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Calibration of Safety Factors for wave impact on Hydraulic Asphalt Concrete Revetments

WTI Cluster C

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Summary
The fatigue resistance of asphalt revetments on dikes is assessed with the Golfklap model (named WaveImpact in WTI2017 software). For WTI2017, the Golfklap model has been implemented in a probabilistic safety assessment model. The objective of this report is to derive semi-probabilistic safety factors using the probabilistic model.

Based on assessment data from previous assessment rounds a realistic range for different input parameters was determined, resulting in a representative test set for the Netherlands. Based on this test set, safety factors were determined using FORM calculations.

The result of the calibration is a set of safety factors with which it is possible to assess asphalt revetments for failures due to wave impact. The safety factors differ for different areas in the Netherlands (based on the boundary conditions) and for different asphalt qualities, based on the coefficient of variation of the cracking strength.

The safety factors found from the calibration have been applied to a test case in order to assess their applicability. The results from this test case lead to the conclusion that the derived safety factors are sufficiently safe and enable level 2a assessments of asphalt revetment for wave impact failures.

References
See Chapter 12

Version	Date	Author	Initials	Review	Initials	Approval	Initials
2	Sep. 2014	Wouter Jan Klerk		Robert 't Hart		Annemargreet de Leeuw	
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Samenvatting

De vermoeiing van asfaltbekledingen op dijken door golfaanval (faalmechanisme AGK) wordt doorgaans beoordeeld met het Golfklap model (in WTI2017 WaveImpact genoemd), waarbij de vermoeiing van een asfaltbekleding door een reeks golfinslagen in een storm wordt berekend. Voor WTI2017 is het Golfklap model in een probabilistisch model geïmplementeerd. In dit rapport zijn semi-probabilistische veiligheidsfactoren afgeleid met behulp van dit model, om niveau 2a toetsingen voor dit mechanisme mogelijk te maken.

Gebaseerd op toetsgegevens van voorgaande toetsrondes is een realistisch bereik van de verschillende invoerparameters bepaald. Dit resulteert in een set cases die representatief kan worden geacht voor het areaal aan bekledingen in Nederland. Gebaseerd op deze verzameling cases zijn met behulp van probabilistische berekeningen relaties tussen faalkansen en veiligheidsfactoren bepaald. Dit is gedaan door voor verschillende cases met ontwerpen te maken met verschillende veiligheidsfactoren, zodanig dat ze precies aan de toetsregel voldoen. Vervolgens zijn hierbij faalkansen uitgerekend met een probabilistische berekening. Dit resulteert in een relatie tussen faalkanseis en benodigde veiligheidsfactor.

De afgeleide veiligheidsfactoren maken het mogelijk asfaltbekledingen op semi-probabilistische basis te toetsen op falen door golfaanval. Dat betekent dat de toetsing qua *vorm* gelijk is aan voorgaande toetsronden, terwijl tegelijkertijd de nieuwe veilgheidsfilosofie en nieuwe normen zijn verwerkt in de veiligheidsfactoren. De veiligheidsfactoren verschillen per deelgebied in Nederland (afhankelijk van de randvoorwaarden) en voor verschillende asfaltkwaliteitsklassen. Deze kwaliteitsklassen worden bepaald aan de hand van de variatiecoëfficiënt van de breuksterkte.

De veiligheidsfactoren gevonden in de kalibratie zijn toegepast op een test casus met verschillende dijkvakken om de toepasbaarheid te beoordelen. De resultaten van deze casus laten zien dat de afgeleide veiligheidsfactoren voldoende veilig zijn en het mogelijk maken niveau 2a toetsingen van asfaltbekledingen voor falen door golfklap uit te voeren.

