of the calibration exercise (essentially a comparison of probabilistic and semi-probabilistic assessments) makes it irrelevant to deal with the uncertainty related to response surfaces does not mean this uncertainty is gone. It only pops up in a place that is not directly relevant to our objectives, i.e. in the precise dimensions of blocks.

The impact of using a response surface rather than Steentoets has been investigated by carrying out semi-probabilistic and probabilistic calculations with both models for a number of test cases (Appendix E). The results of these calculations suggest that the results of a calibration exercise based on the response surface for blocks on their side are likely to be almost identical to a calibration exercise based on Steentoets: the differences in the calculated partial safety factors are smaller than 1%. The use of the response surface appears to be slightly conservative yet unlikely to lead to noticeably higher partial safety factors.

It is emphasized that all semi-probabilistic assessments of block revetments are to be carried out with Steentoets. The response surfaces will not be part of the WTI2017.

4.3 Modelling residual strength

A levee fails when the storm duration is greater than the time to failure of the revetment, granular filter, geotextile, base layer and the remainder of the levee (up to a critical profile). To gain insight into the effect of residual strength on required block thicknesses, a process-based erosion model has been developed by Kaste and Klein Breteler (2014).

The model for calculating the time to failure of the block revetment and the granular filter rests on formulae from Steentoets (Klein Breteler, 2012). These formulae form the basis for a time step model that calculates the time to failure of the block revetment and the filter layer. The time to failure of the clay layer and sand core is calculated with an erosion model that calculates, for each time step, the erosion volume and the remaining dike profile. The levee fails when the levee is breached. The model is capable of dealing with a series of storms during a single storm season. The time to failure of geotextiles has so far been ignored.

As indicated by Rijkswaterstaat, including the time to failure of a levee's sand core in levee safety assessments is controversial. This is why the outcomes of probabilistic calculations in which the erosion of the sand core played an important role have been interpreted with caution. The chosen safety format of the semi-probabilistic assessment rule is also such that it will be directly visible to what extent residual strength plays a role in the outcome of a semi-probabilistic assessment (see also chapter 9).

5 Calibration procedure

The objective of the calibration procedure is to develop a semi-probabilistic assessment rule for the failure of block revetments under wave attack and subsequent erosion. The failure of a block revetment does not necessarily lead to a levee failure, i.e. flooding. For that to happen, the failure of the block revetment would have to be followed by erosion, leading to a breach. When it comes to revetments, the resistance of filter and base layers as well as the levee's core is often called residual strength.

Ideally, residual strength would have been an integral part of the calibration procedure. However, because of time constraints, the studies into the stability of blocks and the study into the effect of residual strength and had to be carried out in parallel. This also meant that the results of these studies had to be integrated at a later stage, which is reflected in the step-wise handling of residual strength in the calibration procedure (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 that would be suitable for cases without residual strength. 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 block thickness 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.

Include the effect of residual strength:

4 Develop a procedure for including residual strength in semi-probabilistic assessments. This step comprises the following activities:
   - 4.1 Establish easily identifiable, distinct residual strength classes on the basis of probabilistic calculations.
   - 4.2 For each class, determine a safe estimate of the conditional probability of flooding, i.e. the probability of flooding given the failure of the revetment.

Interpret the results and establish the semi-probabilistic assessment rule

5 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.

6 Compare the semi-probabilistic assessment rule from step 5 with the present-day rule (WTI2011).

An overview of the abovementioned steps is presented in Figure 5.1.

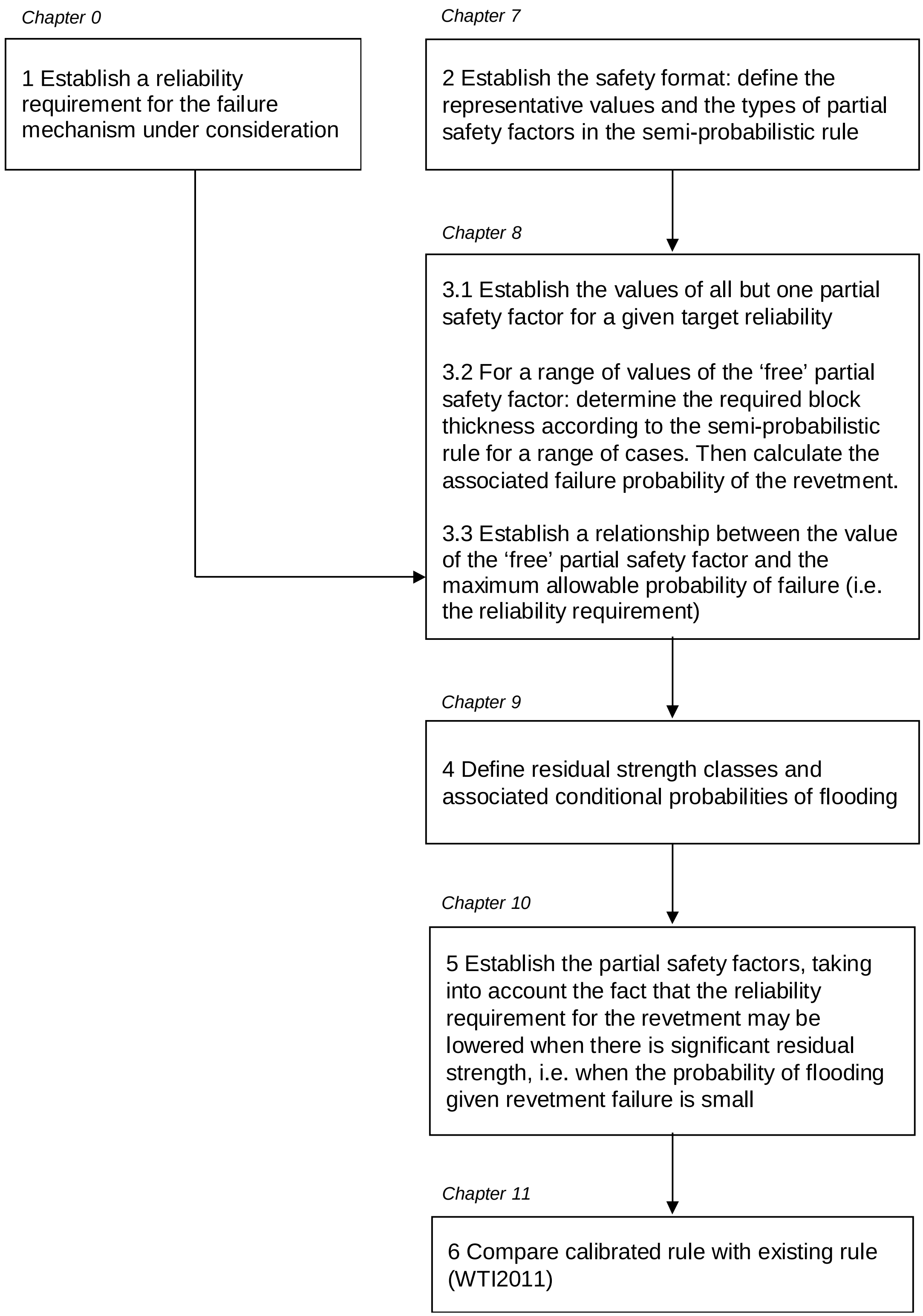


Figure 5.1. Schematic overview of the calibration procedure.

6 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 maximum allowable probability of flooding (section 6.1), from which the reliability requirement for revetments is derived (section 6.2). Because there are numerous types of revetments and (sub)failure mechanisms, the latter requirement has to be turned into a reliability requirement for the stability of block revetments under wave attack and subsequent erosion. This is the subject of section 6.3. Section 6.4 then discusses how the residual strength of e.g. base and filter layers can be taken into account in semi-probabilistic assessments. A summary is provided in section 6.5.

6.1 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. Unlike most levee systems, segments are rarely over 20 km long, they have fairly uniform orientations and they are never located along more than one water system (e.g. lake, river or sea). Segments may consist of numerous dike sections and/or hydraulic structures.

6.2 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 tree's top event. The combined probabilities of the various failure mechanisms may not exceed the maximum allowable probability of flooding. To ensure this requirement is met, the maximum allowable 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 maximum allowable probability of flooding.

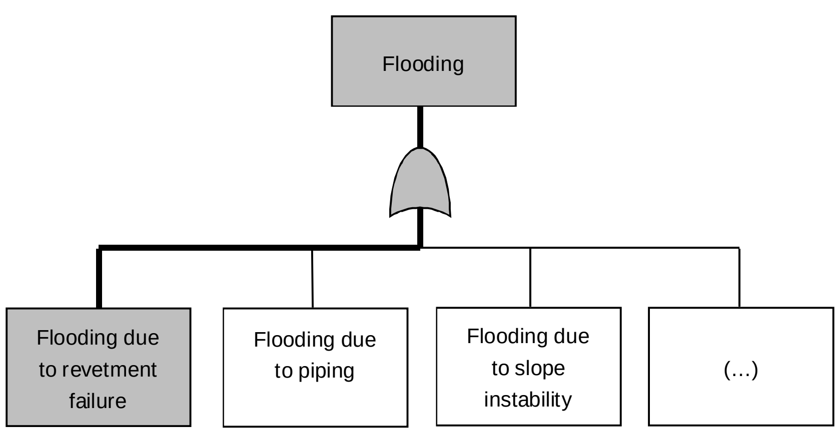


Figure 6.1. A fault tree with different failure mechanisms.

The maximum allowable contributions of the different failure mechanisms to the maximum allowable probability of flooding are shown in Table 6.1. The percentages in Table 6.1 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 6.1. Maximum allowable failure probabilities per failure mechanism, defined as a percentage of the maximum allowable 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 6.1 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.

6.3 Reliability requirement for block revetments under wave attack

The 10%-value in Table 6.1 relates to all revetments, not only block revetments, and to a range of (sub-)failure mechanisms, see Figure 6.2. But this study is concerned solely with the stability of block revetments under wave attack. A reliability requirement for block revetments and this particular failure mechanism can, again, be derived from a fault tree analysis.

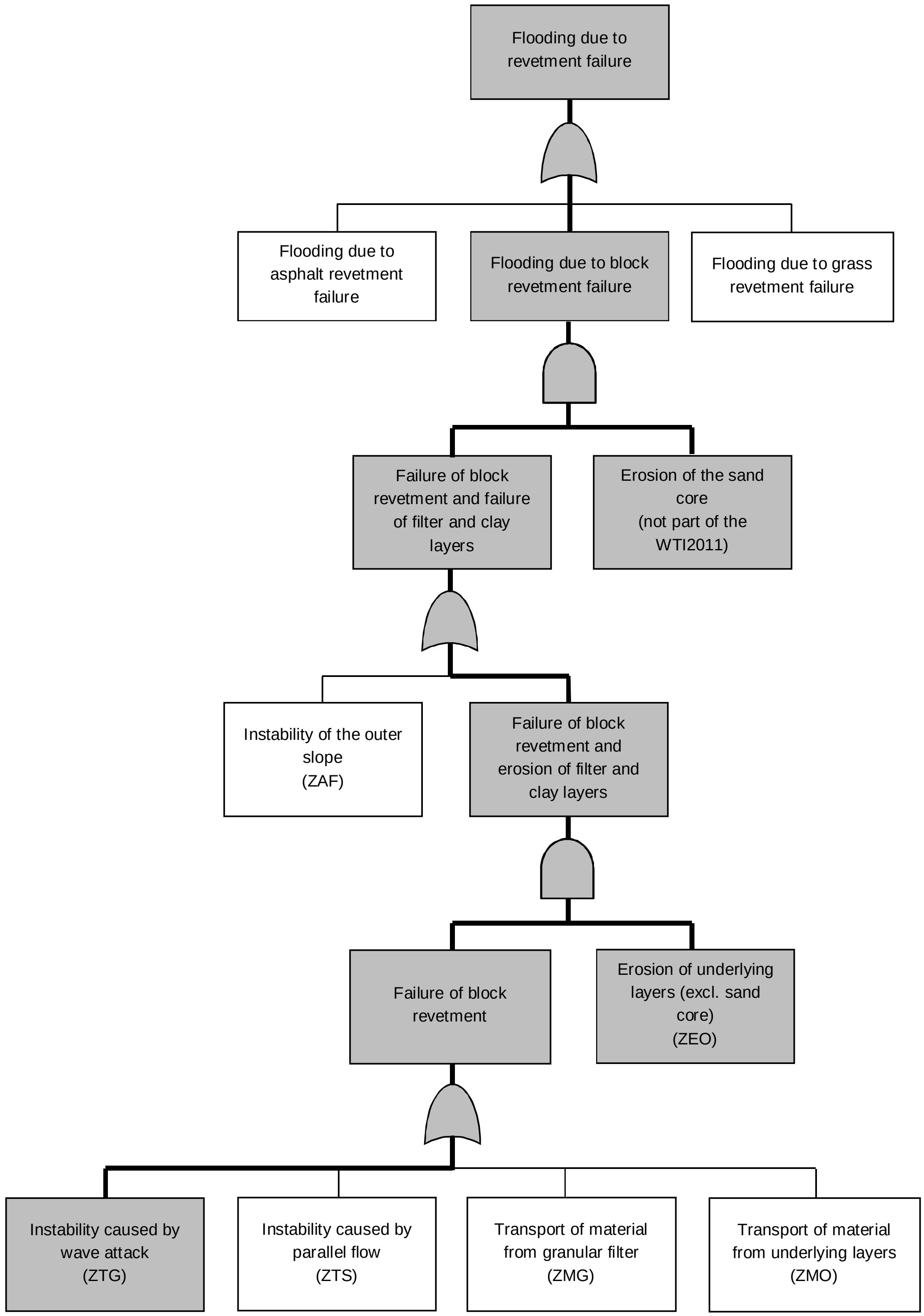


Figure 6.2. Fault tree for flooding due to revetment failure. The parts of the fault tree that this study is concerned with have been highlighted.

Let f be the maximum allowable contribution of revetment failures to the probability of flooding (f=0.10, see Table 6.1), let λ_1 be the contribution of *block revetments* to the probability of flooding due to revetment failures (all types), λ_2 the contribution of failures of block revetments and subsequent erosion to the probability of flooding due to block revetment failure, and let λ_3 be the contribution of failures of block revetments caused by *wave attack* to the overall probability of failure of a block revetment. The reliability requirement for the stability of block revetments under wave attack then becomes:

$$P_T = f \lambda_1 \lambda_2 \lambda_3 P_{Norm} \tag{6.3.1}$$

where

P_T maximum allowable probability of flooding due to the series of events triggered by the instability of a block revetment under wave attack 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: The results of e.g. the VNK2-project indicate that block revetments are often reliable compared to grass and asphalt revetments. If λ_1 would be set equal to the average contribution of block revetments to revetment failures, λ_1 would get a (very small) value. Yet there may well be segments in which block revetments are relatively important. Choosing a relatively small value of λ_1 would lead to unduly stringent semi-probabilistic assessments for those cases. An intermediate value of λ_1=0.5 was therefore chosen, leaving a fraction of 0.5 to grass and asphalt revetments.

λ_2: Failures of the outer slope due to slope instability (“ZAF”) are assumed to be relatively improbable. A value of λ_2=0.9 was therefore assumed for failures of the outer slope due to revetment failure. Note that any value close to one would yield similar reliability requirements. For instance, if we were to assume λ_2=0.99, the maximum allowable probability of failure would increase by a factor 0.99/0.9=1.1. Similarly, λ_2=0.8 would lead to a maximum allowable failure probability that would only be 0.9/0.8=1.125 times smaller.[1]

λ_3: In the VNK2-project, probabilistic analyses are only carried out for potential failures of block revetments due to wave impacts. Hence, the results of the VNK2-project cannot readily be used to decide on an appropriate value of λ_3. The outcomes of past statutory assessments suggest that instability of block revetments due to wave attack is a relatively important failure mechanism, suggesting a relatively high value of λ_3 should be chosen. Because of the number of other failure mechanisms for block revetments (bottom row, Figure 6.2), a value of λ_3=0.7 was chosen (ZTS, ZMO are relatively unlikely).

Note that inappropriate/inaccurate values of f, λ_1, λ_2, λ_3 can only lead to overly conservative safety assessments. This is because an unduly high (lenient) value of f, λ_1, λ_2, λ_3 implies an unduly stringent reliability requirement for another failure mechanism (see also Jongejan, 2013).

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[1] *Strictly speaking, the failure mechanisms “ZAF”, “ZTS” and “ZMO” could also be placed in the category “other failure mechanisms” for which a separate failure probability budget has been reserved (30%, see Table 6.1); λ_2= λ_3=1 would then be appropriate. As discussed with and confirmed by PMO (minutes of April 28, 2015, no. 1220077-000-HYE-0007), the failure mechanisms “ZAF”, “ZTS” and “ZMO” are treated as sub-failure mechanisms for “flooding due to revetment failure”, for which a (combined) failure probability budget of 10% of the safety standard (default value) is available. Note also that λ_2= λ_3=1 would hardly lead to different reliability requirements/safety factors.*

The abovementioned values of f, λ_1, λ_2, λ_3 and the resulting maximum allowable failure probabilities for block revetments under wave attack (P_T) are shown in Table 6.2. The reliability requirements are also expressed in terms of reliability indices (β_T).

Table 6.2. Reliability requirement for a range of arbitrarily selected flood protection standards.

f (-)	λ_1 (-)	λ_2 (-)	λ_3 (-)	P_{Norm} (yr^{-1})	Reliability requirement (entire segment)	
					$P_T = f\lambda_1\lambda_2\lambda_3 P_{Norm}$ (yr^{-1})	$\beta_T = -\Phi^{-1}(P_T)$ (on an annual basis)
0.10	0.5	0.9	0.7	1/300	1.05E-04	3.71
				1/1000	3.15E-05	4.00
				1/3000	1.05E-05	4.25
				1/10000	3.15E-06	4.52
				1/30000	1.05E-06	4.74

It should be noted that the reliability requirements (P_T or β_T) in Table 6.2 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 or β_T) and cross-sectional failure probabilities is discussed in greater detail in chapter 8.

6.4 Reliability requirement for block revetments under wave attack and subsequent erosion

The reliability requirements presented in Table 6.2 apply to the instability of block revetments under wave attack *and* subsequent erosion. The probability of failure of a block revetment and subsequent erosion equals:

$$P_f = P\left(F_{BR} \cap F_{RS}\right) = P\left(T_{storm} > T_{BR} + T_{RS}\right) \qquad (6.4.1)$$

where

F_{BR} Failure of block revetment
F_{RS} Exceedance of the residual strength
T_{storm} Storm duration
T_{BR} Time to failure of block revetment
T_{RS} Time to failure of base and filter layers (and possibly a geotextile) and the remainder of the levee (time to failure of residual strength)

For calibrating a semi-probabilistic rule for assessing (only) the stability of block revetments under wave attack, the reliability requirement could safely be set equal to P_T from Table 6.2. After all, the condition:

$$P\left(F_{BR} \cap F_{RS}\right) \leq P_T \qquad (6.4.2)$$

is certainly fulfilled when:

$$P\left(F_{BR}\right) \leq P_T \qquad (6.4.3)$$

since

$$P\left(F_{BR} \cap F_{RS}\right) \leq P\left(F_{BR}\right) \tag{6.4.4}$$

or

$$P(T_{storm} > T_{BR} + T_{RS}) \leq P(T_{storm} > T_{BR}) \tag{6.4.5}$$

The reliability requirement for the calibration of a semi-probabilistic assessment rule for residual strength could be obtained analogously: it could also (safely) be set equal to P_T. Treating block revetments and the residual strength of e.g. base and filter layers in isolation will certainly yield sufficiently safe partial safety factors. These may easily be unduly stringent however. This is because the time to failure of the block revetment reduces the duration of the loads on e.g. filter and base layers, which cannot be accounted for when residual strength is assessed in isolation.

While it would be optimal to carry out integrated assessments of block revetments and subsequent erosion (residual strength), a step-wise procedure has been implemented in the WTI2006 and WTI2011 (Figure 6.3). The WTI2006 and WTI2011 assessment rules for residual strength have been relaxed somewhat, however, to account for the time to failure of block revetments. This has been done on the basis of engineering judgment, not on the basis of reliability theory.

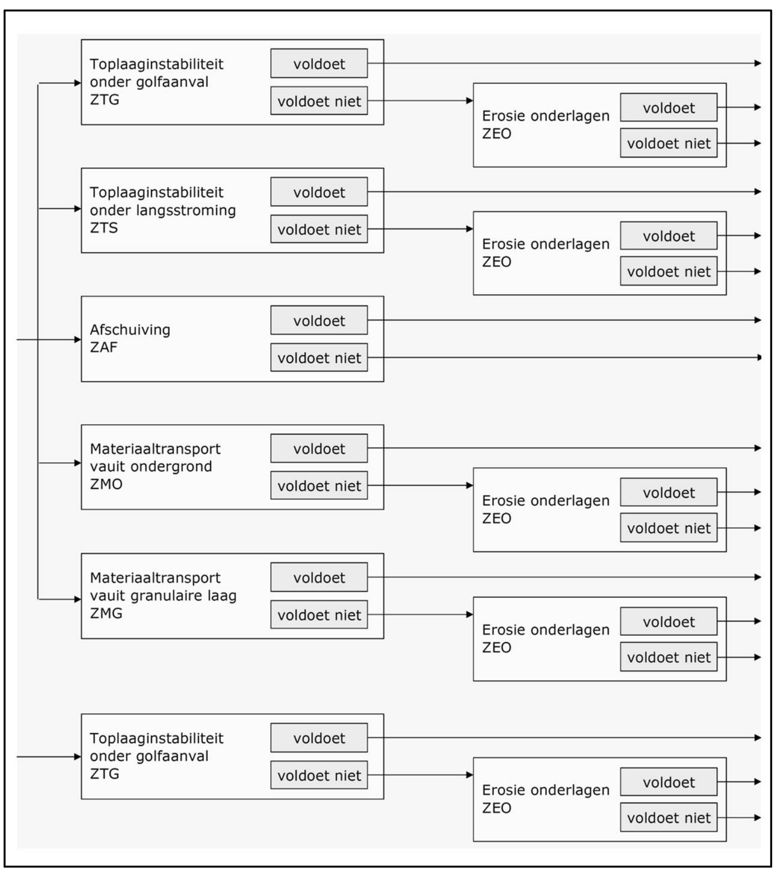

Figure 6.3. Excerpt of the assessment procedure for block revetments according to the WTI2011 (VTV2011: figure 10-1).

For the WTI2017, it is proposed to properly integrate the effect of residual strength in semi-probabilistic assessments. This can be done by correcting the reliability requirement (and hence the β_T-dependent safety factor) for the stability of block revetments under wave attack. After all, when the cross-sectional probability of failure is smaller than required, i.e. when:

$$P\left(F_{RS} \middle| F_{BR}\right) \cdot P\left(F_{BR}\right) < P_{T,cross} \tag{6.4.6}$$

the following must hold:

$$P\left(F_{BR}\right) < \frac{P_{T,cross}}{P\left(F_{RS} \middle| F_{BR}\right)} \tag{6.4.7}$$

Equation (6.4.7) effectively says that the target failure probability for the block revetment (without residual strength) may be increased by a factor equal to the inverse of the conditional probability of failure of the underlying filter and base layers.

6.5 Summary

Figure 6.4 shows how the reliability requirements for block revetments are linked to the flood protection standards. These reliability requirements apply to entire segments. They are linked to cross-sectional reliabilities in chapter 8.

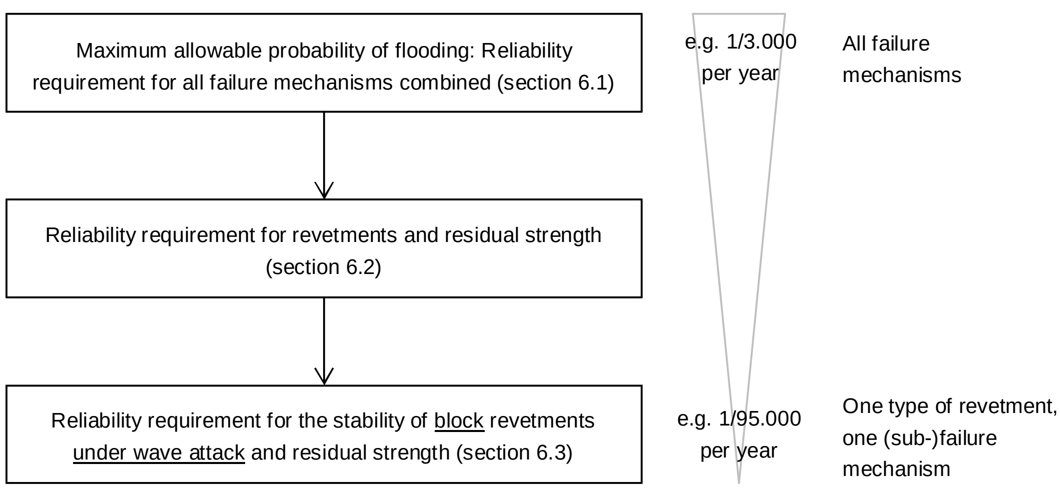

Figure 6.4. Illustration of the derivation of a reliability requirement for the calibration of partial safety factors for semi-probabilistic assessments of the stability of block revetments under wave attack. Note: all reliability requirements in this figure apply to entire segments, they should not be confused with cross-sectional reliability requirements.

In case of significant residual strength, the reliability requirement for a block revetment could be lowered. Residual strength can thus be taken into account in semi-probabilistic assessments by making the partial safety factors for assessments of block revetments depend on (key parameters describing) the residual strength.

7 Step 2: Establishing the safety format

The safety format concerns the definition of representative values and the types of partial safety factors that are to be included in the semi-probabilistic assessment rule. The safety format presented here applies to cases without residual strength. A format that allows for the handling of cases with residual strength is introduced in chapter 9.

7.1 Partial safety factors

The selection of safety factors normally involves a trade-off between accuracy and practicality. In theory, a partial safety factor could be defined for each stochastic variable. Bundling these partial factors into a few or even one safety factor may strongly reduce the complexity of the semi-probabilistic assessment rule. This usually comes at the cost of some conservatism. When it comes to the stability of block revetments, reducing the number of partial factors would hardly reduce the accuracy of the semi-probabilistic assessment rule however. This is because the probabilistic analyses show that the relative importance of the uncertainties related to numerous stochastic variables is relatively small.

The dominant sources of uncertainty are the hydraulic load (α_S is typically smaller than -0.9) and the model uncertainty (α_m is typically 0.1-0.2), see also Appendix B.1. This implies that the design point values of the other variables are close to their 50%-quantile values. Appropriate partial safety factors for such variables would thus be close to 1 (provided their representative values are 50%-quantile values).

Considering the above, only two partial safety factors have been considered:

1. A model factor (β_T-invariant, i.e. derived for a fixed target reliability)
2. A β_T-dependent safety factor, applied to the block thickness

Note that the β_T-invariant model factor and the β_T-dependent safety factor can be combined into a single, overall safety factor without any loss of accuracy as they are multiplicative: both factors apply to the required block thickness, similar to the partial safety factor in today's design rule for block revetments.

7.2 Representative values

By default within the WTI2017, a representative load is defined as a load with an exceedance probability equal to the maximum allowable probability of flooding (Jongejan, 2013). This ensures consistency across failure mechanisms and facilitates comparisons between today's rules and the WTI2017. For block revetments, the representative values of the load parameters should therefore be obtained for an exceedance probability equal to the maximum allowable probability of flooding (using the Q-variant in Hydra-Ring).

The representative value for the model uncertainty factor is equal to one. This means that analysts only have to consider a single model factor. This is because the partial safety factor is then applied to a representative value equal to one.

For pragmatic reasons, the representative values for block revetment parameters should be defined as uniformly as possible. The consistent use of 5% quantiles is preferable over the use of e.g. the 10% quantile for variable X_1, the 25% quantile for X_2, the 55% quantile for variable X_3 and so on. This also means that design values may differ from their theoretical optima. However, for assessments of the stability of block revetments, the consistent use of *average* values for almost all stochastic variables would be close to the theoretical ideal. This is because their design point values are close to 50%-quantile values. The use of average values as representative values is also practical. Because the number of stochastic variables in Steentoets with different distribution types is considerable, and because some variables concern spatial averages rather than point values, calculating e.g. a 5% quantile for each variable would be tedious and error-prone. Also, the parameter values that are currently used in safety assessments and design are close to average values. The only exception concerns the block density, which appears to be close to a 5% quantile in practice. For this variable, the use of the 5%-quantile as representative value is preferred so that existing datasets can be re-used in future semi-probabilistic assessments.

In short, the representative values are defined as follows:

1. Representative values for wind and wave parameters are to be derived from the so-called Q-variant for an exceedance probability equal to the maximum allowable probability of flooding.
2. The representative value of the model uncertainty parameter is equal to one.
3. The representative value of the block density is the 5%-quantile value.
4. The representative values of all other stochastic variables are average values.

The last two choices imply that the parameter values that are currently stored in databases may be reused in semi-probabilistic assessments with the WTI2017.

8 Step 3: Calibrating partial safety factors

This chapter discusses the calibration of partial safety factors for semi-probabilistic assessments of the stability of columns and block on their side under wave attack. Safety factors should be sufficiently safe but not unduly stringent. A calibration criterion is used to decide 'how safe is safe enough'. This criterion is introduced in 8.1. Corrections for strong correlations with overtopping failures are discussed in 8.2. Section 8.3 then deals with the β_T-invariant model factor. The remaining uncertainties and the combined effect of the flood protection standard and the length effect are covered by a β_T-dependent safety factor. Section 8.4 discusses the application of calibration criteria to define this factor.

8.1 The calibration criterion

According to the WTI2017 calibration criteria, the failure probability of a segment should, on average, be smaller than the flood protection standard (Jongejan et al., 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 (see Figure 8.1).

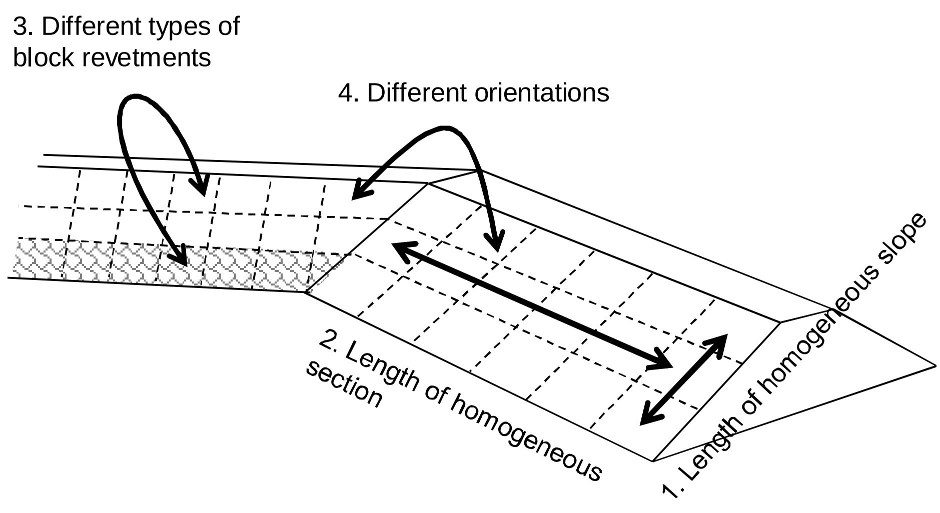

Figure 8.1. Schematic representation of the different contributors to the length effect.

1 For a particular type of block revetment, the "vertical" length effect (no. 1 in Figure 8.1) is likely to be small (note, however, that there may be different types of revetments along a slope, see point 3 below). This is because, for each loading event, the highest loads are concentrated in a fairly narrow band. Note also that the scales of fluctuation of the dominant stochastic variables are much greater than the length of a slope (assuming isotropic conditions, i.e. similar autocorrelation functions along and perpendicular to a slope).

2 For a particular type of revetment, the "horizontal" length effect (no. 2 in Figure 8.1) is likely to be very small. This follows from the fact that only strongly spatially correlated stochastic variables have non-negligible influence coefficients (see Appendix B.1 and Kaste & Klein Breteler, 2012; 2014). This can be explained by the way in which block revetments are made (small tolerances). This is further supported by the calculated length effect for a statistically homogenous revetment: it seems reasonable to assume that the failure probability of a statistically homogeneous stretch is about two times greater than the failure probability of a cross section (see Appendix F; the length effect calculations in Appendix F are based on the influence coefficients of the stochastic variables and their autocorrelation functions, as in PC-Ring and Hydra-Ring).

3 The model uncertainty parameters of different types of revetments are likely to be uncorrelated, which means that the presence of different types of revetments could give rise to a length effect (no. 3 in Figure 8.1). The limit state functions of different types of block revetments may be treated as independent when the squared influence coefficients (α_m^2) of the model uncertainty parameters are greater than e.g. 0.25 (or $\alpha_m=\sqrt{0.25}=0.5$). This is because the correlation coefficient between the limit state functions of two different revetment types cannot be greater than $1-\alpha_m^2=1-0.25=0.75$ when this is the case. If so, the failure probability of a series of block revetments may be approximated by the sum of the (total) failure probabilities of the different revetment types (see also Figure 8.3). However, when the value of α_m^2 is significantly smaller than 0.25 and the squared influence coefficients of the (uncorrelated) material properties are relatively small as well (see also Appendix B.1), the correlation between the limit state functions of two different types of revetments will be strong. In that case the combined failure probability of two different types of revetments will be smaller than the sum of their respective failure probabilities.

4 A length effect may arise from cross sections having different orientations (no. 4 in Figure 8.1). Most segments have fairly uniform orientations, especially when compared to today's levee systems that sometimes cover all wind sectors. The orientation of the levees within a segment is typically smaller than 90-180 degrees.

The impact of the abovementioned contributors to the length effect on the probability of failure of a segment will be reduced by the differences between the failure probabilities of different block revetments and block revetment types within that segment.

The length effect may be characterised by a factor N, which may be interpreted as a number of independent, equivalent revetments. The following condition should be met:

$$N \cdot P_{cross,avg} \leq P_T \quad (8.1.2)$$

which is equivalent to:

$$P_{cross,avg} \leq \frac{P_T}{N} = P_{T,cross} \quad (8.1.3)$$

where

$P_{cross,avg}$ average cross-sectional probability of failure

$P_{T,cross}$ cross-sectional target failure probability

Rather than by a length-effect factor N, the length effect could be characterised by a length of independent, equivalent sections b (e.g. $b=L/N$). The use of a factor N is proposed here, since the main contributors to the 'length-effect' are, in fact, not length-dependent, such as the number of different orientations and block revetment types.

In the VNK2-project, the probability of flooding due to a revetment failure is typically dominated by one revetment with a relatively high failure probability. Since the autocorrelation functions in PC-Ring (the program used in VNK2) are such that the length effect within statistically homogeneous sections is small, this indicates $N \approx 1$. It should be kept in mind, however, that the revetment model in PC-Ring differs from Steentoets. Assuming a value of N that is significantly greater than 1 thus seems prudent.

Hereafter, $N=4$ will be used as a default value (for the consequences of selecting a different value, see e.g. Table 9.4). This value rests on the assumption that $N=2$ within statistically homogeneous stretches and that the combined effect of different orientations and independent revetment types is strongly reduced, to an overall factor of 2, by the fact that the failure probabilities of different stretches are often far apart.

The suggested default value of the length effect factor $N=4$ for use in semi-probabilistic assessments implies that cross-sectional failure probabilities of block revetments should, on average, be 4 times smaller than the failure probabilities for entire segments. The ratio P_T/N will be referred to as a cross-sectional reliability requirement hereafter. It is emphasized that this requirement is related to *average* cross sectional failure probabilities; requiring e.g. 95% of all cross-sections to meet the cross-sectional requirement would be overly conservative. The values of the cross-sectional reliability requirement are shown in the last column of Table 8.1, for $N=4$. The first six columns of Table 8.1 correspond to Table 6.2.

Table 8.1. Cross-sectional reliability requirements for N=4.

f (-)	λ_1 (-)	λ_2 (-)	λ_3 (-)	P_{Norm} (per year)	Reliability requirement (entire segment)	Reliability requirement (cross-section)	
					$P_T=f\lambda_1\lambda_2\lambda_3P_{Norm}$ (per year)	$P_{T,cross}=P_T/N$ (per year)	$\beta_{T,cross}=-\Phi^{-1}(P_{T,cross})$ (annual basis)
0.10	0.5	0.9	0.7	1/300	1.05E-04	2.63E-05	4.04
				1/1000	3.15E-05	7.88E-06	4.32
				1/3000	1.05E-05	2.63E-06	4.55
				1/10000	3.15E-06	7.88E-07	4.80
				1/30000	1.05E-06	2.63E-07	5.02

8.2 Correcting for correlations with overtopping

The cross-sectional reliability requirements for block revetments presented in Table 8.1 are relatively stringent compared to the cross-sectional requirements for overtopping. This is because of differences between the failure probability “budgets” for these failure mechanisms and differences between their length effects. The overtopping requirement for an individual cross section can be approximated by (RWS, 2013a; 2013b):

$$P_{T,cross,O} = \frac{f_O \cdot P_{norm}}{N_O} \quad (8.2.1)$$

where

$P_{T,cross,O}$	cross-sectional target failure probability for overtopping
P_{norm}	maximum allowable probability of flooding (flood protection standard)
f_O	maximum allowable contribution of overtopping failure to the probability of flooding (f_O=0.24, see Table 6.1)
N_O	length-effect factor for overtopping (1≤N_O≤3 according to RWS, 2013b)

With f_O=0.24 and N_O=3, $P_{T,cross,O}$ equals 0.08P_{norm}, while the cross-sectional requirement for block revetments, $P_{T,cross}$, equals 0.008P_{norm}. The difference is a factor 10. Such a difference would hardly be relevant if it was not for the strong correlations between overtopping and revetment failures, as implied by the (very) high FORM-influence coefficients of the hydraulic loads (see Appendix B.1).

Since the maximum allowable contributions of overtopping and revetment failures to the probability of flooding (f and f_O) are additive, the allowable probabilities of failure for overtopping ($P_{T,cross,O}$) and block revetments ($P_{T,cross}$) are additive as well. Yet the actual probabilities of failures are far from additive when correlations are strong. The criterion given by equation (8.1.3) may thus be relaxed. There is little reason to require revetments to withstand loads under which levees will fail due to overtopping.

Let us consider which cross-sectional reliability requirement should be used when evaluating block revetments in isolation to ensure that the reliability requirement for block revetments is met in the real world, where overtopping also plays a part. Let Z_1 be a cross-section’s limit state function for overtopping failure and let Z_2 be the same cross-section’s limit state function for block revetment failure under wave attack. The probability of failure due to any of these two failure mechanisms should meet the following requirement:

$$P(Z_1 < 0 \cup Z_2 < 0) \le 0.08P_{norm} + 0.008P_{norm} \quad (8.2.2)$$

or:

$$P(Z_2 < 0 \cap Z_1 > 0) + P(Z_1 < 0) \le 0.088P_{norm} \quad (8.2.3)$$

When the overtopping requirement is just met, so that $P(Z_1<0)$ equals 0.08P_{norm}, the reliability of the block revetment should be such that the following requirement is met:

$$P(Z_2 < 0 \cap Z_1 > 0) \le 0.008P_{norm} \quad (8.2.4)$$

Note that a smaller failure probability for overtopping than $0.08P_{norm}$ leads to a less stringent requirement.

The required reliability index of the block revetment is the reliability index revetment that just satisfies equation (8.2.4). The procedure that was used for finding this reliability index is shown in Figure 8.2. In this procedure, the limit state functions were defined as follows:

$$Z_1 = \beta_1 - u_1 \tag{8.2.5}$$

$$Z_2 = \beta_2 - u_1\rho_{Z1,Z2} - u_2\sqrt{1-\rho_{Z1,Z2}{}^2} \tag{8.2.6}$$

where
$\rho_{Z1,Z2}$ correlation between Z_1 and Z_2
u_1, u_2 independent, standard normally distributed variables

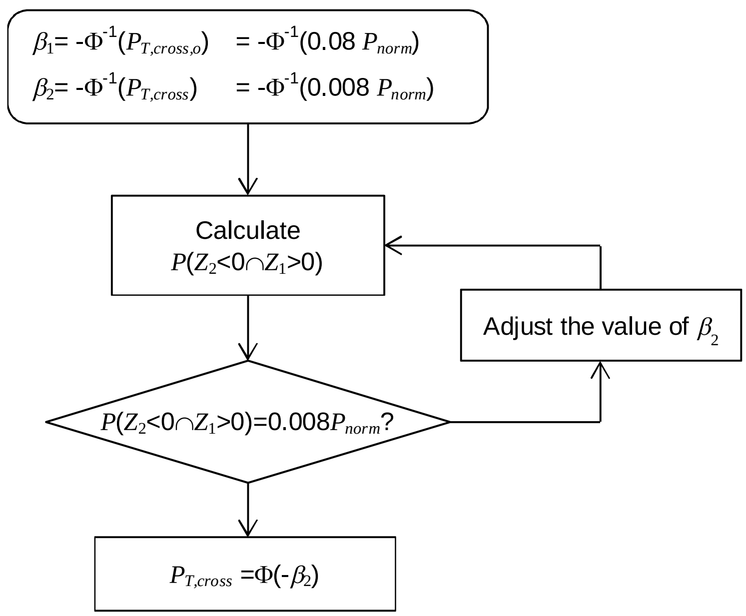

Figure 8.2. The procedure for correcting the cross-sectional reliability requirement for revetment failures for correlation with the limit state function for overtopping.

Figure 8.3 shows the ratio of cross-sectional target failure probabilities with and without accounting for correlations with overtopping as a function of the correlation coefficient $\rho_{Z1,Z2}$, for different values of P_{norm}. The figure shows that this ratio only differs strongly from 1 when two limit state functions are strongly correlated, i.e. when correlation coefficients are greater than 0.8 (see also VanMarcke, 1971).

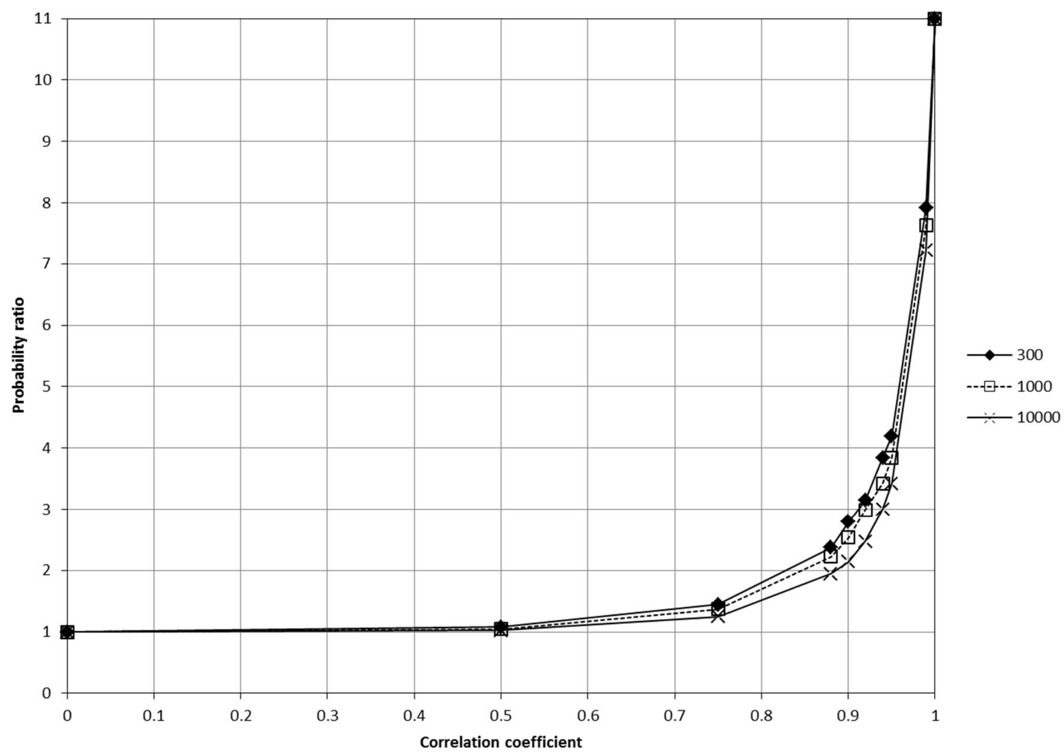

Figure 8.3. The ratio of the cross-sectional target failure probabilities with and without accounting for correlations with overtopping (vertical axis). The ratio is shown as a function of the correlation coefficient between the limit state functions for overtopping and revetment failure (horizontal axis). The ratios differ slightly for the different flood protection standards, as indicated by the differences between the three different lines (the standards are listed as return periods in years).

The correlation coefficient between Z_1 and Z_2 ($\rho_{Z1,Z2}$) can be calculated as follows:

$$\rho_{Z_1,Z_2} = \alpha_{S1}\alpha_{S2} \tag{8.2.7}$$

where

α_{Si} influence coefficient of the hydraulic load in limit state function i

Equation (8.2.7) assumes that the hydraulic loads in both limit state functions are perfectly correlated and that all other stochastic variables are uncorrelated. The adjusted cross-sectional reliability requirements are shown in Table 8.2, see Jongejan et al. (2015a, 2015b) for further details.

Table 8.2. Cross-sectional reliability requirements before and after adjustments for correlations with overtopping.

Type of blocks	Water system	Flood protection standard (per year)	Cross-sectional target failure probability Before accounting for corrections with overtopping (per year)	Probability ratio (-)	Cross-sectional target failure probability after accounting for correlations with overtopping (per year)	Cross-sectional target reliability index with correcting for correlations with overtopping (on annual basis)
Columns	Western Scheldt	1/300	2.63E-05	2,79	7,32E-05	3,80
		1/1000	7.88E-06	2,53	1,99E-05	4,11
		1/3000	2.63E-06	2,19	5,75E-06	4,39
		1/10000	7.88E-07	2,14	1,69E-06	4,65
		1/30000	2.63E-07	2,05	5,38E-07	4,88
	Wadden Sea	1/300	2.63E-05	2,37	6,22E-05	3,84
		1/1000	7.88E-06	2,22	1,75E-05	4,14
		1/3000	2.63E-06	2,00	5,25E-06	4,41
		1/10000	7.88E-07	1,94	1,53E-06	4,67
		1/30000	2.63E-07	1,76	4,62E-07	4,91
	Lake IJssel	1/300	2.63E-05	3,84	1,01E-04	3,72
		1/1000	7.88E-06	3,42	2,69E-05	4,04
		1/3000	2.63E-06	3,29	8,65E-06	4,30
		1/10000	7.88E-07	3,00	2,36E-06	4,58
		1/30000	2.63E-07	2,92	7,66E-07	4,81
Blocks on their side	Western Scheldt	1/300	2.63E-05	3,84	1,01E-04	3,72
		1/1000	7.88E-06	3,42	2,69E-05	4,04
		1/3000	2.63E-06	3,29	8,65E-06	4,30
		1/10000	7.88E-07	3,00	2,36E-06	4,58
		1/30000	2.63E-07	2,92	7,66E-07	4,81

The probability ratios in Table 8.2 are relatively small, indicating that the effect of correlations between overtopping and revetment failures on target reliabilities (and hence partial safety factors) should not be overstated.

8.3 Deriving a Beta-invariant model factor

Model uncertainty has to be accounted for in probabilistic and semi-probabilistic safety assessments. The value of the model factor depends on:

1. The fixed value of the cross-sectional target reliability $\beta_{T,cross}$ that serves as the basis for the fixed model factor
2. The distribution function of the model uncertainty parameter
3. The representative influence coefficient for the model uncertainty

Ad. 1 The cross-sectional reliability
A relatively low reliability index of 4.0 on an annual basis was chosen as a basis for the derivation of the model factor. The selection of a relatively low, fixed target reliability is a standard practice that ensures that the β_T-dependent safety factor will almost always be greater than one. Otherwise, the β_T-dependent safety factor might have to compensate for a relatively high model factor by being smaller than one.