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List of Symbols

Symbol	Describtion	Unit
α	Influence coefficient of the random variable	[-]
α_a	Curved fatigue parameter for fatigue line	[-]
β_a	Curved fatigue parameter for fatigue line	[-]
β_{CS}	Cross sectional reliability index	[-]
β_{norm}	Reliability index of safety standard	[-]
β_T	Target reliability index	[-]
$\beta_{T,cs}$	Target cross sectional reliability index	[-]
ΔL	Length of equivalent independent reaches	[m]
Δx	Distance between two cross sections	[m]
γ_m	Partial safety factor for model uncertainty m	[-]
γ_s	Beta-dependent safety factor	[-]
γ_s^*	Beta-dependent safety factor (alternative safety format)	[-]
$\lambda_{1,2,3}$	Relative contribution of failure mechanism in failure tree	[-]
ν	Poisson's ratio	[-]
ρ	Auto-correlation	[-]
ρ_k	Auto-correlation of random variable X_k	[-]
ρ_Z	Auto-correlation of the limit state function Z	[-]
σ_b	Cracking strength of asphalt	[MPa]
a	Part of segment where failure mechanism can occur	[-]
b	Representative scale for the length effect	[m]
c	Soil modulus	[MPa/m]
c_a, c_b	Calibration parameters in β-γ relation	[-]
c_{norm}	fitting parameter that depends on the safety standard	[-]
d, d1	Layer thickness	[m]
d_k	Correlation length of random variable X_k	[m]
d_Z	Correlation length of limit state function Z	[m]
E, E1	Young's modulus of asphalt layer	[MPa]
f	Failure probability budget	[-]
H_s	Significant wave height	[m]
L_{avg}	Average length of dike sections in a segment	[m]
L_i	Length of dike section i	[m]
$L_{segment}$	Length of the dike segment	[m]
m	Model uncertainty	[-]
Miner	Miner sum	[-]
N	Number of samples tested	[-]
N_{avg}	Average number of sections in a segment	[-]
N_{mech}	Number of sections where mechanism can occur	[-]
P_T	Target failure probability of a segment for the considered failure mechanism	[-]
$P_{T,CS}$	Target failure probability of a cross section for the considered failure mechanism	[-]
P_{norm}	Safety standard	[-]
P_{CS}	Averaged failure probability for the considered failure mechanism for cross sections [-]	[-]
$P_{seg,AGK}$	Failure probability of dike segment for wave impact	[-]

$P_{segment}$	Failure probability of dike segment	[-]
R_d	Design resistance	[-]
S_d	Design load	[-]
s	Revetment slope	[-]
T_m	Average wave period	[s]
T_p	Peak wave period	[s]
Z	Limit state function	[-]

1 Introduction

1.1 Rationale and goal

The Dutch primary flood defences are periodically tested against statutory safety standards. These standards were previously defined in terms of design loads. Recently, policymakers have contemplated a move towards safety standards defined in terms of maximum allowable probabilities of flooding. To facilitate such a move, a new framework for assessing the safety of flood defences is being developed within the context of the WTI2017-project.

The new safety assessment consists of 3 levels: level 1 being a simple assessment, level 2 a detailed assessment and level 3 an advanced assessment. The framework will allow for probabilistic (level 2b) as well as semi-probabilistic assessments (level 2a). For both levels, a probabilistic model for asphalt revetments is needed, this was developed in the report by Kanning & Den Hengst (2013). In order to facilitate the semi-probabilistic assessments it is necessary to calibrate the safety factors for the mechanism of asphalt revetment failure due to wave impact in such a way that they fulfill the target reliabilities.

1.2 Scope of this report

This report deals with the calibration of safety factors for asphalt revetments on dikes, specifically for the failure mechanism ‘Failure of top layer revetment due to wave impact’. Golfklap is the model used for assessments of asphalt revetments. In 2013 the Golfklap v1.3 model was used (Kanning & Den Hengst, 2013). Within WTI, Golfklap 1.3 is implemented in a new software kernel, therefore the first part of this report deals with the new Golfklap kernel (named WaveImpact), and its implementation in the probabilistic framework that was created in 2013.

After the successful implementation of WaveImpact in the probabilistic framework, safety factors have to be derived in order to enable the semi-probabilistic assessment (level 2a) of asphalt revetments for the considered failure mechanism.