Ad. 2 The distribution function of the model uncertainty parameter
A model uncertainty parameter (m_{BR}) for assessing the stability of block revetments under wave attack with Steentoets was first defined by 't Hart (2012):

$$m_{BR} = \frac{\left(H_{s,crit} / \Delta D\right)_{\text{large scale tests}}}{\left(H_{s,crit} / \Delta D\right)_{\text{Steentoets}}} \tag{8.3.1}$$

where
$H_{s,crit}$ critical significant wave height
D block thickness
Δ relative block density

The definition of the model uncertainty parameter implies that it should be treated as a resistance variable.

The original probability distribution of the model uncertainty parameter by 't Hart (2012) was later refined by Kaste & Klein Breteler (2012) by including data from more recent Delta flume experiments. Within the context of the WTI2017-project, this single model uncertainty parameter was replaced by different model uncertainty parameters for different types of block revetments.

The proposed model uncertainty parameters for columns and blocks on their side are shifted lognormal distributions. These distributions both have a lower bound of 0.87 but their mean values and standard deviations differ. The lower bound was introduced because values lower than 0.87 were considered physically impossible by block revetment specialists.

The means and standard deviations are based on the differences between Steentoets calculations and the outcomes of flume experiments. To account for the fact that the conditions in flume experiments may differ from real-life conditions, the standard deviations resulting from these comparisons have been increased by 20%. Expert judgment played a role in each of these steps. As an illustration, Figure 8.4, shows the distribution function of the model uncertainty parameter before ("original distribution") and after ("modified distribution") accounting for the differences between flume experiments and real-life cases.

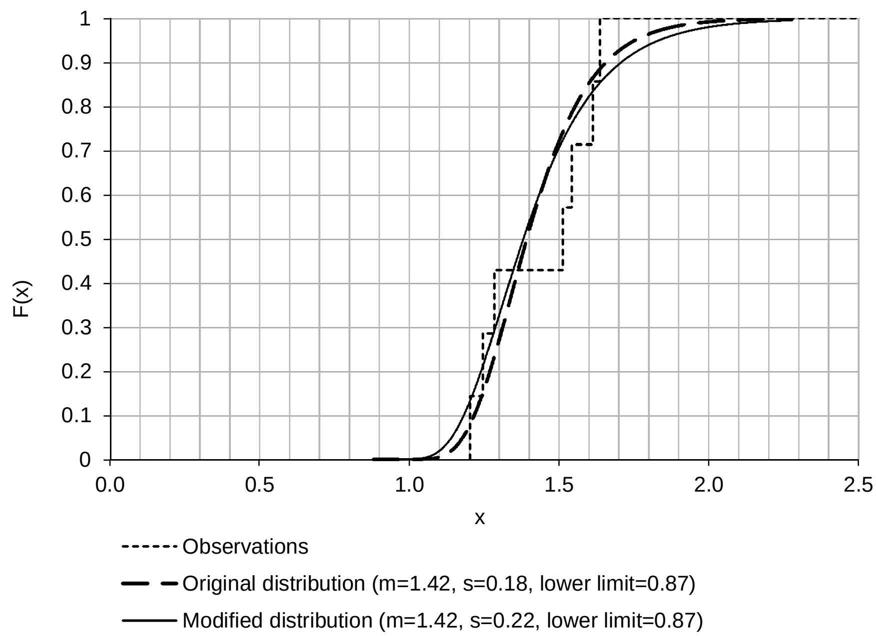

Figure 8.4 The empirical distribution function of the observed differences between Steentoets and flume experiments, the original distribution function of the model uncertainty parameter and the modified distribution function with a 20% greater standard deviation

In sensitivity analyses, the calibrated partial safety factors were found to be relatively insensitive to the distribution function of the model uncertainty parameter. This is because this distribution influences the outcomes of both the probabilistic and the semi-probabilistic assessment: a less favourable model factor leads to a greater probability of failure, but also to a greater model factor in a semi-probabilistic assessment (see also Appendix C.3).

Ad. 3 The representative influence coefficient for the model uncertainty

The calculated influence coefficients of the model uncertainty factor for both columns and blocks on their side are typically around 0.1-0.30.

The model uncertainty parameter acts as a resistance parameter. The probability of non-exceedance of the design point value of the model uncertainty parameter equals $\Phi(-\alpha\beta_{T,cross})$. For $\beta_{T,cross}$=4 and α=0.1-0.3, these probabilities would be about 0.1-0.35. For such probabilities of non-exceedance, the design value of the model uncertainty parameter would still be greater than 1.0, for both columns and blocks on their side. The required block thickness according to Steentoets should then be *reduced* to account for model uncertainty in semi-probabilistic assessments. Such a model factor would effectively introduce an unsafe rather than a safe bias. This unusual result can be explained by the fact that Steentoets is significantly biased towards the safe side.

Safety factors greater than 1 would only be obtained for $\beta_{T,cross}$=4 when the representative influence coefficient would be far greater than values that have been obtained from probabilistic analyses (greater than about 0.9 for columns and about 0.5 for blocks on their side).

Safety factors smaller than 1 are counter-intuitive and error-prone. It has therefore been decided to use a model factor equal to 1. The difference with the theoretical ideal is then compensated for by the β_T-dependent safety factor.

8.4 Calibrating Beta-dependent safety factors

The greater the value of the overall safety factor, the greater the required block thickness and the greater the reliability index. The overall safety factor (γ) equals:

$$\gamma = \gamma_m \gamma_\beta \tag{8.4.1}$$

where

γ_m β_T-invariant model factor
γ_β β_T-dependent safety factor

Since the model factor was set equal to 1 (see section 8.3), the β_T-dependent safety factor on the block thickness is equal to the overall safety factor, or equivalently, it is the only partial safety factor. Hereafter, the model factor is ignored.

For a range of values of the β_T-dependent safety factor and flood protection standards, the required block thicknesses and corresponding reliability indices have been calculated. Figure 8.5 gives an illustration of the results for blocks on their side along the Western Scheldt (also see Appendix B.2). From such a figure, an equation can be derived that governs the relationship between $\beta_{T,cross}$ and γ_β.

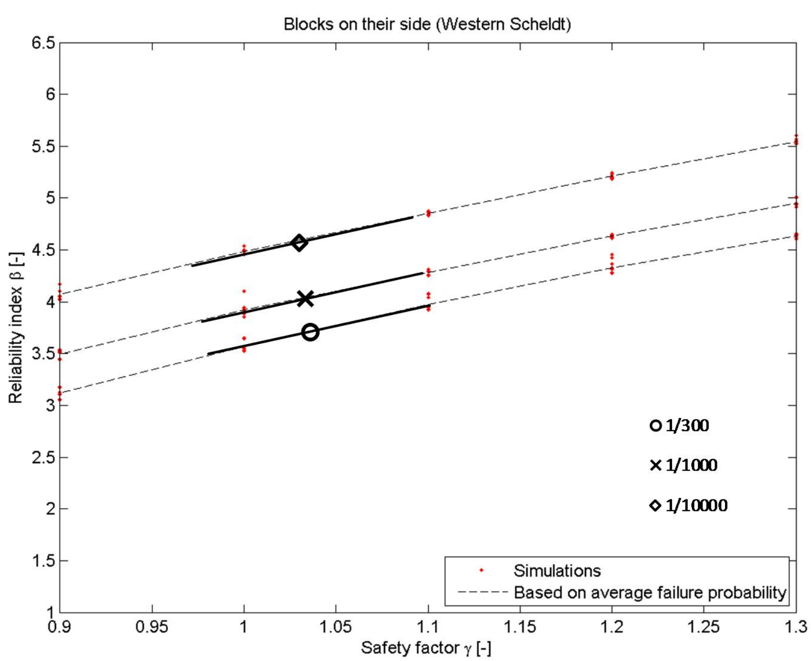

Figure 8.5. Reliability indices as a function of the overall safety factor for blocks on their side along the Western Scheldt, for hydraulic loads with exceedance probabilities of 1/300, 1/1000 and 1/10000 per year. The cross-sectional target reliabilities corresponding to flood protection standards of 1/300, 1/1000 and 1/10000 per year (see Table 8.2) have been marked.

Note that a more stringent flood protection standard corresponds to a more extreme representative load. This means that a ten times more stringent flood protection standard leads to a different β_T-dependent safety factor than a ten times stronger length effect, even though their effect on the cross-sectional reliability requirement is identical. This is why the β_T-dependent safety factors are defined as follows:

$$\gamma_\beta = a \cdot \beta_{T,cross} + b \cdot \beta_{norm} + c \quad \text{with} \quad \beta_{T,cross} = -\Phi^{-1}\left(\frac{f\lambda_1\lambda_2\lambda_3 P_{Norm}}{N}\right) \tag{8.4.2}$$

where:

a, b, c	constants
$\beta_{T,cross}$	cross sectional reliability requirement (reliability index)
β_{norm}	reliability index that corresponds to the flood protection standard
f	maximum allowable contribution of revetment failure to the probability of flooding (default f=0.1; this default value should be increased to account for correlations with overtopping by multiplying it with the appropriate correction factor from Table 8.2)
λ_1	maximum allowable contribution of *block revetments* to the probability of flooding due to revetment failures (all types) (default λ_1=0.5)
λ_2	maximum allowable contribution of failures of block revetments and subsequent erosion (default λ_2=0.9)
λ_3	maximum allowable contribution of failures of block revetments caused by *wave attack* to the overall probability of failure of a block revetment (default λ_3=0.7)
P_{norm}	maximum allowable probability of failure (flood protection standard)
N	length effect factor (default value N=4)

The equations governing the calibrated β_T-dependent safety factors for columns and blocks on their side are given in Table 8.3, for different water systems (see Appendix for B further details).

Table 8.3. Safety factors without residual strength: overview of the results of calibration exercises for block revetments.

Type of blocks	Water system	β_T-dependent safety factor γ_β, with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
Columns	Western Scheldt	$\gamma_\beta = 0.21\beta_{T,cross} - 0.20\beta_{Norm} + 0.80$
	Wadden Sea	$\gamma_\beta = 0.18\beta_{T,cross} - 0.19\beta_{Norm} + 0.88$
	Lake IJssel	$\gamma_\beta = 0.33\beta_{T,cross} - 0.37\beta_{Norm} + 0.95$
Blocks on their side	Western Scheldt	$\gamma_\beta = 0.26\beta_{T,cross} - 0.23\beta_{Norm} + 0.69$

The associated values of the partial safety factors are given in Table 8.4 for different flood protection standards and length/effect factors. Readers should be aware that the accuracy of the presented material is smaller than the number of decimal places may suggest (an accuracy of one decimal place seems justified).

Table 8.4. β_T-dependent safety factors including the effect of residual strength, for different length effect factors (assuming f=0.1, including corrections for correlations with overtopping as specified in calibration studies, λ_1=0.5 λ_2=0.9 and λ_3=0.7). Results for the default length effect factor N=4 are shown in bold. Results for non-existent maximum allowable probabilities of flooding have been greyed out (note: the probabilities that have been presented in the Delta Programme (2014) are one class (a factor 3) smaller than maximum allowable probabilities of flooding).

Type of blocks	Water system	Maximum allowable probability of flooding (per year)	β_T-dependent safety factor		
			N=2	N=4	N=8
Columns	Western Scheldt	1/300	1,02	**1,06**	1,09
		1/1000	1,01	**1,05**	1,08
		1/3000	1,01	**1,04**	1,07
		1/10000	1,00	**1,03**	1,06
		1/30000	1,00	**1,03**	1,06
	Wadden Sea	1/300	1,03	**1,06**	1,09
		1/1000	1,01	**1,04**	1,07
		1/3000	1,00	**1,03**	1,06
		1/10000	0,99	**1,02**	1,04
		1/30000	0,99	**1,01**	1,04
	Lake IJssel	1/300	1,12	**1,18**	1,23
		1/1000	1,09	**1,14**	1,20
		1/3000	1,06	**1,11**	1,16
		1/10000	1,04	**1,09**	1,14
		1/30000	1,02	**1,07**	1,11
Blocks on their side	Western Scheldt	1/300	0,99	**1,04**	1,08
		1/1000	0,99	**1,03**	1,07
		1/3000	0,99	**1,03**	1,07
		1/10000	0,99	**1,03**	1,07
		1/30000	0,99	**1,03**	1,06

From Table 8.4, the following trends emerge:

1. For a given flood protection standard, a greater length effect is associated with a higher β_T-dependent safety factor.
2. For a given length effect factor (N), a higher flood protection standard is often associated with a lower β_T-dependent safety factor. This phenomenon is caused by the fact that the representative load increases with a more stringent flood protection standard. This increase is often greater than strictly required, leading to a lower safety factor.
3. The β_T-dependent safety factors for Lake IJssel appear are about 5-10% higher than those for the other water systems. This is caused by the fact that the representative load has an exceedance probability equal to the value of the flood protection standard, while the exceedance probability of the design point value of the load is about 10-30 times smaller. The difference between the design point value and the representative value has to be bridged by the partial safety factor, which requires a higher safety factor along Lake IJssel. Blocks along Lake IJssel are relatively thin compared to blocks along the Wadden Sea and the Western Scheldt. Along Lake IJssel, the impact on the required block thickness of the difference between the representative load and its design point value is greater in relative terms (hence the greater partial safety factor) but smaller in absolute terms. When the representative load is defined by an exceedance probability that is 20 times smaller than the flood protection standard, the calibrated partial safety

factors for columns would become virtually identical for all water system, albeit smaller than 1.0 for N=4 (see Jongejan et al., 2015a for more details). Such a safety format would complicate historical comparison however, and be inconsistent with the definition of representative loads for other failure mechanisms.

9 Step 4: Including residual strength

This chapter discusses the impact of residual strength on the required partial safety factors for semi-probabilistic assessments of the stability of columns and block on their side under wave attack. The theory on which procedure for including residual strength in semi-probabilistic assessments rests is presented in section 9.1. Section 9.2 then discusses the formulation of residual strength classes and section 9.3 shows to what extent the calibrated partial safety factors could be reduced in case of a particular degree of residual strength.

9.1 Including residual strength in semi-probabilistic assessments of block revetments

The effect of residual strength can be included in semi-probabilistic assessments of block revetments by correcting the target reliability for the stability of block revetments under wave attack for the effect of residual strength, see also section 6.4.

Apart from its effect on target reliabilities, residual strength also leads to a reduction of the influence coefficients of the uncertainties related to the stability of a block revetment. The influence coefficients of the stochastic variables related to the residual strength will be greater than 0. Since the squared influence coefficients sum up to 1 (i.e. $\sum\alpha_i^2$=1 where α_i is the influence coefficient of stochastic variable i), the influence coefficients of the uncertainties related to the stability of the block revetment must go down in case of non-negligible and uncertain residual strength.

The effects of residual strength on target reliabilities and influence coefficients imply that smaller design values, and hence smaller safety factors, may be used when evaluating the reliability of block revetments in case of non-negligible residual strength.

9.2 Defining residual strength classes

The design of semi-probabilistic rules involves a trade-off between accuracy and ease of use. In theory, the target reliability for the stability of a block revetment under wave attack could be written as a continuous function of the design values of residual strength variables. Safety factors that depend on elaborate residual strength computations would strongly complicate semi-probabilistic assessments however. Developing such sophisticated rules would also be impossible within the timeframe of the WTI2017-project. Three easily identifiable residual strength classes have therefore been defined. Safe estimates of conditional probabilities of flooding have been assigned to each class. Semi-probabilistic assessments can then be carried out as follows:

1. Decide to which residual strength class the cross section belongs.
2. Divide the cross-sectional target failure probability for the stability of the block revetment by the (safe estimate of the) conditional probability of flooding associated with the residual strength class from step 1.
3. Calculate the β_T-dependent safety factor for the adjusted reliability requirement.
4. Carry out the semi-probabilistic assessment for the stability of (only) the block revetment under wave attack with the β_T-dependent safety factor from step 3.

The abovementioned procedure rests on two conservative assumptions (i.e. assumptions that create a bias towards the safe side):

1 Within each residual strength class, the highest conditional probability of flooding is used for all cross sections in that class.
2 Only the reliability requirement is adjusted, effectively ignoring the (small) reduction in the influence coefficients of the uncertainties related to the block revetment.

Using an advanced prototype model, Kaste and Klein Breteler (2014) carried out a series of probabilistic calculations for the stability of concrete columns and subsequent erosion. Based on the insights obtained from these calculations, they proposed the following residual strength classes (Klein Breteler, 2015a; 2015b):

1 Small residual strength, if:
   – Hs > 2.0 m or
   – d_c/H_s < 0.6 and B_{dike}/H_s < 20
2 Large residual strength, if:
   – H_s < 1.5 m and d_c/H_s > 0.8
   – H_s < 1.5 m and B_{dike}/H_s > 30
3 Medium residual strength, if the residual strength is not small nor large

Where:

H_s Significant wave height at the toe of the structure as used for the simple safety assessment (m)

d_c Thickness of the clay layer (m). Criteria for deciding whether a base layers may be considered sufficiently clayey will be provided by the “schematiseringshandleiding”.

B_{dike} Width of the dike at the assessment water level as used for the simple safety assessment (m)

Given the differences between the β_T-dependent safety factors for the different residual strength classes, introducing a greater number of classes hardly seems worthwhile (see also section 9.3).

In a previous report by Klein Breteler (2015b), the upstream locations (“bovenrivierengebied”) were placed in the small residual strength class. This was done because such locations have not been studied in Kaste and Klein Breteler (2014). Here, it is proposed to treat the upstream locations like all the other. Doing so would hardly be consequential since block revetments that have to resist significant wave impacts are scarce in the upstream region. Moreover, it is to be expected that the levees in this region have significant residual strength because of the relatively low significant wave heights. This is confirmed by e.g. VNK2 results, even though a rudimentary residual strength model was used in VNK2. Considering this, it seems unnecessarily conservative to make a distinction between upstream locations (‘bovenrivierengebied”) and other locations. Accepting such conservatism would also unnecessarily complicate guidelines (“schematiseringhandleidingen”) and software: it would mean that residual strength would have to be ignored for a list of clearly defined segments.

9.3 Establishing conditional probabilities of flooding for the residual strength classes

Kaste and Klein Breteler (2014) calculated probabilities of failure for cross sections with varying degrees of residual strength. The residual strength of filter and base layers and the levee’s sand core were considered in these calculations. The resistance of geotextiles to wave impacts was ignored. The residual strength classes were defined afterwards, on the basis of the outcomes of the calculations.

Kaste and Klein Breteler (2014) computed the required block thicknesses according to a probabilistic model as well as the required block thickness according to Steentoets by feeding it with representative values (without a partial safety factor and without accounting for residual strength). The reliabilities associated with a particular block thickness and a varying degree of residual strength are not directly available. This means that estimates of conditional probabilities of flooding associated with each residual strength class (i.e. the probabilities of flooding given the failure of revetments) have to be inferred from the computational results that are available.

Kaste and Klein Breteler (2014) found that residual strength may strongly reduce the required thickness of a block revetment, even when considering the effect of a second storm during a storm season. The probabilistic calculations were carried out with Monte Carlo-simulation. In each simulation, the two storms were drawn from ‘storm distributions’ for half a storm season and assumed to take place directly after each other. The latter implies that the opportunities for (emergency) repairs between storms have been ignored. The ‘deterministic’ calculations with Steentoets were carried out with a load with a particular probability of exceedance, without considering the second storm.

An overview of the computational results is provided in Table 9.1 to Table 9.3 for the Western Scheldt, Wadden Sea and Lake IJssel. The tables show the ratios of the block thickness according to the probabilistic model (including residual strength) and the block thickness according to the Steentoets (without residual strength and without the use of a partial safety factor), for different residual strength classes. Because the ‘deterministic’ calculations with Steentoets were carried without a partial safety factor, the ratios are sometimes greater than one.

The results for the different water systems are presented in different tables because the computed block thickness ratios cannot be compared across water systems. Only the differences within water systems are meaningful. One should also be cautious to interpret these ratios as required partial factors that can be used in the WTI2017. This is because of the following:

1. The block thickness ratios in Table 9.1 to Table 9.3 rest on computations with different models for the stability of block revetments (response surfaces versus Steentoets, see also section 4.2).
2. All probabilistic calculations involved at least some degree of residual strength. This may partly explain the differences between the block thickness ratios for cases without residual strength from Table 9.1 and the calibrated partial safety factors presented in chapter 8.
3. Kaste and Klein Breteler (2014) assumed a different distribution function for the model uncertainty factor for columns from the one used in the calibration exercise (truncated normal versus shifted lognormal). The impact this may have on the partial safety factor on the block thickness is likely to be small, see also the sensitivity analyses in Jongejan et al. (2015b). This because the characterisation of the model uncertainty influences the outcomes of both probabilistic and semi-probabilistic assessments.
4. Kaste and Klein Breteler (2014) only considered a design load with an exceedance probability of 1/4.000 per year in their semi-probabilistic calculations with Steentoets.
5. Kaste and Klein Breteler (2014) assumed a cross-sectional failure probability of 0.01 x 1/4000 per year ($P_{T,cross}$=2.5x10^{-6} per year, or β_{Tcross}=4.56).

Table 9.1. Significant wave heights, residual strength classifications and ratios of the block thicknesses according to the probabilistic model (including residual strength) and Steentoets (without residual strength and without the use of a partial safety factor) (from Kaste and Klein Breteler, 2014). Results for the Western Scheldt.

Water system	**Significant wave (height H_s (m)**	**Thickness of clay layer d_c (m)**	**Dike width at storm surge level B_{dike} (m)**	**Residual strength class**	**Block thickness ratio (excl. second storm)**	**Block thickness ratio (incl. second storm)**
Western Scheldt	2.5	0.8	32	small	1,05	1,11
	3.0	0.8	39	small	1,07	1,14
	2.0	0.8	22	small	1,00	1,14
	1.0	0.8	11	medium	0,28	0,63
	2.5	0.2	32	small	1,04	n/a
	2.5	2.0	32	small	0,90	1,03
	2.5	0.8	45	small	0,85	n/a
	2.5	0.8	32	small	1,02	n/a
	2.5	0.8	32	small	1,05	1,12
	2.5	0.8	37	small	1,03	1,09

Table 9.2. Significant wave heights, residual strength classifications and ratios of the block thickness according to the probabilistic model (including residual strength) and Steentoets (without residual strength and without the use of a partial safety factor) (from Kaste and Klein Breteler, 2014). Results for the Wadden Sea.

Water system	**Significant wave (height H_s (m)**	**Thickness of clay layer d_c (m)**	**Dike width at storm surge level B_{dike} (m)**	**Residual strength class**	**Block thickness ratio (excl. second storm)**	**Block thickness ratio (incl. second storm)**
Wadden Sea	2.0	0.8	24	small	0,76	0,85
	2.5	0.8	31	small	0,66	0,82
	1.0	0.8	10	medium	0,30	0,58
	2.0	2.0	24	medium	0,10	0,46
	2.0	0.8	24	small	0,71	n/a
	2.0	0.8	24	small	0,77	0,87
	2.0	0.8	21	small	0,82	0,98

Table 9.3. Significant wave heights, residual strength classifications and ratios of the block thickness according to the probabilistic model (including residual strength) and Steentoets (without residual strength and without the use of a partial safety factor) (from Kaste and Klein Breteler, 2014). Results for Lake IJssel.

Water system	Significant wave (height H_s (m)	Thickness of clay layer d_c (m)	Dike width at storm surge level B_{dike} (m)	Residual strength class	Block thickness ratio (excl. second storm)	Block thickness ratio (incl. second storm)
Lake IJssel	1.5	0.8	16	small	1,09	1,20
	2.0	0.8	23	small	1,18	1,20
	1.0	0.8	11	medium	0,78	1,08
	1.5	2.0	16	large	0,69	0,87
	1.5	0.8	29	small	0,85	n/a
	1.5	0.8	16	small	1,07	n/a
	1.5	0.8	16	small	1,10	1,20
	1.5	0.8	16	small	1,14	1,20

The variations within each residual strength class stem from the fact that many more site-specific conditions determine the impact of residual strength than the ones used to define the residual strength classes in section 9.2.

From the computations, Klein Breteler (2015a) concluded that the required block thickness drops by at least a factor 1.1 when the residual strength increases from small to medium, and again by at least factor 1.1 when it increases from medium to large.[2] These subjective estimates are biased towards to safe side because of the limited number of calculations. While the changes are far greater in some cases, more daring conclusions would require further study. Such studies could also be tailored to site-specific conditions in the “toets op maat”. The cautious handling of residual strength is also in line with Rijkswaterstaat’s position on this topic: the factors mentioned above imply that the residual strength of the sand core has largely been kept ‘in reserve’. The procedure set out above can easily accommodate future refinements.

With the β_T-dependent safety for columns (see Table 8.3), the changes associated with a decrease in γ_β by a factor 1.1 and a factor 1.1x1.1=1.21 have been translated into reductions in cross-sectional target reliabilities. Note that P_{Norm}=1/4,000 per year corresponds to β_{Norm}=3.48 and $P_{T,cross}$=1/4,000 x 0.01=1/400,000 per year corresponds to β_{Tcross}=4.56 to (see points and 1 and 2 above). With these reductions, the β_T-dependent safety factors have been recalculated using the β_T-dependent safety for columns (see Table 8.3). The results of these recalculations are shown in Table 9.4 below (numerical results without rounding). Note that the same reductions have been applied to columns and blocks on their side.

[2] *For instance, for the cases along the Western Scheldt (Table 9.1), the differences in required block thicknesses for small and medium residual strength are, at a minimum, 0.85-0.28=0.57 without a second storm and 1.03-0.63=0.40 with a second storm. For the cases along the Wadden Sea (Table 9.2), these differences are 0.66-0.30=0.36 and 0.82-0.58=0.24 respectively. For the cases along Lake IJssel (Table 9.3), the differences in required block thicknesses for small and medium residual strength are, at a minimum, 0.85-0.78=0.07 and 1.2-1.08=0.12, and the differences in required block thicknesses for medium and large residual strength are, at a minimum, 0.78-0.69=0.09 and 1.08-0.87=0.21.*

Table 9.4. β_T-dependent safety factors including the effect of residual strength, for different length effect factors (assuming f=0.1 including corrections for correlations with overtopping as specified in calibration studies, λ_1=0.5 λ_2=0.9 and λ_3=0.7). Results for the default length effect factor N=4 are shown in bold. Results for non-existent maximum allowable probabilities of flooding have been greyed out (note: the probabilities that have been presented in the Delta Programme (2014) are one class (a factor 3) smaller than maximum allowable probabilities of flooding).

Type of blocks	**Water system**	**Maximum allowable probability of flooding (per year)**	**β_T-dependent safety factor**								
			Small residual strength			Medium residual strength			High residual strength		
			N=2	N=4	N=8	N=2	N=4	N=8	N=2	N=4	N=8
Columns	Western Scheldt	1/300	1,02	**1,06**	1,09	0,90	**0,94**	0,98	0,78	**0,83**	0,88
		1/1000	1,01	**1,05**	1,08	0,90	**0,94**	0,98	0,79	**0,84**	0,88
		1/3000	1,01	**1,04**	1,07	0,91	**0,94**	0,98	0,81	**0,85**	0,89
		1/10000	1,00	**1,03**	1,06	0,91	**0,94**	0,97	0,82	**0,85**	0,89
		1/30000	1,00	**1,03**	1,06	0,91	**0,94**	0,97	0,82	**0,86**	0,89
	Wadden Sea	1/300	1,03	**1,06**	1,09	0,91	**0,95**	0,98	0,79	**0,84**	0,88
		1/1000	1,01	**1,04**	1,07	0,90	**0,94**	0,97	0,80	**0,84**	0,88
		1/3000	1,00	**1,03**	1,06	0,90	**0,93**	0,96	0,81	**0,84**	0,88
		1/10000	0,99	**1,02**	1,04	0,90	**0,93**	0,95	0,81	**0,84**	0,87
		1/30000	0,99	**1,01**	1,04	0,90	**0,92**	0,95	0,81	**0,84**	0,87
	Lake IJssel	1/300	1,12	**1,18**	1,23	0,98	**1,05**	1,11	0,85	**0,92**	0,99
		1/1000	1,09	**1,14**	1,20	0,96	**1,02**	1,08	0,85	**0,91**	0,97
		1/3000	1,06	**1,11**	1,16	0,94	**1,00**	1,05	0,84	**0,90**	0,95
		1/10000	1,04	**1,09**	1,14	0,93	**0,98**	1,03	0,83	**0,89**	0,94
		1/30000	1,02	**1,07**	1,11	0,92	**0,97**	1,01	0,82	**0,87**	0,92
Blocks on their side	Western Scheldt	1/300	0,99	**1,04**	1,08	0,86	**0,91**	0,96	0,74	**0,80**	0,86
		1/1000	0,99	**1,03**	1,07	0,87	**0,92**	0,97	0,77	**0,82**	0,87
		1/3000	0,99	**1,03**	1,07	0,88	**0,92**	0,97	0,78	**0,83**	0,87
		1/10000	0,99	**1,03**	1,07	0,89	**0,93**	0,97	0,80	**0,84**	0,88
		1/30000	0,99	**1,03**	1,06	0,89	**0,93**	0,97	0,81	**0,85**	0,89

As shown in Table 9.4, the differences between the partial safety factors for adjacent residual strength classes still differ by about a factor 1.1, despite the underlying transformations.

10 Step 5: Establishing the partial safety factors

The β_T-dependent safety factors listed in Table 9.4 of the previous chapter are meant to inform a decision about partial safety factors for use in the WTI2017. It would be ill-advised to use these values or the underlying formulae directly. This is because, as previously noted, the design of a semi-probabilistic rule involves a trade-off between accuracy and practicality. The effort or complexity associated with greater accuracy is only worthwhile when greater accuracy may significantly influence the outcomes of safety assessments.

A proposal was developed/verified in an expert meeting with Ton Vrouwenvelder (TNO), Hans van der Sande (Scheldestromen), Mark Klein Breteler (Deltares) and Ruben Jongejan (RMC) (Deltares, 2015). It was proposed to adopt partial safety factors on the block thickness that only depend on the residual strength classification:

- Small residual strength: γ=1.1
- Medium residual strength: γ=1.0
- Large residual strength: γ=0.9

The abovementioned proposal rests on the following considerations:

1. Differences in safety factors of less than e.g. 5% lie within the uncertainty bandwidth surrounding the results presented in Table 9.4. Such small differences would also be of little practical relevance (5% corresponds to 1 cm on a block thickness of 20 cm, or 2cm on a block thickness of 40 cm).
2. Given the strong similarities of the partial safety factors across the different maximum allowable probabilities of flooding, there is little need for making the partial safety factors depend on the maximum allowable probabilities of flooding (or equivalently, the value of β_{Norm}).
3. Differentiating between Lake IJssel and the other water systems would lead to partial safety factors that are about 5% to 10% greater along Lake IJssel than elsewhere (see Table 9.4). The greatest differences would be found for the highest maximum allowable probabilities of flooding: 1/300 and 1/1000 per year. The associated representative hydraulic loads are significantly smaller than the representative loads for today's flood protection standards (exceedance frequencies of 1/4,000 and 1/10,000 per year along Lake IJssel). Note also that there is still a significant degree of conservatism related to the handling of residual strength. It has been assumed that the impact of residual strength is nil for all cases that fall within the lowest residual strength class. Furthermore, the conditional probabilities of flooding for medium and large residual strength rest on cautious estimates of the impact of residual strength: for Lake IJssel, the block thickness ratios from Table 9.1 for medium and large residual strength differ more than a factor 1.1 from the ratios for small residual strength. Because of this, no distinctions have been made between Lake IJssel and the other water systems.
4. The differences between the required block thicknesses for the subsequent residual strength classes in Table 9.4 is about 10%, which starts to be significant. The difference between the required block thicknesses for the small and large residual strength classes is about 20% (see Table 9.4).

A summary of the resulting semi-probabilistic assessment rule is given in Appendix H.

11 Step 6: Comparison with current practice

Ignoring the residual strength of filter and base layers leads to a semi-probabilistic assessment rule that is directly comparable to the WTI2006 and WTI2011 rules for assessing the stability of block revetments under wave attack. The partial safety factor for the block thickness in the WTI2006 and WTI2011 is equal to 1.0.

The present-day flood protection standards along Lake IJssel, the Wadden Sea and the Zeeland estuaries are exceedance probabilities of 1/4000 and 1/10000 per year. This means that safety assessments for columns have to be carried out with 1/4000 and 1/10000 per year loading conditions, using a partial safety factor of 1.0.

For new flood protection standards of 1/4000 and 1/10000 per year (maximum allowable probabilities of flooding), blocks would also have to be assessed with 1/4000 and 1/10000 per year loading conditions but with partial safety factors of 0.9, 1.0 or 1.1, depending on the amount of residual strength. Without any residual strength, blocks would have to be 10% thicker to pass a semi-probabilistic assessment with identical load and resistance models. Note that changes in the load model or Steentoets may also influence required block thicknesses.

It is stressed that the abovementioned 10% increase in the required block thickness only applies to cases that fall in the small residual strength class. For medium residual strength, the proposed new partial safety factor is equal to 1.0, identical to the value of the partial safety factor in the WTI2011. For large residual strength, the proposed new value is 0.9, allowing blocks to be 10% thinner. All in all, the proposed new partial safety factors are expected to lead to assessments that are broadly in line with previous assessments. Differences are most likely to come from differences in load models and safety standards. The differences are likely to work two ways: the new safety standards will sometimes lead to more stringent requirements and sometimes to less stringent ones.

Residual strength was covered by a separate assessment rule in the WTI2006 and WTI2011. It has, however, been ignored in past safety assessments for blocks on their side. For other types of revetments, residual strength is believed to have only sporadically influenced the outcomes of safety assessments. This means that the differences in the treatment of residual strength, is unlikely to influence the abovementioned conclusions related to the consequences of the introduction of the proposed new rule.

The calibrated partial safety factors should not be used for design purposes. The following aspects may necessitate the use of higher safety factors when designing block revetments:

1 (uncertainties related to) changes in the hydraulic loads during the expected lifetime of the block revetment,
2 (uncertainties related to) changes in the resistance of blocks due to e.g. degradation and clogging,
3 possible changes in failure mechanism models during the expected lifetime of the block revetment,
4 (uncertainties related to the) differences between the designed and constructed revetment.

For design purposes, present-day guidelines prescribe the use of a partial safety factor of 1.1. The Design Guide 2014 (RWS, 2013a; 2013b) recently proposed an increase to 1.2. The

Design Guide 2014 was written in a relatively short period of time so that it rests largely on expert judgment. The higher value of 1.2 was proposed to further reduce the probability of underinvestment. While the costs of using a partial safety factor of 1.2 rather than 1.1 in the design of a block revetment are relatively low, having to replace a revetment prematurely is very expensive. Hence, from an economic perspective, the use of a partial safety factor of 1.2 when designing new revetments does not appear unreasonable. Such a factor would be comparable to the “robuustheidstoeslag” that is used in the design of levees. The increase in the partial safety factor for the design of block revetments to 1.2 seems justified by the results of this calibration study: a partial safety factor of 1.2 creates a safe margin between the design of new revetments and assessments of existing ones.

12 Conclusions and recommendations

12.1 Conclusions

1 A new safety format has been developed that integrates assessments of the stability of the block revetment and subsequent erosion (or residual strength) into a single procedure. In the WTI2011, both aspects were evaluated independently. In the new safety format, the partial safety factor for the block thickness is adjusted for the amount of residual strength. This allows for less conservatism in semi-probabilistic assessments. Three different residual strength classes have been defined, with a different partial safety factor assigned to each class.

2 The representative values for block revetment parameters have been defined in such a manner that the parameter values stored in databases can be re-used as representative values. Because these representative values are close to their design point values, the choice for these practical representative values does not harm the accuracy of the semi-probabilistic rule.

3 The representative load should ideally be a load with an exceedance probability that is an order of magnitude smaller than the value of the flood protection standard. However, for reasons of consistency across failure mechanisms and historical comparability, the exceedance probability of the representative load has been set equal to the value of the flood protection standard. This leads to somewhat higher calibrated partial safety factors along Lake IJssel than along the Western Scheldt and Wadden Sea. The differences are greatest (up to about 10%) for the lowest standards (1/300 and 1/1000 per year). The differences may be explained by the differences between load distributions and the relatively small required block thicknesses along Lake IJssel. In an expert meeting, differentiating between Lake IJssel and the other water systems was judged to be impractical and of little practical relevance. This is why the proposed partial safety factors are the same for all water systems.

4 The calibrated partial safety factor for the block thickness is virtually independent of the flood protection standard. This is because the representative value of the hydraulic load is tied to the flood protection standard. A more stringent flood protection standard thereby leads to a higher representative load. Because of the dominance of the uncertainty related to the hydraulic load, the change in the representative load is sufficient to offset the effect of a more stringent flood protection standard.

5 Residual strength (i.e. the time to failure of filter and base layers, geotextile and the remainder of the levee) may strongly impact the probability of failure. Given the controversy surrounding residual strength (as indicated by Rijkswaterstaat) and the present state of knowledge, the outcomes of residual strength computations have been interpreted with caution. The format of the semi-probabilistic assessment rule is such that it will be directly visible to what extent residual strength plays a role in the outcomes of semi-probabilistic assessments. The format can also easily accommodate further refinements, possibly within the context of an advanced assessment (“toets op maat”).

6 The proposed new partial safety factors rest on calibration studies for two types of block revetments and a study into the effect of residual strength. While not all types of blocks

have been studied, the calibrated partial safety factors are believed to be broadly applicable to block revetments. This is because the models for the other block types rest on similar theories and conservative assumptions and because the results for columns and blocks on their side are similar, despite the different behaviours of these blocks under wave attack.

7 The proposed new partial safety factors for the WTI2017 range from 0.9 (for large residual strength) to 1.1 (for small residual strength). The WTI2011-assessment rule contains a partial safety factor of 1.0, independent of the amount of residual strength. Representative values are defined similarly. The proposed new partial safety factors thus do not lead to a radical departure from status quo.

A summary of the calibrated semi-probabilistic assessment rule is given in Appendix H.

12.2 Recommendations

1 The residual strength classes and their link with the proposed partial safety factors rest on a cautious interpretation of a study into the effects of the time to failure of e.g. filter and base layers on probabilities of failure (Kaste and Klein Breteler, 2014). Additional probabilistic analyses may make it possible to enlarge the differences between the partial safety factors associated with the different residual strength classes. The safety format can easily accommodate such changes. It is recommended to carry out such studies within the context of advanced assessments (the “toets op maat”) and to keep track of their outcomes to facilitate future improvements.

2 Using the calibrated partial safety factors for design purposes is strongly advised against. This is because (the uncertainties related to) changes in the hydraulic loads, resistance variables and models should not be ignored when designing revetments (see also RWS, 2013a; 2013b). For design purposes, the use of higher partial safety factors is recommended, such as the partial factor of 1.2 that is proposed by the Design Guide 2014 (RWS, 2013a; 2013b).

3 It is recommended to consider a wider variety of block revetments in future updates of this study, and to treat residual strength as an integral part of the failure mechanism in the calibration procedure. A step-wise procedure may give rise to suboptimal results.

4 This report dealt with the stability of block revetments under wave attack and subsequent erosion. Other failure mechanisms for block revetments, such as material transport and erosion caused by parallel flow, have not been studied. It is recommended to also calibrate the assessment rules for these failure mechanisms and/or to evaluate and document their adequacy in the light of the new reliability requirements.

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A The test set

The calibration studies for columns and blocks on their side rest on a large number of probabilistic analyses for different cases. This Appendix presents an overview of the characteristics of these cases. Section A.1 and A.2 discusses the block revetment parameters, section A.3 the hydraulic loading parameters.

A.1 Block revetment characteristics: columns

Table A.1 provides an overview of the block revetment parameters that were varied and the average values that were assumed. The values that are underlined are most common in practice. For the outer slope of the dike, all three values are equally predominant. With these parameter values, the total number of test cases (per water system) is 81.

Table A.1 Parameter ranges (Basalton blocks)

Parameter		Average values
Outer slope	$\cot\alpha$ [-]	3.0, 3.5, 4.0
Thickness of the filter layer	b_f [m]	<u>0.1</u>, 0.2, 0.3
Density of the blocks	ρ_S [kg/m^3]	<u>2300</u>, 2600, 2900
Grain size of the filter material	D_{f15} [m]	0.01, <u>0.02</u>, 0.03

Table A.2 lists the parameter values that were considered for Basalton blocks along the Western Scheldt, the Wadden Sea and Lake IJssel. All parameter values were selected by block revetment specialists (M. Klein Breteler and D. Kaste).

Table A.2 Parameter values for Basalton blocks

Parameter	Unit	Symbol in software code	Distr. Type*	Mean	Standard deviation	Coeff. of variation	Lower limit
General parameters							
gravitational acceleration	[m/s²]	g	D	9.81			
kinematic viscosity of water	[m²/s]	nu	D	1.20E-06			
density of the water	[kg/m³]	rohW	D	1025			
Dike Geometry (Western Scheldt)							
thickness of the clay layer	[m]	dc	T	0.8		0.1	0.1
outer slope	[-]	cotau	N	**Range**		0.04	
height of the berm relative to NAP	[m]	zBerm	N	5.5	0.05		
slope of the berm	[-]	cotab	N	20		0.04	
width of the berm	[m]	Bb	N	5		0.02	
slope of the foreshore	[-]	tanaBodem	N	0.02		0.1	
height of the dike toe relative to NAP	[m]	zBodem	N	-5	0.2		
Dike Geometry (Wadden Sea)							
thickness of the clay layer	[m]	dc	T	0.8		0.1	0.1
outer slope	[-]	cotau	N	**Range**		0.04	
height of the berm relative to NAP	[m]	zBerm	N	5	0.05		

Parameter	Unit	Symbol in software code	Distr. Type*	Mean	Standard deviation	Coeff. of variation	Lower limit
slope of the berm	[-]	cotab	N	20		0.04	
width of the berm	[m]	Bb	N	5		0.02	
slope of the foreshore	[-]	tanaBodem	N	0.02		0.1	
height of the dike toe relative to NAP	[m]	zBodem	N	0	0.2		
Dike Geometry (Lake IJssel)							
thickness of the clay layer	[m]	dc	T	0.8		0.1	0.1
outer slope	[-]	cotau	N	**Range**		0.04	
height of the berm relative to NAP	[m]	zBerm	N	1.8	0.05		
slope of the berm	[-]	cotab	N	20		0.04	
width of the berm	[m]	Bb	N	5		0.02	
slope of the foreshore	[-]	tanaBodem	N	0.02		0.1	
height of the dike toe relative to NAP	[m]	zBodem	N	-3	0.2		
Block revetment parameters							
type of the blocks	[-]	type	D	2			
indicator of infilling material	[-]	fill	D	1			
block thickness	[m]	D	N	**Varied**	0.0013		
width of the blocks	[m]	B	N	0.3	0.001		
length of the blocks	[m]	L	N	0.3	0.001		
open surface of the revetment	[-]	omega	T	0.12		0.14	0
density of the blocks	[kg/m³]	rohS	N	**Range**		0.01	
thickness of the filter layer	[m]	b1	T	**Range**		0.1	0.01
porosity of the filter material	[-]	nf1	N	0.35		0.09	
grain size of the filter material	[m]	Df151	N	**Range**		0.06	
porosity of the infilling material	[-]	ni	D	0.7			
grain size of the infilling material	[m]	Di15	N	0.007		0.17	
Model factors							
model factor for the calculation of the failure of the block revetment for concrete columns	[-]	mBR	L	1.42	0.18		0.87

** N = Normal distribution, L = Lognormal distribution, T = truncated normal distribution, D = deterministic variable.*

A.2 Block revetment characteristics: blocks on their side

Table A.1 provides an overview of the block revetment parameters that were varied and the average values that were assumed. These variables were selected by block revetment specialists (D. Kaste and M. Klein Breteler). With these parameter values, the total number of test cases (per water system) equals 27.

Table A.3. Parameter ranges.

Parameter	Unit	Symbol in software code	Average values
Outer slope	[-]	cotau	3.0, 3.5, 4.0
Thickness of the filter layer	[m]	b1	0.1, 0.2, 0.3
Level of the top of the revetment relative to NAP	[m]	zB	2.5, 4, 5,5

Table A.2 lists the parameter values that were considered for blocks on their side. All parameter values were selected by block revetment specialists (M. Klein Breteler and D. Kaste).

Table A.4. Parameter values for blocks on their side.

Parameter	Unit	Symbol in software code	Distr. Type*	Mean	Standard deviation	Coeff. of variation	Lower limit
General parameters							
gravitational acceleration	[m/s²]	g	D	9.81			
kinematic viscosity of water	[m²/s]	nu	D	1.20E-06			
density of the water	[kg/m³]	rohW	D	1025			
Dike Geometry							
thickness of the clay layer	[m]	dc	T	0.8		0.1	0.1
outer slope	[-]	cotau	N	**Range**		0.04	
height of the berm relative to NAP	[m]	zBerm	N	5.5	0.05		
slope of the berm	[-]	cotab	N	20		0.04	
width of the berm	[m]	Bb	N	5		0.02	
slope of the foreshore	[-]	tanaBodem	N	0.02		0.1	
height of the dike toe relative to NAP	[m]	zBodem	N	-5	0.2		
Block revetment parameters							
type of the blocks	[-]	type	D	3			
indicator of infilling material	[-]	fill	D	0			
block thickness	[m]	D	N	**Varied**	0.0013		
width of the blocks	[m]	B	N	0.25	0.001		
length of the blocks	[m]	L	N	0.5	0.001		
Joint width of vertical joints	[m]	ss	N	0.00365	0.0004		
density of the blocks	[kg/m³]	rohS	N	2370	22		
thickness of the filter layer	[m]	b1	T	**Range**		0.1	0.01
Level of the top of the revetment relative to NAP	[m]	zB	N	**Range**	0.05		
level of the low border over NAP (transition structure)	[m]	zO	N	-3	0.05		
porosity of the filter material	[-]	nf1	N	0.35		0.09	
grain size of the filter material	[m]	Df151	N	0.0052		0.0006	
porosity of the infilling material	[-]	ni	D	0			
grain size of the infilling material	[m]	Di15	D	0			
Model factors							

Parameter	Unit	Symbol in software code	Distr. Type*	Mean	Standard deviation	Coeff. of variation	Lower limit
model factor for the calculation of the failure of the block revetment for concrete columns	[-]	mBR	L	1.38	0.366		0.87

** N = Normal distribution, L = Lognormal distribution, T = truncated normal distribution, D = deterministic variable.*

A.3 Simplified load models

Steentoets requires information about the hydraulic loading conditions during storm events. A probabilistic load model capable of providing such information is still under development. A proof of concept has recently been provided (Morris & Smale, 2014), but further research and development is needed before it can be used for a calibration exercise.

Because of the absence of a suitable load model for probabilistic computations with the Steentoets model, simplified load models were used. These models are characteristic for three different water systems: the Western Scheldt (near Vlissingen and 50km eastwards), Wadden Sea (near Harlingen) and Lake IJssel (near Urk). They have previously been used in studies by Kaste and Klein Breteler (2012; 2014).

The fact that the simplified load model gives inaccurate predictions of the hydraulic loads at actual locations is acceptable for the purpose of calibrating safety factors. This is because the actual location of (arbitrarily selected/generated) test set members is largely irrelevant to the outcomes of a calibration exercise, as a partial safety factor is supposed to be broadly applicable. The calibrated partial safety factors also appear insensitive to changes in the hydraulic loading conditions, as shown by a number of sensitivity analyses (see Appendix C).

According to the simplified load models, the water level and wave conditions during a specific storm are characterised by:

- The maximum water level (h_{max})
- The water level as function of time (hydrograph)
- The maximum wave height ($H_{s,max}$)
- The wave height as function of time (proportional to the hydrograph)

The maximum water level during a storm is modelled by a conditional Weibull distribution, as proposed by Roskam et al (2000). This distribution’s parameter values are given in Table A.5 for the selected location.

Table A.5. Parameters of the conditional Weibull distribution.

Location	Parameters of the Conditional Weibull distribution			
	Threshold relative to NAP (m)	Annual exceedance frequency of threshold	Shape	Scale (m)
Western Scheldt (west)	2.90	3.907	1.040	0.2793
Western Scheldt (east)*	3.30	3.845	0.870	0.1563
Wadden Sea	2.00	5.715	2.17	1.55
Lake IJssel	0.0386	7.023	0.9117	0.1137

** Only considered in a sensitivity study for blocks on their side.*

The water level distributions are based on the HR2006. They include the effects of storms and tides. The relationship between the significant wave height at the top of the storm $H_{s,max}$ and the water level at the top of the storm is based on Bretschneider-calculations:

For the location in the west of the Western Scheldt:

$$H_{s,\max} = 1.305 \cdot \left(0.48 \cdot h_{max} - 0.58\right) \tag{A1.1}$$

where

$H_{s,\max}$ Significant wave height at the top of the storm [m]
$h_{\max}$ Water level at the top of the storm relative to NAP [m]

For the location in the east of the Western Scheldt:

$$H_{s,\max} = 0.870 \cdot \left(0.48 \cdot h_{max} - 0.58\right) \tag{A1.2}$$

For the Wadden Sea:

$$H_{s,\max} = 0.913 \cdot \left(0.60 \cdot h_{Max} - 0.75\right) \tag{A1.3}$$

For Lake IJssel:

$$H_{s,\max} = 1.115 \cdot \left(1.05 \cdot h_{Max} - 0.23\right) \tag{A1.4}$$

The following simplified relations apply to the significant wave heights during a storm:

For the Western Scheldt:

$$H_s = H_{s,\max} - 0.3 \cdot \frac{h_{\max} - h}{2} \tag{A1.5}$$

where

$H_{s,\max}$ Significant wave height at the top of the storm [m]
$h_{\max}$ Water level at the top of the storm relative to NAP [m]
H_s Significant wave height at a particular moment during the storm, at water level h [m]
h Water level at a particular moment during the storm relative to NAP [m]

For the Wadden Sea:

$$H_s = H_{s,\max} - 0.7 \cdot \frac{h_{\max} - h}{2} \tag{A1.6}$$

For Lake IJssel:

$$H_s = H_{s,\max} - 0.7 \cdot \frac{h_{\max} - h}{2} \tag{A1.7}$$

The simplified breaker criterion was used:

$$H_s \leq 0.5 \cdot (h - z_{bottom}) \tag{A1.8}$$

where
H_s Significant wave height at the toe of the dike [m]
h Water level relative to NAP [m]
z_{Bottom} Bed level relative to NAP [m]

The time dependency of the loading conditions was modelled by fixed hydrographs, i.e. hydrographs that are the same for every storm event. Examples are shown in Figure A.1 for the Western Scheldt, in Figure A.2 for the Wadden Sea and in Figure A.3 for Lake IJssel. The lower limits on the y-axes correspond to the foreshore levels.

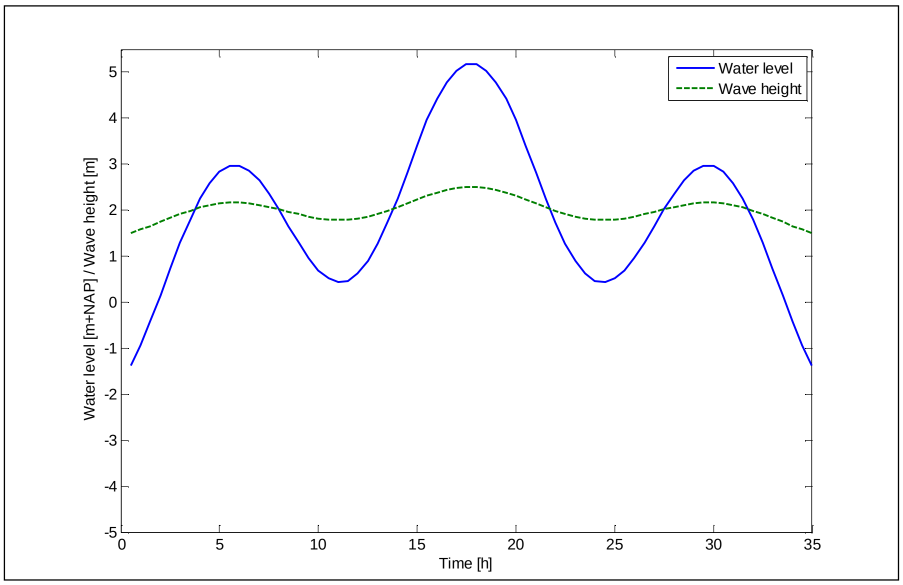

Figure A.1 Example of the development of the water level and significant wave height during a storm event in the Western Scheldt

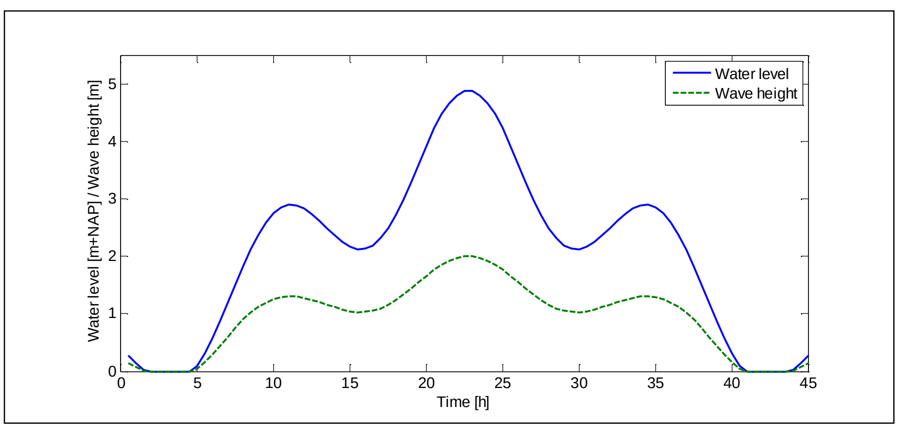

Figure A.2 Example of the development of the water level and significant wave height during a storm event in the Wadden Sea

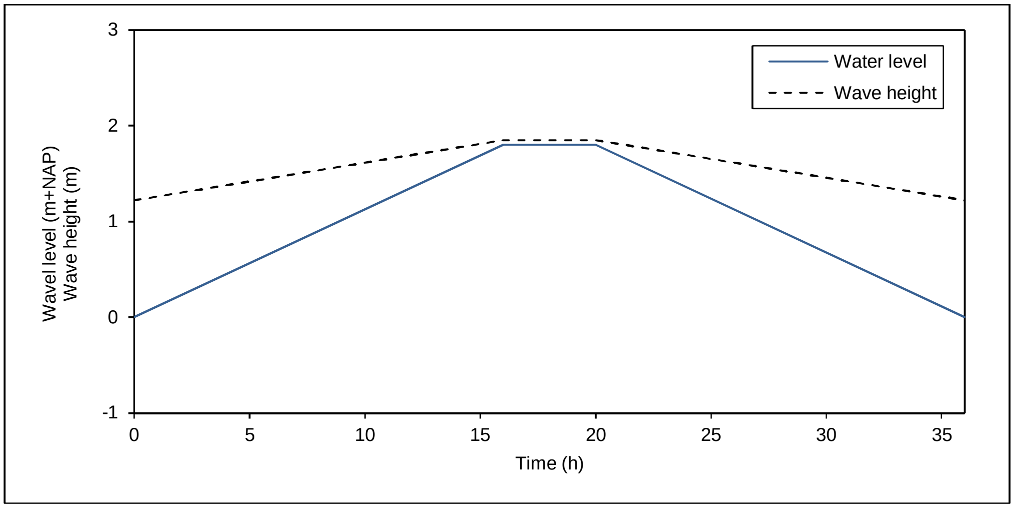

Figure A.3 Example of the development of the water level and significant wave height during a storm event in Lake IJssel.

B Results of the calibration studies

This Appendix gives an overview of the results of the calibration studies that have been carried out for columns and blocks on their side (Jongejan et al., 2015a; 2015b).

B.1 Influence coefficients

The relative importance of the uncertainties related to stochastic variables can be expressed in terms of influence coefficients (see also section 2.1). An inspection of influence coefficients provides useful clues about appropriate representative values (quantiles) and/or the variables for which partial safety factors should be introduced. The following figures show the squared influence coefficients for (groups of) stochastic variables, for test set members with a reliability index in the order of 4.3 ($4.1<\beta<4.5$).

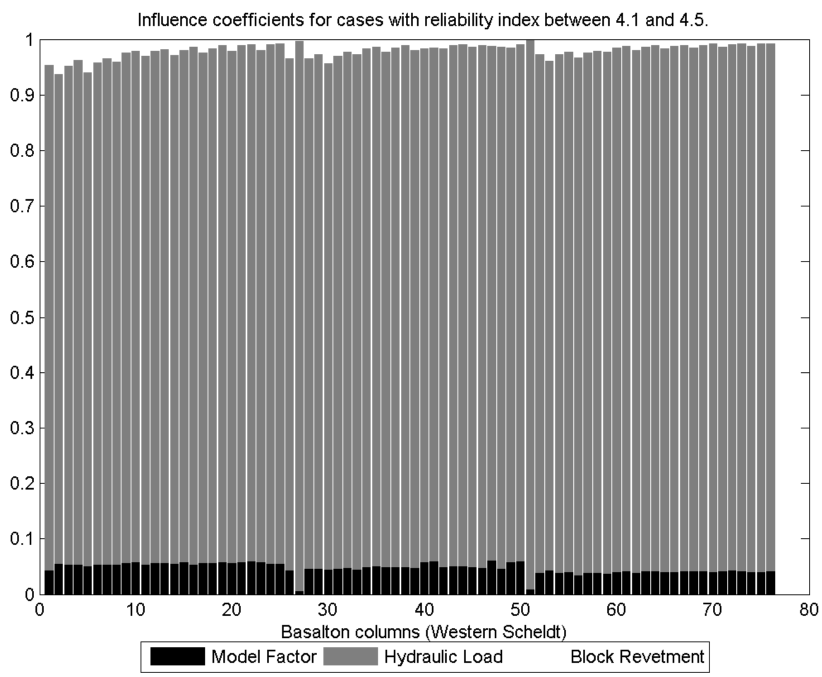

Figure B.1 Columns: squared influence coefficients per test set member at the Western Scheldt, for (4.1<β<4.5)

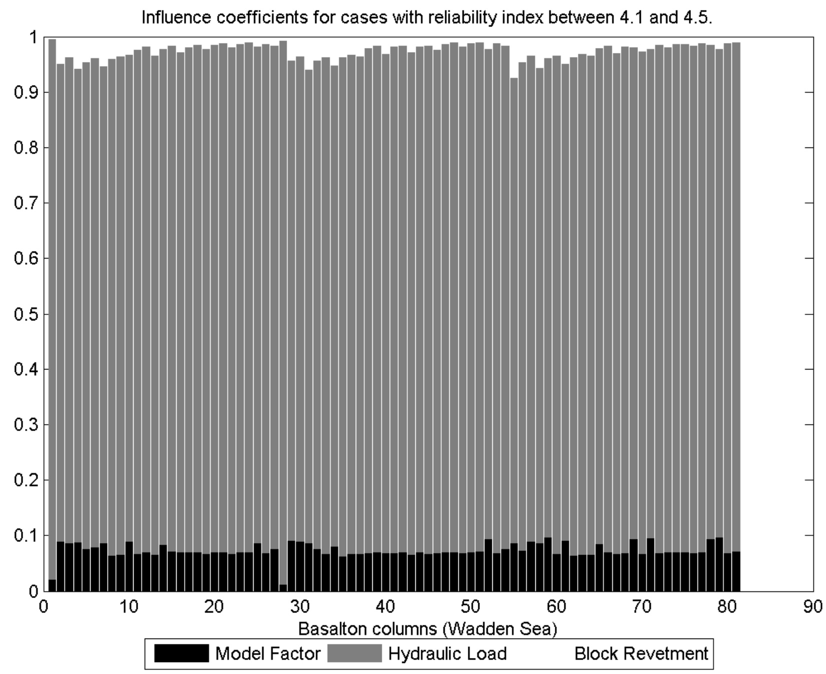

Figure B.2 Columns: squared influence coefficients per test set member at the Wadden Sea, for (4.1<β<4.5)

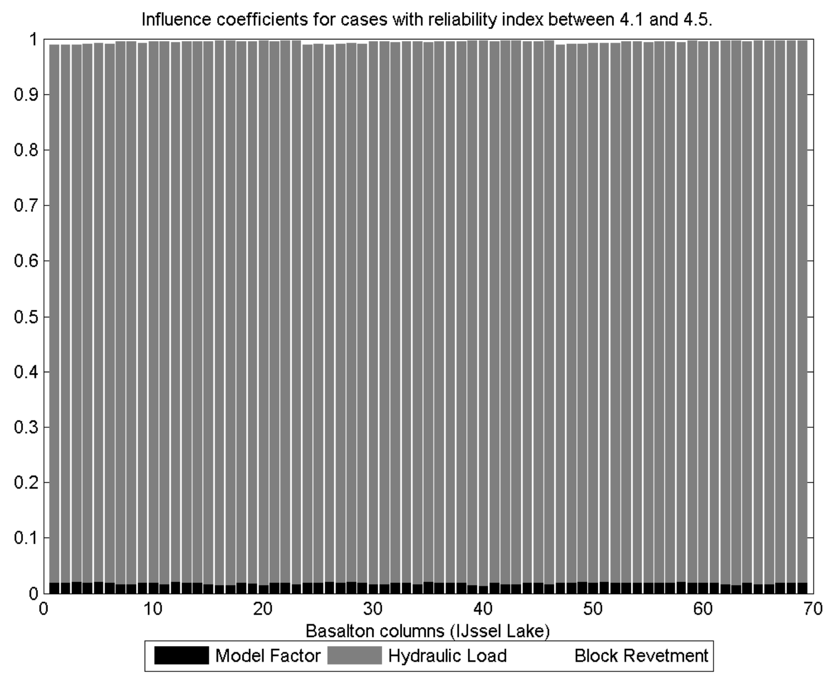

Figure B.3 Columns: squared influence coefficients per test set member at Lake IJssel, for (4.1<β<4.5)

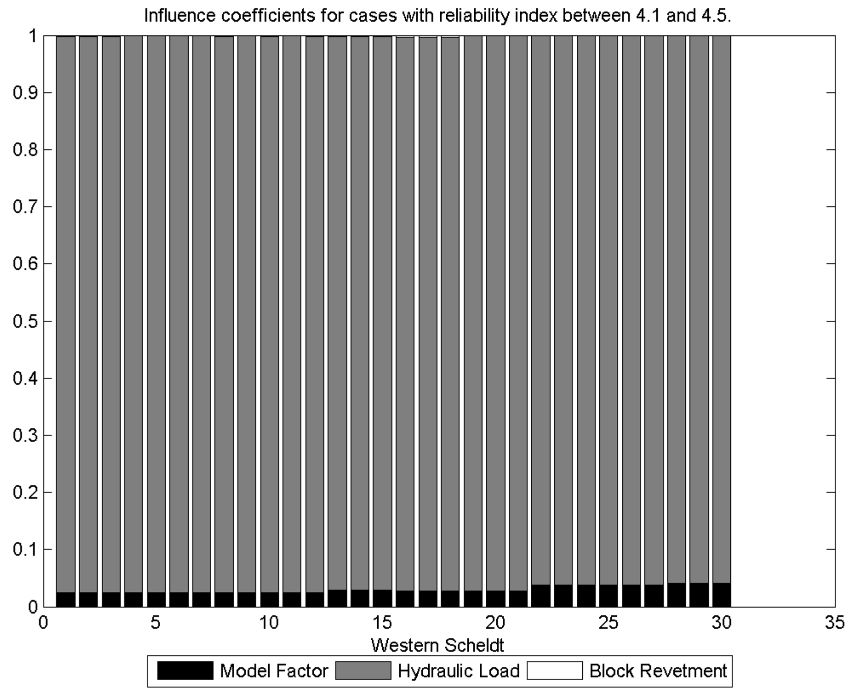

Figure B.4 Blocks on their side: squared influence coefficients per test set member for the Western Scheldt, for (4.1<β<4.5).

For both columns and blocks on their side, the uncertainty related to the hydraulic loading conditions appears to be dominant. The squared influence coefficients for the block revetment parameters (dimensions, geometry and weight) are very small. This can be attributed to the fact that these parameters are relatively well known/their variance is relatively small.

The relative importance of model uncertainty is similar for columns and blocks on their side, and fairly constant across water systems. The squared influence coefficient of the model uncertainty parameter (α_m^2) is around 0.05 for the Western Scheldt (so that α_m=0.22), it is around 0.08 for the Wadden Sea (so that α_m=0.28) and 0.02 for Lake IJssel (so that α_m=0.14). For blocks on their side, the squared influence coefficient of the model uncertainty parameter (α_m^2) is around 0.03 (so that α_m=0.17).

B.2 Partial safety factors and associated reliabilities

The greater the value of the partial safety factor on the block thickness, the greater the required block thickness and the greater the reliability index. This is shown in Figure B.5 to Figure B.8 for columns and in Figure B.9 for blocks on their side.

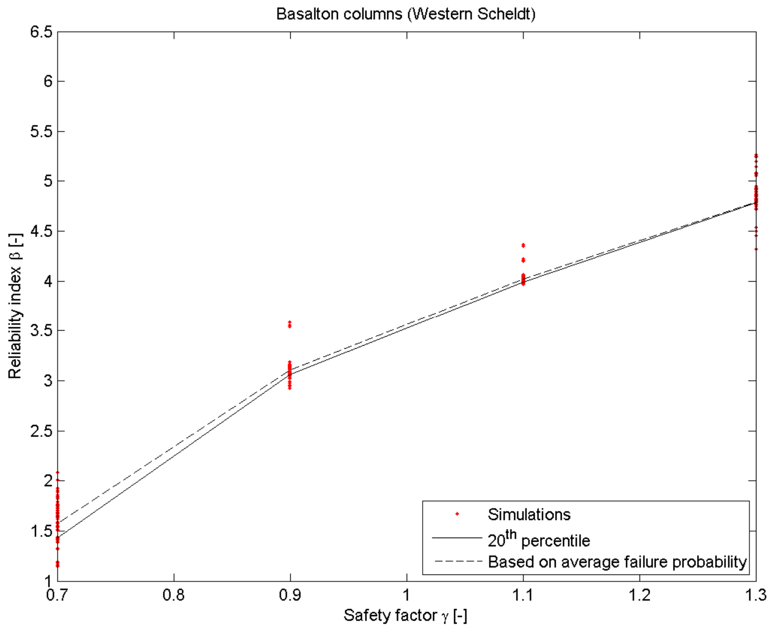

Figure B.5 The reliability index as a function of the partial safety factor for cases along the Western Scheldt, for a safety standard of 1/300 per year

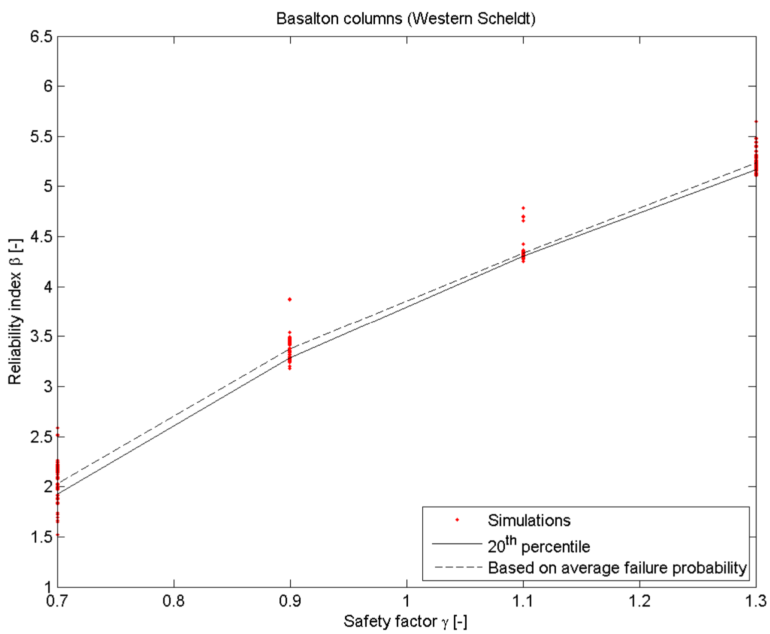

Figure B.6 The reliability index as a function of the partial safety factor for cases along the Western Scheldt, for a safety standard of 1/1000 per year

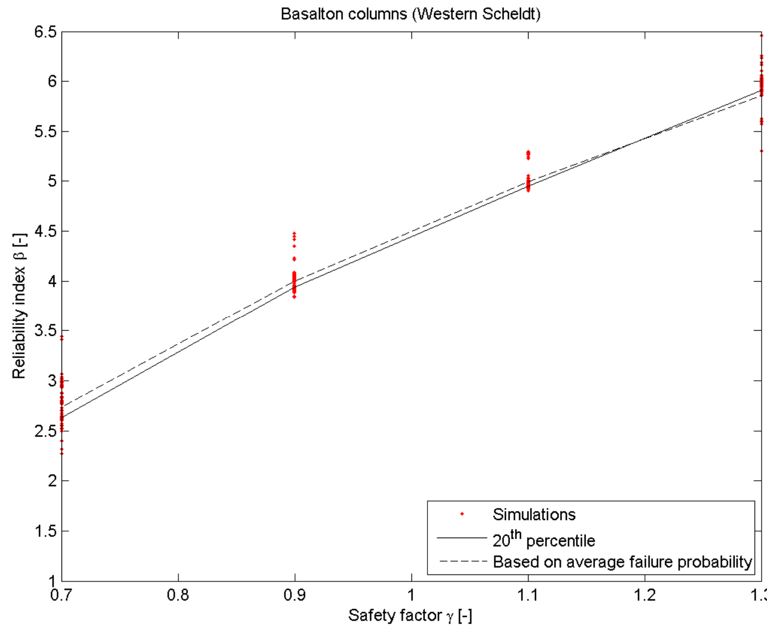

Figure B.7 The reliability index as a function of the partial safety factor for cases along the Western Scheldt, for a safety standard of 1/10000 per year

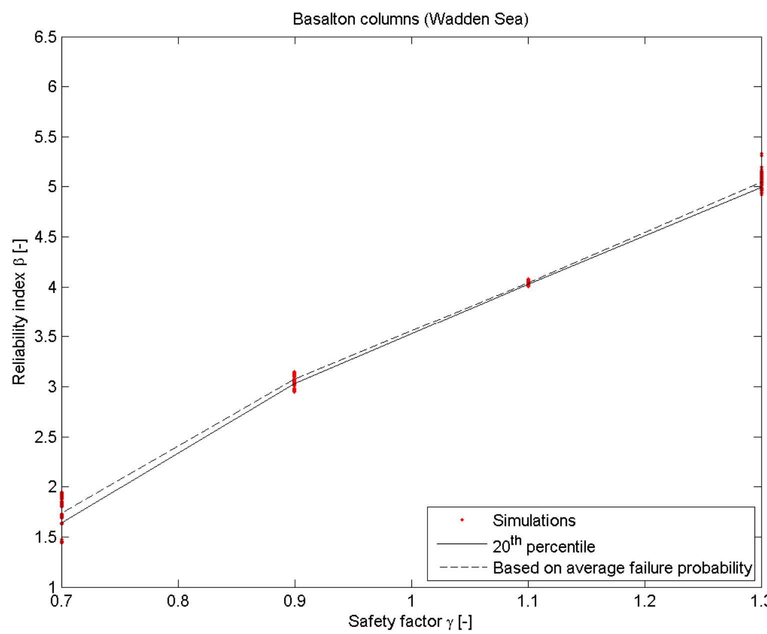

Figure B.8 The reliability index as a function of the partial safety factor for cases along the Wadden Sea, for a safety standard of 1/300 per year

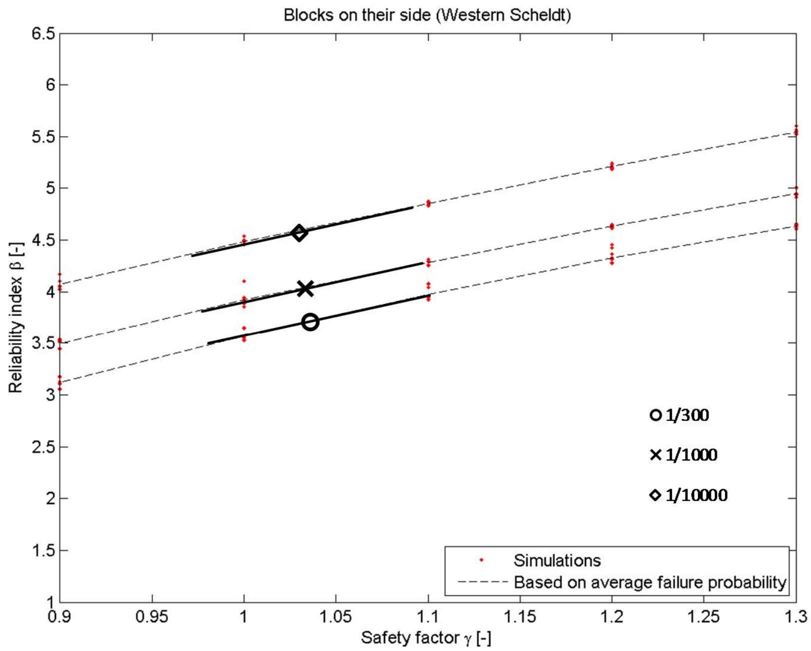

Figure B.9 Reliability indices as a function of the partial safety factor for the Western Scheldt, for hydraulic loads with exceedance probabilities of 1/300, 1/1000 and 1/10000 per year. The cross-sectional target reliabilities corresponding to safety standards of 1/300, 1/1000 and 1/10000 per year have been marked.

C Sensitivity analyses

The calibrated partial safety factors presented in chapter 8 (Table 8.3) are relatively insensitive to changes in input variables. The results of a number of sensitivity analyses are presented in this Appendix, for columns (section C.1) and blocks on their side (section C.2).

C.1 Sensitivity analyses for columns

C.1.1 Reducing the variability of the water levels along Lake IJssel

For various locations along Lake IJssel, a higher significant wave height does not necessarily come with a substantially higher water level. To obtain insight into the effect this may have on the adequacy of the calibrated partial safety factors, probabilistic and semi-probabilistic assessments were carried out assuming a 2 times lower peak water level while maintaining the distribution of the significant wave height, so that:

$$H_{s,\max} = 1.115 \cdot \left(1.05 \cdot \left(2 \cdot h_{Max}\right) - 0.23\right) \qquad \text{(C1.1)}$$

and

$$H_s = H_{s,\max} - 0,7 \cdot \frac{2 \cdot \left(h_{\max} - h\right)}{2} \qquad \text{(C1.2)}$$

For 7 cases (cross sections), the required block thicknesses were calculated with both the original and the modified load model, for safety factors ranging from 0.9 to 1.3. Reliability indices were subsequently calculated for each of these cases. All probabilistic analyses were carried out with FORM. No convergence issues were experienced. Results are shown in Figure C.1.

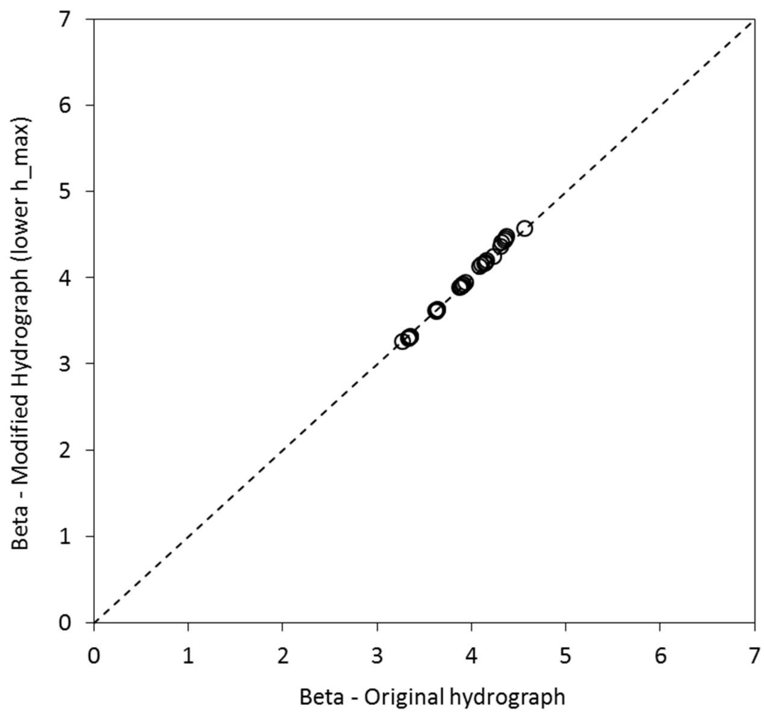

Figure C.1 Comparison of the outcomes of calibration exercises with the original load model (horizontal axis) and the modified load model (2 times smaller peak water level; vertical axis)

Figure C.1 shows that the impact of the changes to the load model on the outcomes of the calibration exercise is negligible. When wave loads become concentrated at a particular height, the revetment will fail under less extreme wave conditions. This implies that the failure probability of the revetment will go up. To avoid this from happening, the block thickness has to be increased. Since the overall effect of these changes on the influence coefficients of the different stochastic variables is negligible, the end-result is a negligible change in the required safety factor/the reliability implied by a particular safety factor.

C.1.2 Increasing the duration of the peak water level

When the peak water level lasts longer, the wave loads also become concentrated at a particular level. To obtain insight into the effects this may have on the adequacy of the calibrated partial safety factors, probabilistic and semi-probabilistic assessments were carried out assuming a 3 times longer peak water level during each storm event (12 hours instead of 4 hours).

For 7 cases (cross sections), the required block thicknesses were calculated with both the original and the modified load model, for safety factors ranging from 0.9 to 1.3. Reliability indices were subsequently calculated for each of these cases. All probabilistic analyses were carried out with FORM. No convergence issues were experienced. Results are shown inFigure C.2.

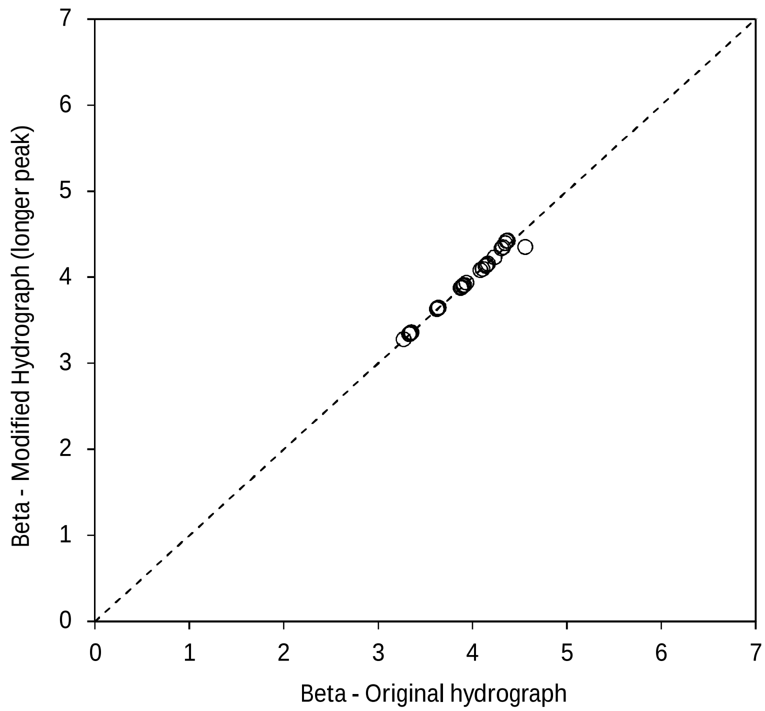

Figure C.2 Comparison of the outcomes of calibration exercises with the original load model (peak water level duration of 4 hours; horizontal axis) and the modified load model (peak water level duration of 12 hours; vertical axis)

Figure C.2 shows that the impact of a longer duration of the peak water level on the outcomes of the calibration exercise is negligible. This is in line with the results shown in section C.4.1.

C.2 Sensitivity analyses for blocks on their side

C.2.1 Using the load model for the eastern part of the Western Scheldt

The use of the load model for a location in the eastern part of the Western Scheldt gives results that are virtually identical to those for the location in the western part (Vlissingen). This is shown in Figure C.3 below. This may be explained by the fact that the load model influences the outcomes of both the probabilistic and the semi-probabilistic assessment: a less favourable load model leads to a greater probability of failure, but also to a less favourable semi-probabilistic assessment.

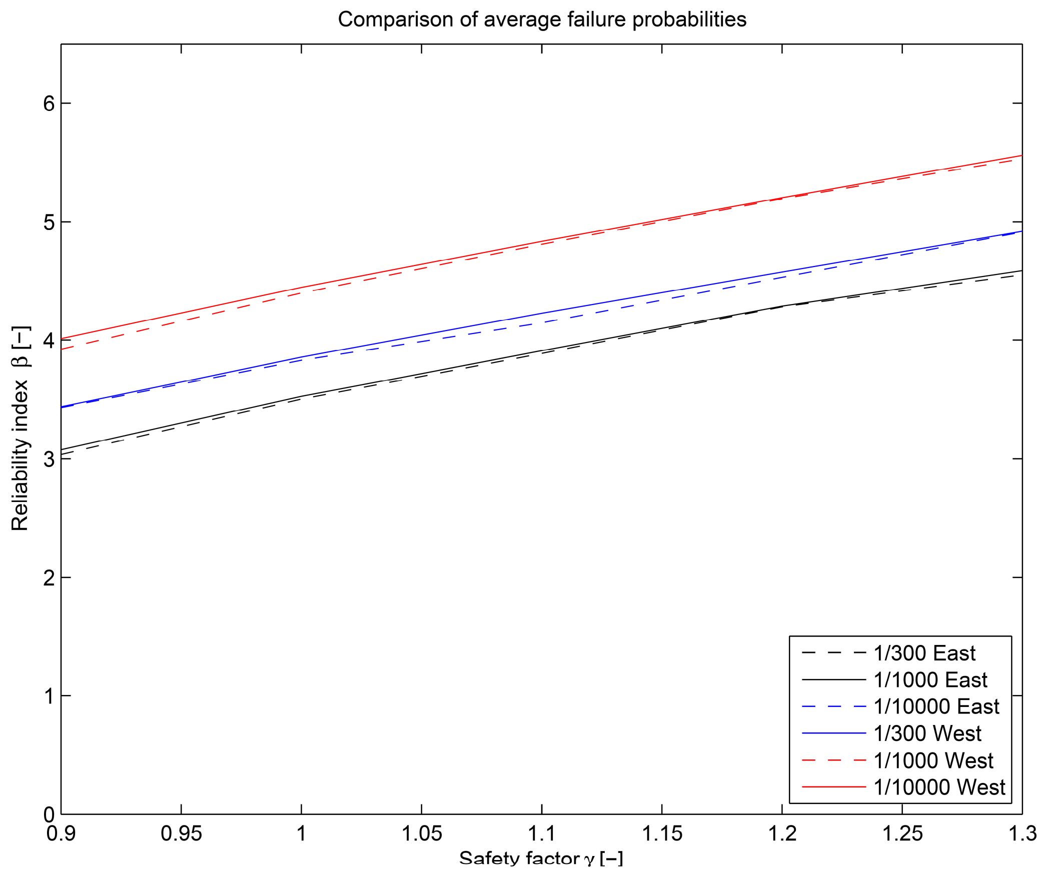

Figure C.3. The load models for the eastern and western part of the Western Scheldt give virtually identical results.

Because of the strong similarities between the results for the different locations, it was decided to only present the results for the western part of the Western Scheldt in the main text and in the remainder of this Appendix, for reasons of simplicity.

C.3 Varying the model uncertainty parameter

To evaluate the sensitivity of the calibrated semi-assessment rule to the distribution of the model uncertainty parameter, the required block thicknesses were calculated for the following lognormal distribution functions:

1 μ=1.380, σ=0.336 and lower limit=0.87 (base case)
2 μ=1.380, σ=0.288 and lower limit=0.87 (optimistic)
3 μ=1.314, σ=0.312 and lower limit=0.87 (conservative)

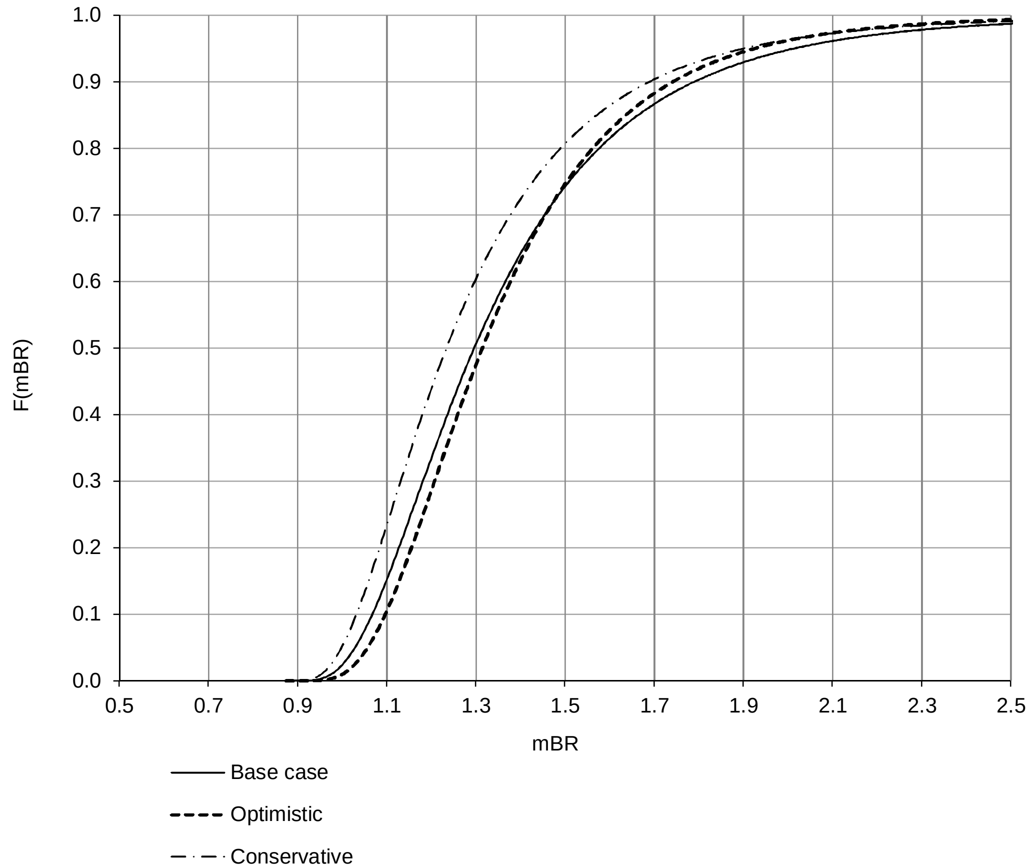

Figure C.4. Alternative distribution functions of the model factor.

The required block thicknesses was calculated for each test case, for safety factors ranging from 0.9 to 1.3 and flood protection standards of 1/300, 1/1000 and 1/10,000 per year. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. No convergence issues were experienced. Results are shown in Figure C.5 and Figure C.6.

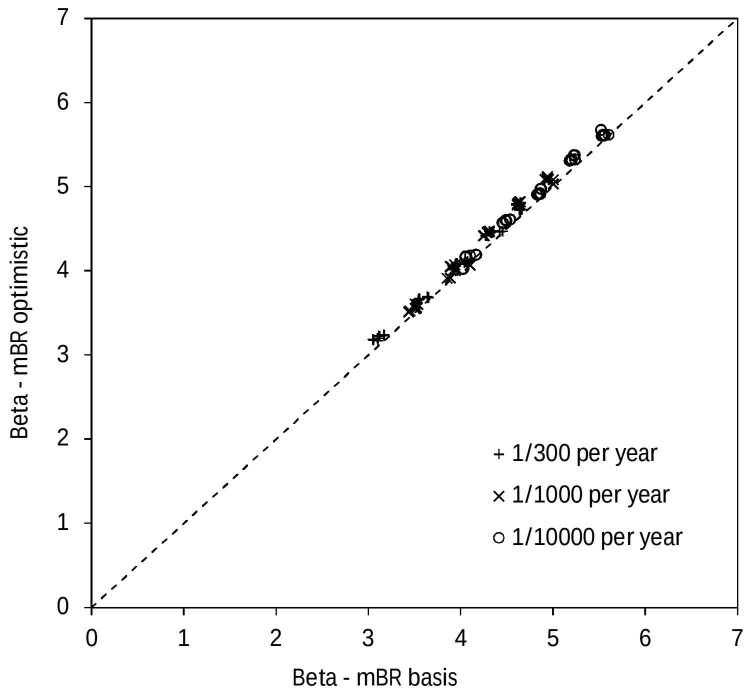

Figure C.5. Comparison of the outcomes of calibration exercises with the original model factor (base case, horizontal axis) and an optimistic model factor (vertical axis).

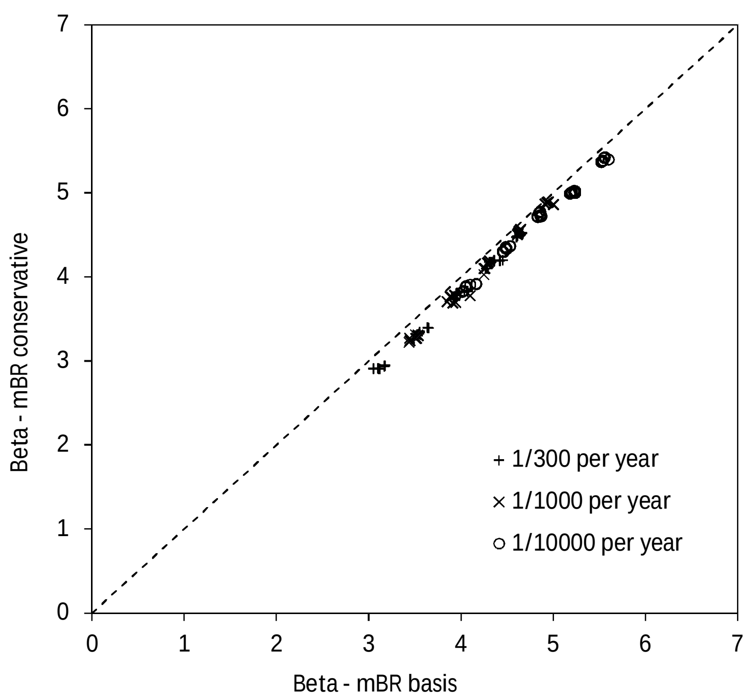

Figure C.6. Comparison of the outcomes of calibration exercises with the original model factor (base case, horizontal axis) and a conservative model factor (vertical axis).

Figure C.5 and Figure C.6 show that the reliability implied by a particular safety factor is rather insensitive to the distribution function of the model factor. This is because the model factor influences the outcomes of both the probabilistic and the semi-probabilistic assessment: a less favourable model factor leads to a greater probability of failure, but also to a stricter model factor in a semi-probabilistic assessment.

While the reliability implied is rather insensitive to the distribution function of the model factor, the calibrated partial safety factors are slightly different for the different distribution functions, as shown below. It is important to keep in mind however that such (minor) differences are insignificant in terms of the associated reliabilities, as illustrated by Figure C.5 and Figure C.6 above.

Calibrating a partial safety factor with an optimistic model factor

The required block thicknesses and corresponding reliability indices have been calculated for a range of values of the overall safety factors. Figure C.7 shows the calculated reliability indices for the different test set members (red dots), the corresponding average failure probabilities, as well as the resulting β_T-dependent safety factor (or overall safety factor) within the range of β_T of interest.

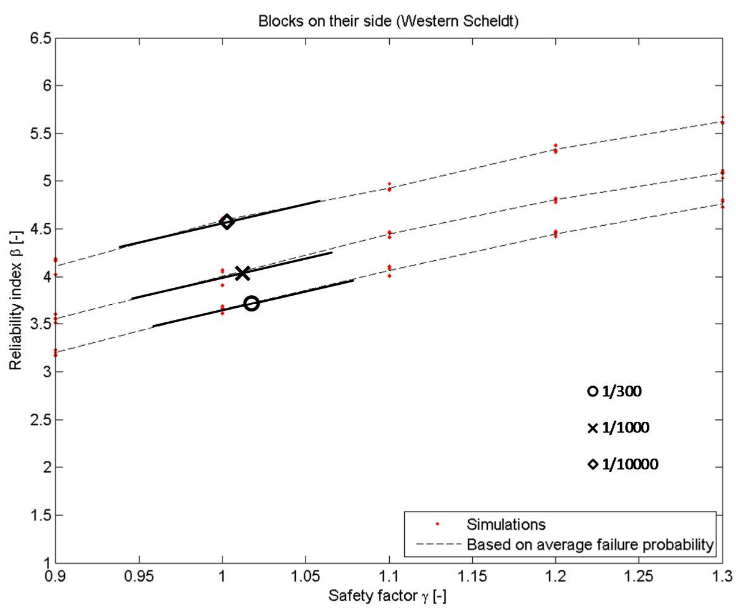

Figure C.7. Reliability indices as a function of the overall safety factor for the Western Scheldt, for hydraulic loads with exceedance probabilities of 1/300, 1/1000 and 1/10000 per year. The cross-sectional target reliabilities corresponding to flood protection standards of 1/300, 1/1000 and 1/10000 per year have been marked. Results shown for an optimistic model factor (μ=1.380, σ=0.288, lower limit=0.87).

The results shown in Figure C.7 point to safety factors in the range of 1-1.02 for the target reliabilities marked in these figures. These safety factors are only marginally smaller than the values presented in chapter 8 (1.03-1.04).

Calibrating a partial safety factor with a conservative model factor

The required block thicknesses and corresponding reliability indices have also been calculated for a range of values of the overall safety factors for a conservative model factor. Figure C.8 shows the calculated reliability indices for the different test set members (red dots), the corresponding average failure probabilities, as well as the resulting β_T-dependent safety factor (or overall safety factor) within the range of β_T of interest.

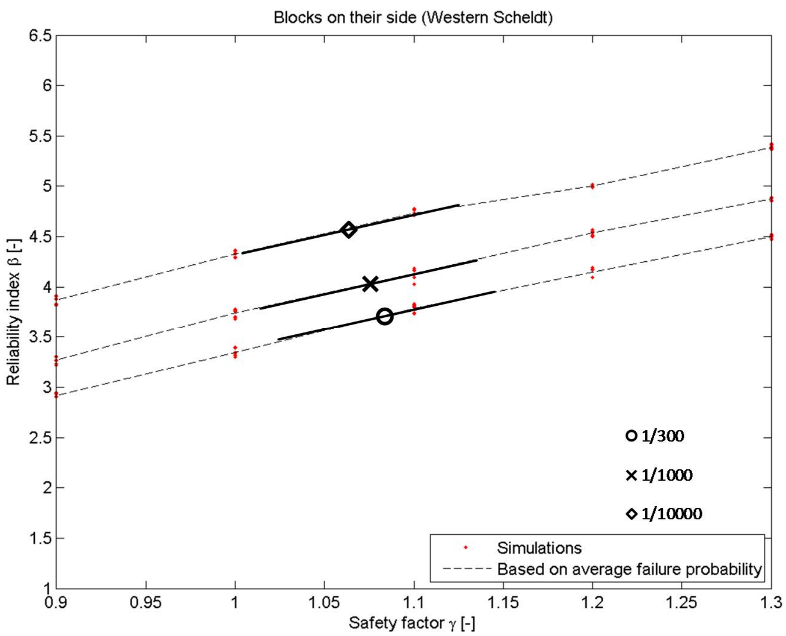

Figure C.8. Reliability indices as a function of the overall safety factor for the Western Scheldt, for hydraulic loads with exceedance probabilities of 1/300, 1/1000 and 1/10000 per year. The cross-sectional target reliabilities corresponding to flood protection standards of 1/300, 1/1000 and 1/10000 per year have been marked. Results shown for an optimistic model factor (μ=1.380, σ=0.288, lower limit=0.87).

The results shown in Figure C.7 and Figure C.8 point to safety factors in the range of 1.05-1.08. These values are close to the values presented in chapter 8 (1.03-1.04). Note also that the consequences of the differences, in terms of the implied reliabilities, are very small.

C.4 Varying the hydraulic loading conditions

C.4.1 Varying the wave steepness

To evaluate the sensitivity of the calibrated semi-assessment rule to the assumed wave steepness, the required block thicknesses were calculated for the following average wave steepnesses (σ=0.002 in all cases):

1. μ=0.04 (base case)
2. μ=0.02 (low)
3. μ=0.06 (high)

The required block thickness was calculated for each test case, for safety factors ranging from 0.9 to 1.3 and a flood protection standard of 1/1000 per year. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM.

No convergence issues were experienced for average wave steepnesses of 0.02 or 0.04. Convergence failed in 21 out of 27 cases for an average wave steepness of 0.06 and a partial safety factor of 1.0. These cases were removed from the dataset, leaving a total of 114 cases. The results from the (converged) calculations are shown in Figure C.9 and Figure C.10.

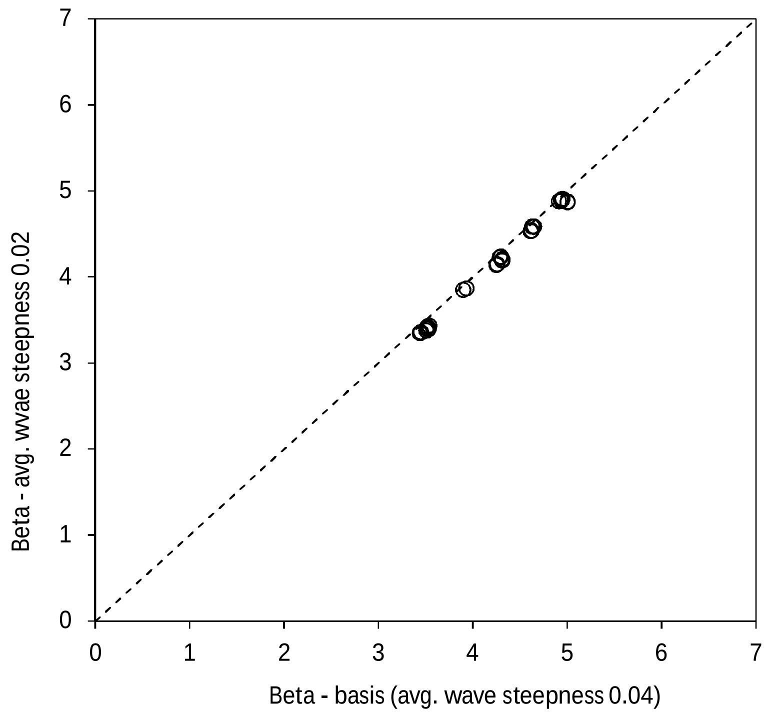

Figure C.9. Comparison of the outcomes of calibration exercises with the original avg. wave steepness of 0.04 (base case, horizontal axis) and an avg. wave steepness of 0.02 (vertical axis).

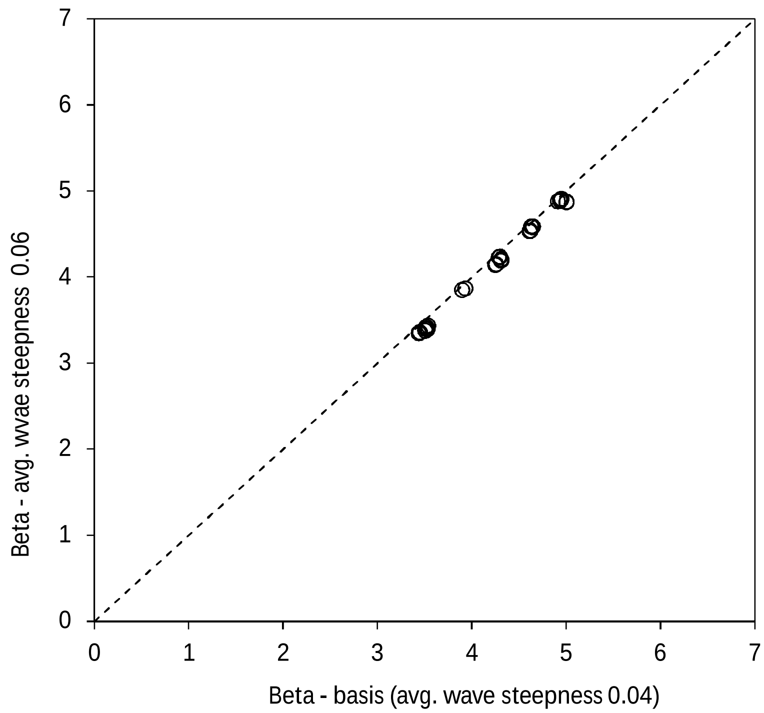

Figure C.10. Comparison of the outcomes of calibration exercises with the original avg. wave steepness of 0.04 (base case, horizontal axis) and an avg. wave steepness of 0.06 (vertical axis).

Figure C.9 and Figure C.10 show that the reliability implied by a particular safety factor is insensitive to average wave steepness that is assumed in the calculations. This is because it influences the outcomes of both the probabilistic and the semi-probabilistic assessment: a less favourable wave steepness leads to a greater probability of failure, but also to a less favourable characteristic load in a semi-probabilistic assessment.

C.4.2 Varying the level of the foreshore

To evaluate the sensitivity of the calibrated semi-assessment rule to the level of the foreshore, the required block thicknesses were calculated for a case in which the level of the foreshore was increased from NAP-5m (base case) to NAP+0m. Such a high foreshore causes significant wave breaking, which reduces the load the revetment.

The required block thickness was calculated for each test case, for safety factors ranging from 0.9 to 1.3 and a flood protection standard of 1/1,000 per year. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. No convergence issues were experienced. Results are shown in Figure C.11.

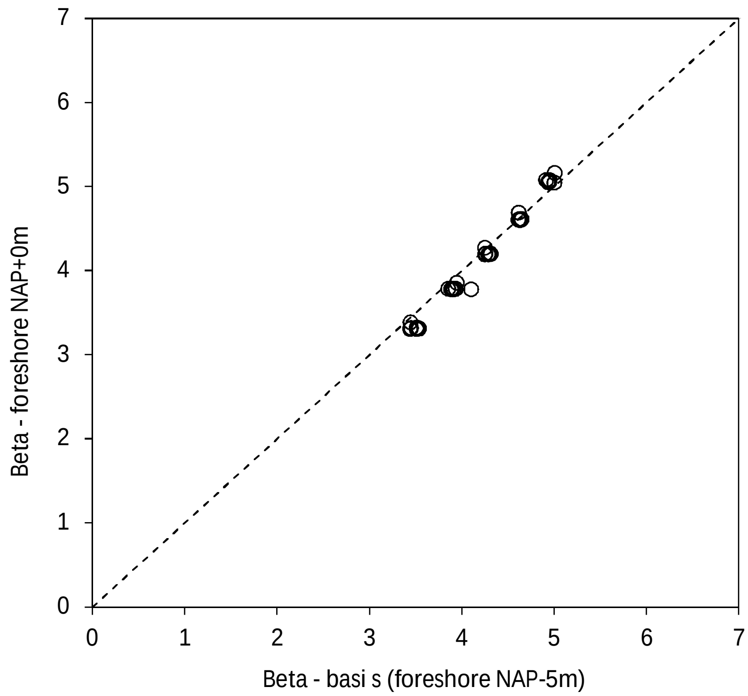

Figure C.11. Comparison of the outcomes of calibration exercises with a foreshore at NAP-5m (base case, horizontal axis) and a foreshore at NAP+0m (vertical axis).

Figure C.11 shows that the reliability implied by a particular safety factor is rather insensitive to the level of the foreshore. This is because it influences the outcomes of both the probabilistic and the semi-probabilistic assessment: a less favourable wave steepness leads to a greater probability of failure, but also to a less favourable characteristic load in a semi-probabilistic assessment.

C.4.3 Varying the duration of the peak water level

When the peak water level lasts longer, more waves will impact the revetment at the peak water level. To obtain insight into the effect this may have on the adequacy of the calibrated partial safety factors, probabilistic and semi-probabilistic assessments were carried out assuming a 4 times longer duration of the peak water level during each storm event (20 hours instead of 5 hours). This was done for all test set members, for safety factors ranging from 0.9 to 1.3.

The required block thicknesses were calculated with both the original and the modified load model. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. Convergence was attained in 264 out of 405 cases. Since the results of the different sensitivity analyses consistently show similar results across a broad range of cases, no effort was made to resolve all convergence issues. The converged results are shown are shown in Figure C.12.

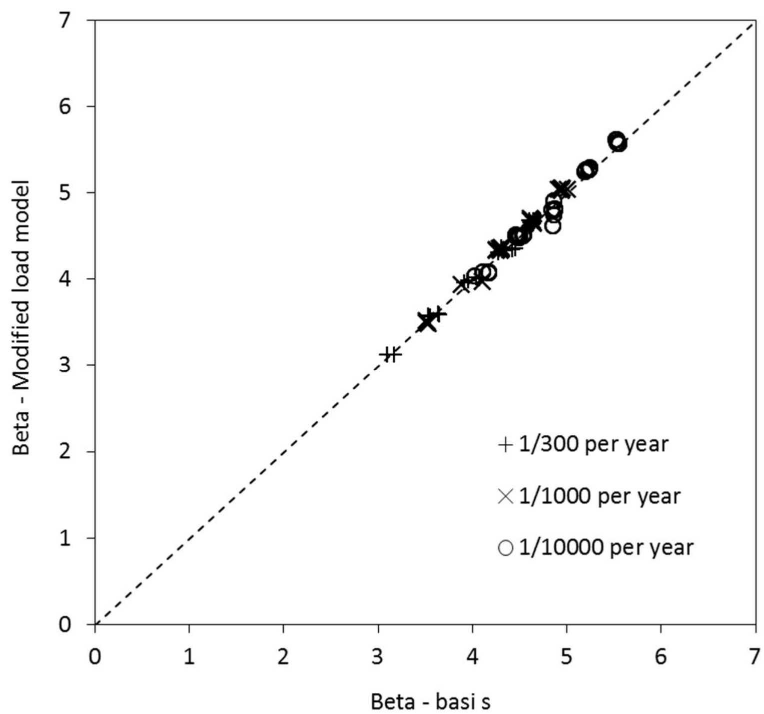

Figure C.12. Comparison of the outcomes of calibration exercises with the original load model (horizontal axis) and the modified load model (peak water level duration of 20 hours; vertical axis).

Figure C.12 shows that the impact of a longer duration of the peak water level on the outcomes of the calibration exercise is negligible. This is because the longer duration of the peak water level influences the outcomes of both the probabilistic and the semi-probabilistic assessment.

C.4.4 Varying the wave height

To obtain insight into the effect of smaller wave heights during storm events on the adequacy of the calibrated partial safety factors, probabilistic and semi-probabilistic assessments were carried out assuming two times smaller significant wave heights during each storm event (before wave breaking). This was done for all test set members, for safety factors ranging from 0.9 to 1.3.

The required block thicknesses were calculated with both the original and the modified load model. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. Convergence was attained in 51 out of 405 cases. Since the results of the different sensitivity analyses consistently show similar results across a broad range of cases, no effort was made to resolve all convergence issues. The converged results are shown in Figure C.13.

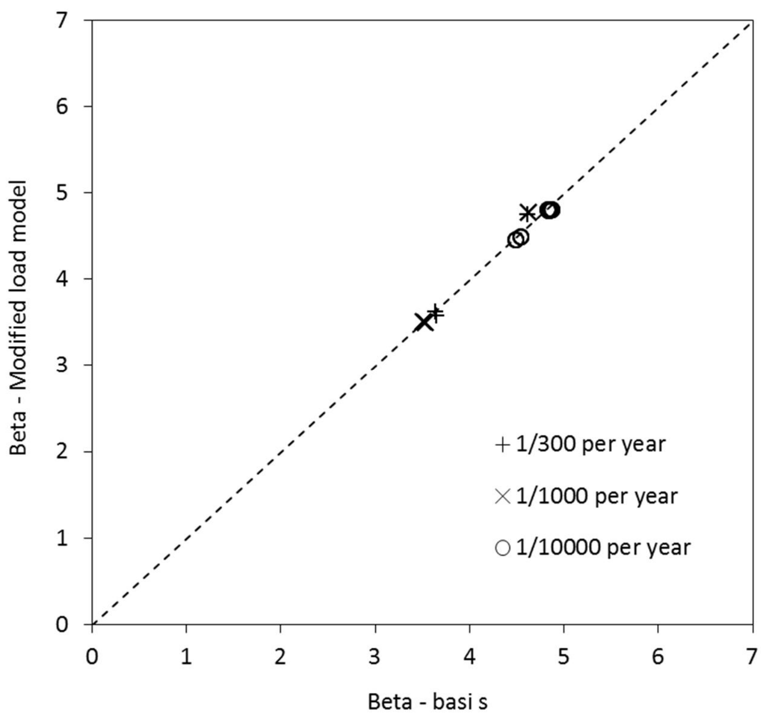

Figure C.13. Comparison of the outcomes of calibration exercises with the original load model and the modified load model (two times smaller wave heights).

Figure C.13 indicates that the impact of smaller wave heights on the outcomes of the calibration exercise is negligible. This is because the wave heights influence the outcomes of both the probabilistic and the semi-probabilistic assessment.

C.4.5 Varying the duration of the peak water level and the wave height

Probabilistic and semi-probabilistic assessments were carried out assuming a four times longer duration of the peak water level (20 hours instead of 4 hours) and two times smaller wave heights during each storm event (before wave breaking). This was done for all test set members, for safety factors ranging from 0.9 to 1.3.

For all cases (cross sections), the required block thicknesses were calculated with both the original and the modified load model. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. Convergence was attained in 69 out of 405 cases. Since the results of the different sensitivity analyses consistently show similar results across a broad range of cases (also for computations with different settings, in which different cases converge), no effort was made to resolve all convergence issues. The converged results are shown in Figure C.14.

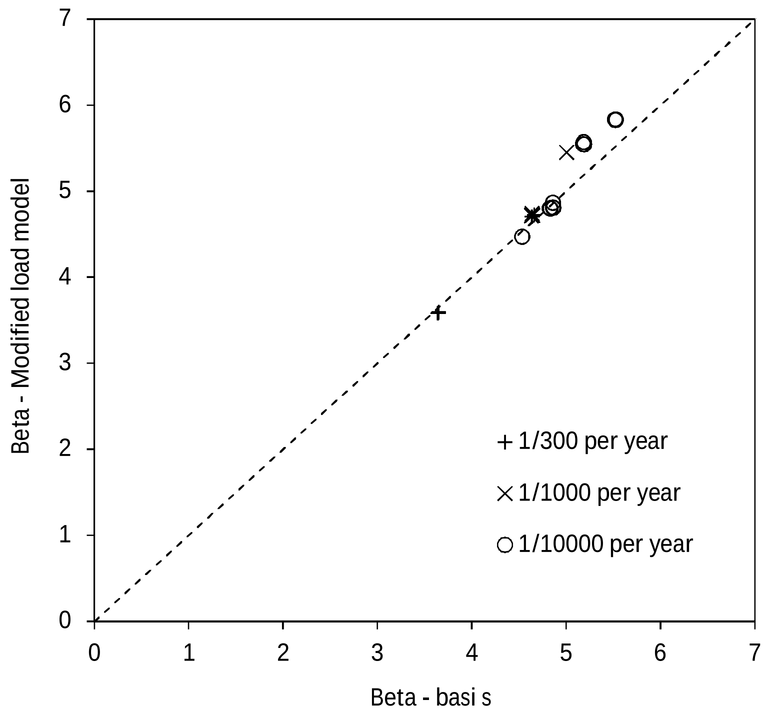

Figure C.14. Comparison of the outcomes of calibration exercises with the original load model (peak water level duration of 4 hours) and the modified load model (peak water level duration of 20 hours and two times smaller significant wave heights).

Figure C.14 indicates that the impact of a longer duration of the peak water level in combination with a smaller significant wave height on the outcomes of the calibration exercise is negligible. This is because these changes influence the outcomes of both the probabilistic and the semi-probabilistic assessment.

C.4.6 Applying the Lake IJssel load model

Blocks on their side cannot be found along Lake IJssel. It was explicitly requested by Rijkswaterstaat however, to also carry out calculations using the simplified load model for Lake IJssel that was used in a calibration exercise for columns, see Jongejan et al. (2015a) for further details. Calculations were made assuming the characteristics of blocks on their side along the Western Scheldt, together with the dike geometries for the cases along Lake IJssel from Jongejan et al. (2015a).

For all cases (cross sections), the required block thicknesses were calculated with both the load model for the Western Scheldt and the one for Lake IJssel, for safety factors ranging from 0.9 to 1.3. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. Convergence was attained in 186 out of 405 cases. The converged results are shown in Figure C.14.

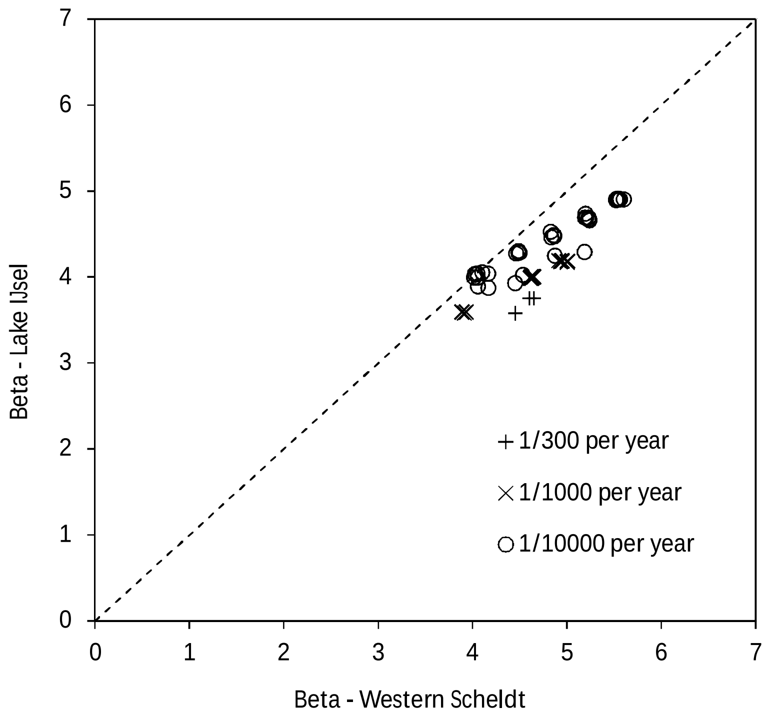

Figure C.15. Comparison of the outcomes of calibration exercises with the Western Scheldt load model and the Lake IJssel load model.

Figure C.15 indicates that the partial safety factor should be somewhat higher for blocks on their side along Lake IJssel. To bridge the difference between the reliability indices shown in Figure C.15, the partial safety factors for blocks on their side would have to be about 10% higher along Lake IJssel. This is similar to what was found for columns along Lake IJssel (see Jongejan et al., 2015a). This can be explained by the fact that the partial safety factor essentially corrects for the effect of the difference between the characteristic value and the design point value of the hydraulic load (model uncertainty apart), Due to the different characteristics of the hydraulic loads along Lake IJssel and the Western Scheldt, a slightly higher safety factor is needed along Lake IJssel. Note also that, according to the probabilistic calculations, the required thickness of a block revetment is quite small along Lake IJssel (about 20 cm) while the required thickness along the Western Scheldt is about 40 cm (the actual thickness of block revetments of blocks on their side is often about 50cm). This also means that the effect of a larger safety factor on the absolute value of the required thickness (i.e., the effect in terms of centimetres) is much smaller along Lake IJssel than along the Western Scheldt. It is stressed that this discussion is purely hypothetical since blocks on their side cannot be found along Lake IJssel.

C.4.7 Applying the Wadden Sea load model

Blocks on their side cannot be found along the Wadden Sea. It was explicitly requested by Rijkswaterstaat however, to also carry out calculations using the simplified load model for the Wadden Sea that was used in a calibration exercise for columns, see Jongejan et al. (2015a) for further details. Calculations were made assuming the characteristics of blocks on their side along the Western Scheldt, together with the dike geometries for cases along the Wadden Sea from Jongejan et al. (2015a).

For all cases (cross sections), the required block thicknesses were calculated with both the original and the modified load model, for safety factors ranging from 0.9 to 1.3. Reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. Convergence was attained in 324 out of 405 cases. The converged results are shown in Figure C.14.

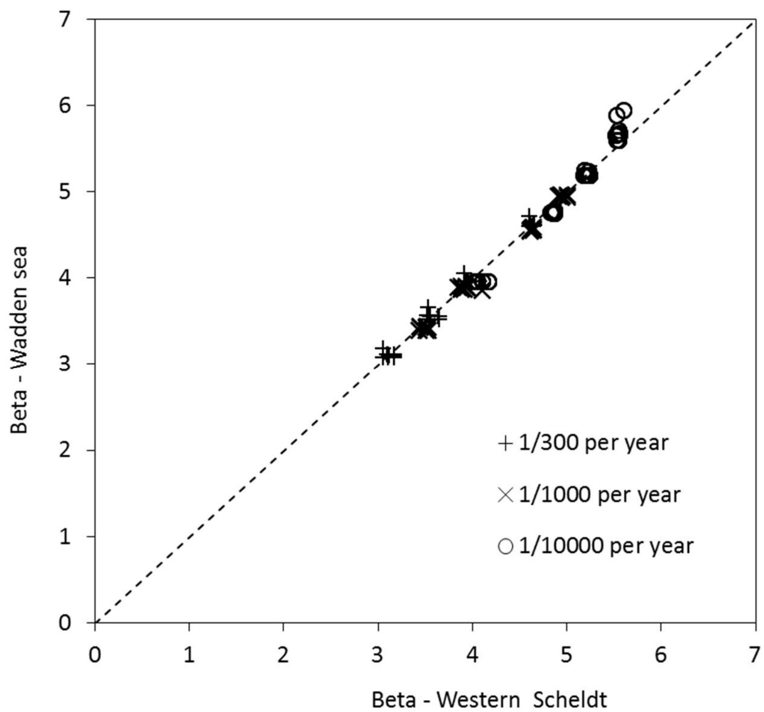

Figure C.16. Comparison of the outcomes of calibration exercises with the Western Scheldt load model and the Wadden Sea load model.

Figure C.16 indicates that the partial safety factor for blocks on their side should be virtually identical for the Wadden Sea and the Western Scheldt. This is similar to what was found for columns (see Jongejan et al., 2015a). Since blocks on their side cannot be found along the Wadden Sea, this result and discussion are purely hypothetical.

D FORM and MC

To evaluate the accuracy of FORM-calculations, results of FORM-calculations have been compared to the results of MC-simulations. Results are shown in Figure D.1 to Figure D.3 for columns and in Figure D.4 for blocks on their side. The close correspondence indicates that FORM yields accurate results: the calculated reliability indices with FORM and MC are very similar: they are often less than 1% apart. The observed differences are irrelevant for the calibration of safety factors.

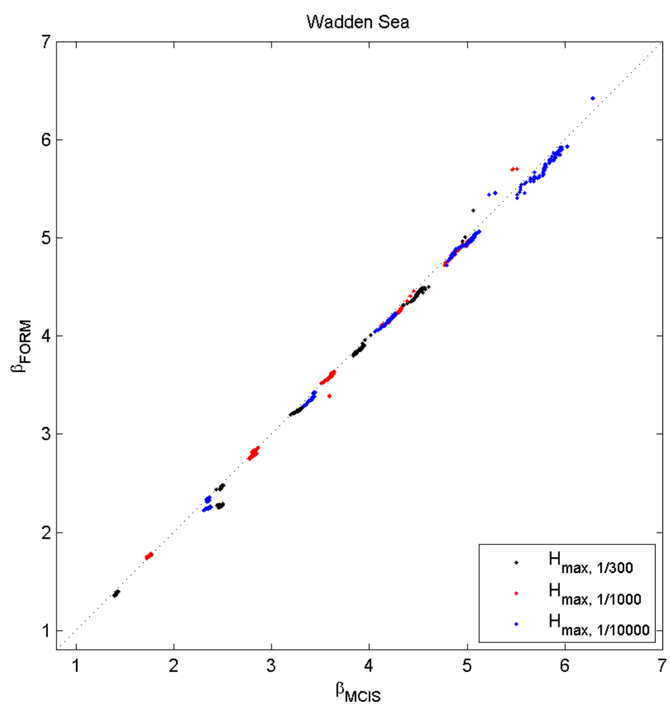

Figure D.1 Columns: the reliability index obtained by a FORM-analysis (vertical axis) versus the reliability index obtained by MC-simulation (horizontal axis) for cases along the Western Scheldt

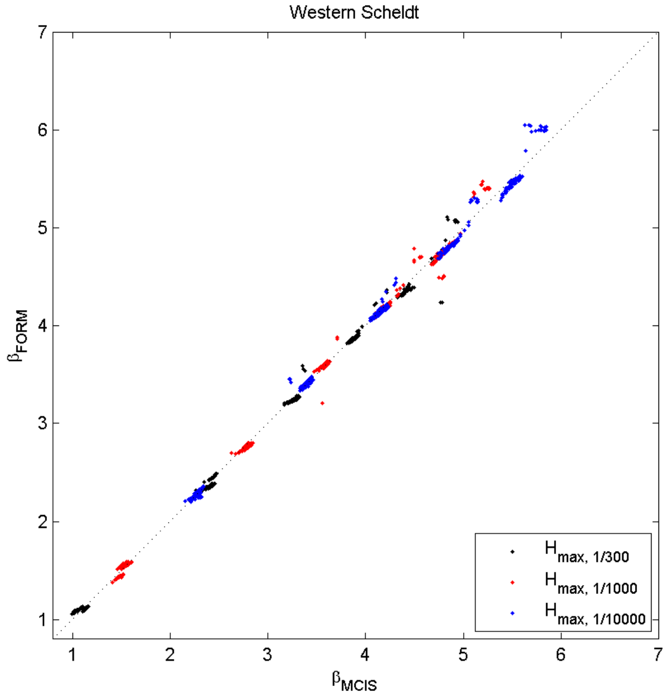

Figure D.2 Columns: the reliability index obtained by a FORM-analysis (vertical axis) versus the reliability index obtained by MC-simulation (horizontal axis) for cases along the Wadden Sea

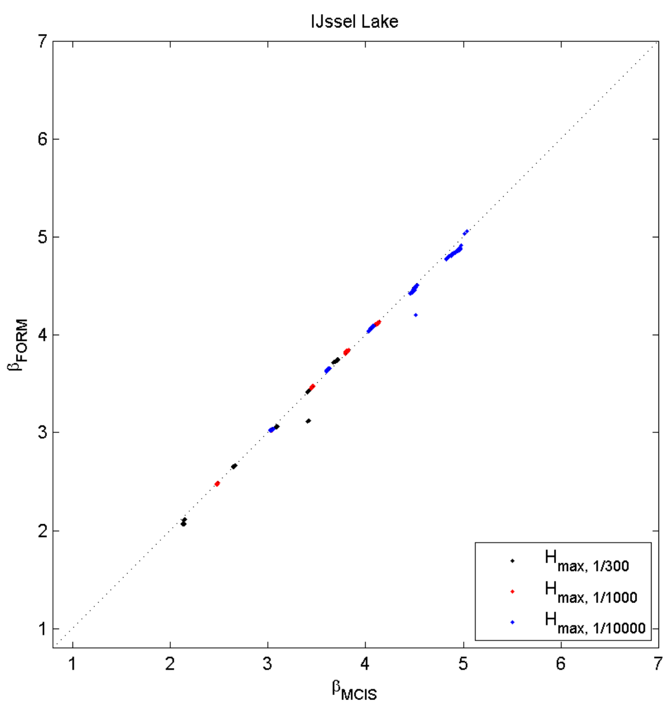

Figure D.3 Columns: the reliability index obtained by a FORM-analysis (vertical axis) versus the reliability index obtained by MC-simulation (horizontal axis) for cases along Lake IJssel

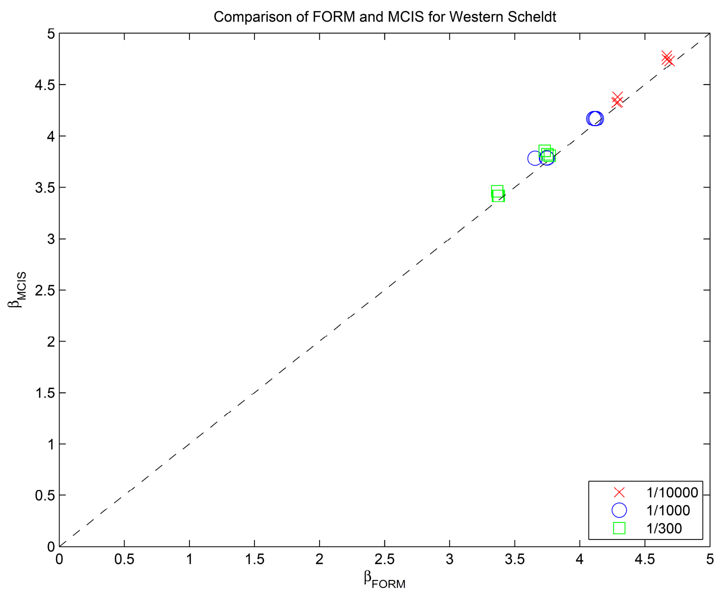

Figure D.4. Blocks on their side: the reliability index obtained from a FORM-analysis (horizontal axis) versus the reliability index obtained from MC-analysis (vertical axis). Results shown for 3 test set members (cross sections) designed in accordance with partial safety factors of 1.0 and 1.1, and a range of safety standards (1/300, 1/1000 and 1/10000 per year).

E Comparing the response surfaces to Steentoets

To test the accuracy of calibration exercise based on response surfaces rather than Steentoets itself, the calibration was carried out with the response surfaces and Steentoets for a number of cases.

E.1 Response surface for columns

For 7 test set members (cross sections) along the Western Scheldt and Lake IJssel, the required block thicknesses were calculated with response surfaces and Steentoets according to the semi-probabilistic method, for safety factors ranging from 0.9 to 1.2. Reliability indices were subsequently calculated for each of these cases. The probabilistic analyses were carried out with FORM. Convergence issues were experienced in a small number of calculations. The results of these calculations were removed from the dataset.

Each dot in Figure E.3 shows, for each case, the calculated reliability index with the response surface if the revetment is “designed” according to the semi-probabilistic method using the response surface (horizontal axis) and the calculated reliability index with Steentoets if the revetment is “designed according to the semi-probabilistic method using Steentoets (vertical axis). Here, the term “design” means that the representative value of the block thickness is chosen such that the revetment would just pass a semi-probabilistic assessment.

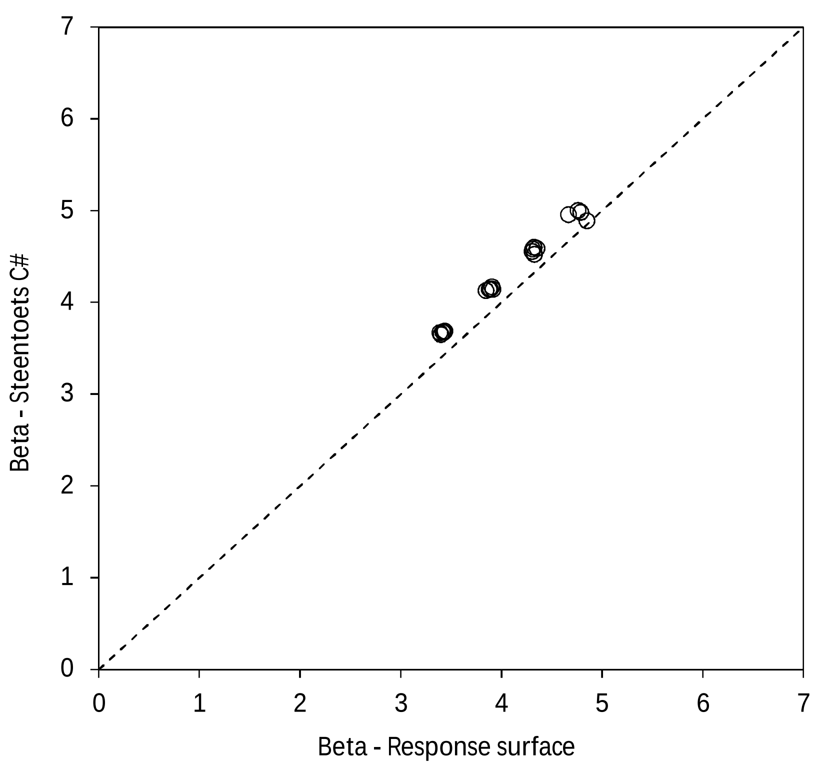

Figure E.1 Comparison of the outcomes of calibration exercises with the response surface and Steentoets. Results shown for cases along the Western Scheldt.

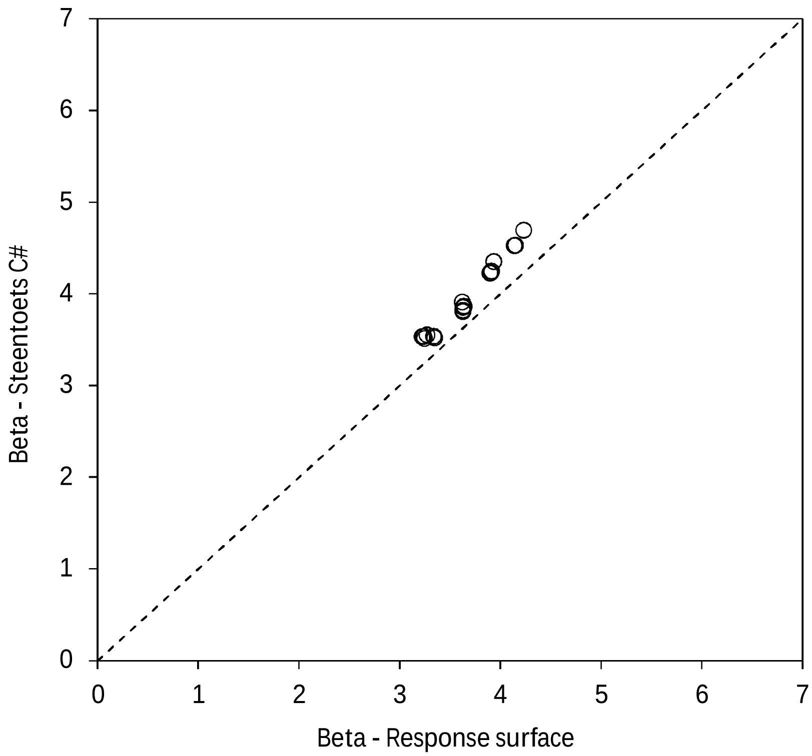

Figure E.2 Comparison of the outcomes of calibration exercises with the response surface and Steentoets. Results shown for cases along Lake IJssel.

The results of series of semi-probabilistic and probabilistic calculations with the response surface for concrete blocks and Steentoets appear to be in relatively close agreement. On average, the calculations with Steentoets give 5% higher reliability indices. This indicates that a calibration exercise for concrete blocks based on a response surface appears to be slightly conservative. The amount of conservatism is quite small, however: a difference of 5% in terms of target reliability indices leads to insignificant differences in terms of safety factors (<1%).

E.2 Response surface for blocks on their side

For 3 test set members (cross sections), the required block thicknesses were calculated with the semi-probabilistic method with both the response surfaces and Steentoets, for safety factors of 1.0 and 1.1, and a range of flood protection standards (1/300, 1/1000 and 1/10000 per year). The reliability indices were subsequently calculated for each case. All probabilistic analyses were carried out with FORM. No convergence issues were experienced.

Each dot in Figure E.3 shows, for each case, the calculated reliability index with the response surface if the revetment is “designed” according to the semi-probabilistic method using the response surface (horizontal axis) and the calculated reliability index with Steentoets if the revetment is “designed according to the semi-probabilistic method using Steentoets (vertical axis). Here, the term “design” means that the representative value of the block thickness is chosen such that the revetment would just pass a semi-probabilistic assessment.

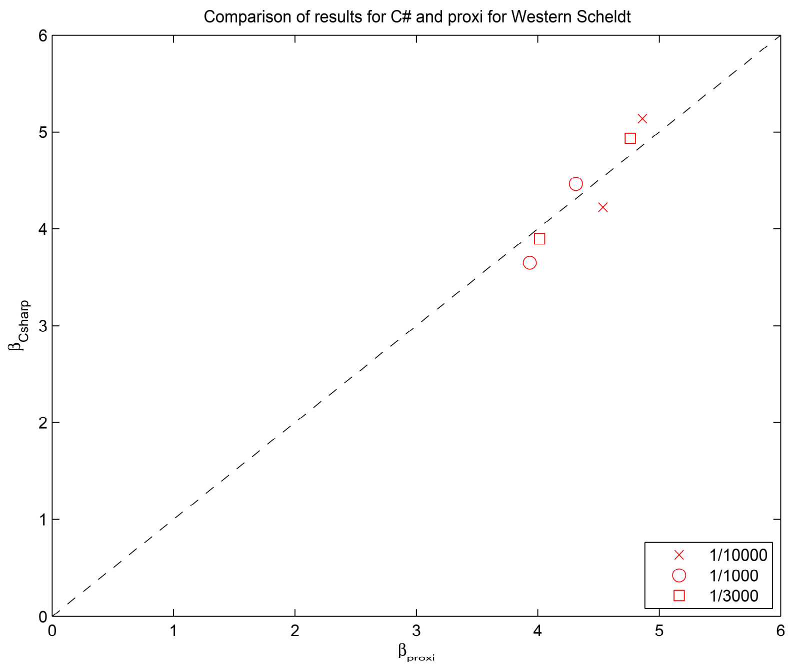

Figure E.3 Comparison of the outcomes of calibration exercises with the response surface ($\beta_{proxi,}$ horizontal axis) and Steentoets ($\beta_{Csharp,}$ vertical axis).

The outcomes of the calculations with Steentoets and the response surface are in good agreement. This indicates that safety factors for blocks on their side may be calibrated with a response surface, without significant loss in accuracy.

F Characterizing the length effect

The one-dimensional, spatial autocorrelation functions in Hydra-Ring will be the same for all stochastic variables:

$$\rho_i(\Delta x) = \rho_{i,\infty} + \left(1 - \rho_{i,\infty}\right)\exp\left(-\frac{\Delta x^2}{d_i^{\,2}}\right) \tag{F1}$$

where

Δx distance between two cross-sections

$\rho_{i,\infty}$ lower limit of the autocorrelation function ('residual correlation') for variable X_i

d_i correlation distance.

The autocorrelation function of the limit state function $Z=f(X_1, X_2,\ldots, X_n)$ is assumed to have a similar shape:

$$\rho_Z(\Delta x) = \rho_{Z,\infty} + \left(1 - \rho_Z\right)\exp\left(-\frac{\Delta x^2}{d_Z^{\,2}}\right) \tag{F2}$$

with

$$\rho_{Z,\infty} = \sum \alpha_i^{\,2} \rho_{i,\infty} \tag{F3}$$

$$\frac{1}{d_Z^{\,2}} = \frac{1}{1-\rho_{Z,\infty}} \sum \frac{\alpha_i^2\left(1-\rho_{i,\infty}\right)}{d_i^2} \tag{F4}$$

For a standard normally distributed limit state function with an autocorrelation function given by equation (F2), the average length of the excursions of a threshold equal to β can be calculated as follows:

$$\Delta l \approx \frac{d_Z\sqrt{\pi}}{\beta\sqrt{\left(1-\rho_{Z,\infty}\right)}} \tag{F5}$$

where

Δl length of independent, equivalent stretches

β reliability index

d_Z correlation distance

Preliminary values of $\rho_{i,\infty}$ and d_i values are given in Table F.1 for blocks on their side. These were based on the spatial variability expected by block revetment specialists.

Table F.1. The autocorrelation function parameters for the stochastic variables (Jongejan et al., 2013). Values of ρ_x d_x for variables that were missing in Jongejan et al. (2013) have been marked by an asterix.

Parameter	Unit	Symbol in software code	ρ_x	d_x (m)
Dike Geometry				
thickness of the clay layer	[m]	dc	0	135
outer slope	[-]	cotau	0	540
height of the berm relative to NAP	[m]	zBerm	0	270
slope of the berm	[-]	cotab	0	540
width of the berm	[m]	Bb	0*	540*
slope of the foreshore	[-]	tanaBodem	0*	540*
height of the dike toe relative to NAP	[m]	zBodem	0	540
Block revetment parameters				
block thickness	[m]	D	0	5
width of the blocks	[m]	B	0	5
length of the blocks	[m]	L	0	5
Joint width of vertical joints	[m]	ssa	0	5
density of the blocks	[kg/m³]	rohS	0	135
thickness of the filter layer	[m]	b1	0	135
Level of the top of the revetment relative to NAP	[m]	zB	0	270
level of the low border over NAP (transition structure)	[m]	zO	0	270
porosity of the filter material	[-]	nf1	0	5400
grain size of the filter material	[m]	Df151	0	135
Model factors				
model factor for the calculation of the failure of the block revetment for concrete columns	[-]	mBR	1	-
Hydraulic load				
Water level	[-]	H	1*	-
Significant wave height	[-]	Hs	1*	-

The influence coefficients of the model factor and the hydraulic loading parameters are dominant across virtually every test set member. This implies that the calculated length-effect is relatively small along a statistically homogeneous revetment. Several statistics of the calculated values of Δl for blocks on their side are shown in Table F.2.

Table F.2. The computed lengths of equivalent, independent lengths (Δl) in meter for test set members that comply with an overall safety factor of 1.0 and 1.1 and a particular flood protection standard. Results apply to blocks on their side..

Flood protection standard	Maximum	Minimum	Average	Median
1/300	1490	98	348	202
1/1000	1247	55	545	491
1/10000	1247	55	544	491

It should be noted that the computed values of Δl are extremely sensitive to the influence coefficients of the stochastic variables for which d_x=5m. This also makes the computed values of Δl very sensitive to numerical errors/approximations in the calculations of the influence coefficients. The average and median values of Δl shown in Table F.2 are in the order of several hundred meters. Based on this result, which indicates a weak length effect, it was assumed that the probability of failure of a revetment is two times greater than the cross sectional failure probability.

For columns, the small value of d_x for the block thickness (5m), in combination with a squared influence coefficient of the block thickness of about 4.10^{-4} led to values of Δl of less than 100m. It seems questionable however whether the uncertainty related to the block thickness can, in reality, give rise to such a strong length effect. The block thickness is a normally distributed variable with a standard deviation of 0.0013 m. The probability of exceeding a 1 cm difference from the average block thickness in a 2 km stretch is smaller than $1-(1-\Phi(-0.01/0.0013))^{2000/5} = 2.9\text{x}10^{-12}$. It seems unlikely that variations within such a narrow range can give rise to a sizeable length effect. It may well be that the calculated value of Δl is the result of (small) inaccuracies in FORM calculations: for β=4.8, the difference between $\alpha_D{}^2=4.10^{-4}$ and $\alpha_D{}^2<<10^{-6}$ corresponds to a difference in design point values of block thicknesses of a mere 0.00012 m but to a difference in Δl of over 1500m.

G Influence coefficients for wave overtopping

To obtain insight into the relative importance of the uncertainties related to the hydraulic loads, wave overtopping calculations were carried for a number of locations with PC-Ring v.5.3.4. The reason for using PC-Ring rather the Hydra-models is that PC-Ring allows for analyses with a stochastic critical overtopping discharge. Probabilistic analyses with a deterministic critical overtopping discharge (e.g. 1 l/s/m) would lead to the overestimation of the relative importance of the uncertainties related to the loading conditions. Results are shown in Table G.1.

Table G.1 Influence coefficients of loading conditions for different levee systems (from VNK2-databases, based on calculations with the CIRIA-model; the use of the CIRIA-model lead to relatively uncertain critical overtopping discharges)

Water system	**Levee system**	**Average value of α_S**	**Average value of α_S^2**	**α_S associated with average value of α_S^2**
Western Scheldt	No. 32 (Zeeuws Vlaanderen)	0.965	0.934	0.966
Wadden Sea	No, 5 (Texel)	0.953	0.910	0.954
	No. 6 (Friesland, along the Wadden Sea and the Eems estuary)	0.936	0.881	0.939
Lake IJssel	No. 6 (Friesland, along Lake IJssel)	0.955	0.915	0.956
	No. 7 (North East Polder)	0.972	0.946	0.973

Based on the results shown in Table G.1, an α_S-value of 0.95 was assumed in this study. Higher values of α_S would lead to stronger correlations with overtopping and, hence, to less stringent reliability requirements for the stability of block revetments under wave attack.

The abovementioned αS-values are based on the influence coefficients of various stochastic load variables, such as the wind speed, storm duration, and water level. But in the simplified load models that were used in the calibration exercise (see section A.3), there is only one basic stochastic load variable (see section A.3). All other load parameters are linked to this stochastic variable via deterministic relationships. The way correlation coefficients are computed in section 8.2 rests on the assumption that the α_S-values in Table G.1 can be given a similar interpretation as the α-value associated with the basic stochastic load variable in the simplified load models

H The calibrated semi-probabilistic assessment rule in short

This Appendix summarizes the semi-probabilistic assessment rule for the stability of block revetments under wave attack for use in levee safety assessments (not design).

Models
Semi-probabilistic assessments for the stability of block revetments under wave attack are to be carried out with Steentoets and the so-called Q-variant in Hydra-Ring, both accessible via Ringtoets (user interface).

Representative values
1 The representative value of the block density is the 5^{th} quantile value.
2 The representative values of all other stochastic variables related to the block revetment are average values.
3 The hydraulic loads should be obtained with the Q-variant in Hydra-Ring for an exceedance probability equal to the maximum allowable probability of flooding.

Safety factors
The required block thickness should be multiplied with the partial safety factor from Table H.1.

Table H.1 Partial safety factors.

Residual strength class ("reststerkteklasse")	Criteria	Partial safety factor
Small	Hs > 2.0 m, or d_c/H_s < 0.6 and B_{dike}/H_s < 20	1,1
Large	H_s < 1.5 m and d_c/H_s > 0.8, or H_s < 1.5 m and B_{dike}/H_s > 30	0.9
Medium	Other	1.0

Where:
H_s Significant wave height at the toe of the structure as used for the simple safety assessment (m)
d_c Thickness of the clay layer (m)
B_{dike} Width of the dike at the assessment water level as used for the simple safety assessment (m)