This report discusses the implementation of WaveImpact in the probabilistic framework created by Kanning & Den Hengst (2013) as well as the application of WaveImpact for deriving safety factors for failures of asphalt revetments due to wave impact. The calibration described in this report only applies to ‘Hydraulic Asphalt Concrete’ (‘Waterbouwasfaltbeton’ in Dutch), which covers 80% of all asphalt revetments in the Netherlands. For the complete assessment procedure, please refer to Wichman & ’t Hart (2013). When referred to asphalt in this report, in fact is being referred to Hydraulic Asphalt Concrete.

1.3 Structure of the report

The report follows a similar general structure as the calibration for block revetments; in order to obtain consistency between the different types of revetments, see Jongejan (2014).

This report first deals with the failures of asphalt revetments and the used failure model (Chapter 2). Subsequently, the calibration of safety factors is discussed in Chapter 3 and its application to asphalt revetments is presented in Chapters 4 to 8. Finally a test case is presented for which the safety factors have been applied to dike ring 5 Texel in Chapter 9.

2 The WaveImpact model for asphalt failure due to wave impact

This chapter describes the deterministic modelling and semi-probabilistic safety assessment of wave impact resistance of asphalt revetments with the WaveImpact or Golfklap model. Section 2.1 deals with the use of WaveImpact for assessing revetments, as well as the implementation of this model in a Matlab environment. Section 2.2 gives a description of the physics in the model and Section 2.3 gives a short overview of the assessment procedure for failure of asphalt revetments due to wave impact.

2.1 Detailed assessment asphalt revetment with WaveImpact.

2.1.1 Asphalt fatigue modelling: Miner's rule

Similar to other failure mechanisms, asphalt revetments are assessed on three levels. The first level is a simple assessment in the form of graphs that show the relation between local wave height and required asphalt thickness. The second, detailed, level is mainly a fatigue test of the revetment. The third level is an advanced assessment which may entail more measurements or more advanced analysis methods. This report focuses on the second, detailed, level of assessment. For the whole assessment procedure of asphalt revetments, please refer to the assessment manual (VTV, 2007).

Asphalt revetments are usually constructed directly on a sand dike body. One of the main failure mechanisms is failure of the asphalt layer due to wave impact. With many wave impacts in a storm, this is mainly a fatigue problem. By summing the effect over all wave loads during a storm, the so-called Miner sum, which is a measure of the damage to the asphalt, can be calculated. This Miner sum is calculated using the WaveImpact model. Theoretically, the revetment fails when the Miner sum is larger than 1. However, due to supposed implicit safety (e.g. residual strength, correlation between parameters), the allowable Miner sum might be larger if an advanced assessment is carried out (Wichman & 't Hart, 2013). Within the old safety assessment (VTV, 2006), this has the following impact:

- A Miner sum smaller than one: Assessment 'Good'.
- A Miner sum between 1 and 5: Assessment 'Questionable', an advanced assessment is needed.
- A Miner sum larger than 5: Assessment: 'Insufficient, the revetment needs to be adjusted.

Advanced assessment may consist of more (advanced) measurements, probabilistic analysis or advanced analysis of deterioration. The maximum Miner sum of 5 was increased to 10 for WTI2011 (Wichman & 't Hart, 2013).

2.1.2 The Golfklap model implemented in Matlab

In Kanning & Den Hengst (2013) the Golfklap 1.3 model was used to calculate the Miner sum for asphalt revetments. This model is publicly available through Helpdesk Water. For this report a new version of the Golfklap model, WaveImpact, which has been developed for WTI2017 is used, specifically version 14.1.1.900.

For the study using Golfklap 1.3, a custom-built Matlab interface has been created to enable use of the Golfklap model for both probabilistic and semi-probabilistic calculations. This was done using the OpenEarth library which is (mainly) developed by Deltares. For the new WaveImpact kernel, the same Matlab interface is used. For the WaveImpact kernel this Matlab implementation is extensively tested and gives the same results (Miner sum, intermediate output) as for the standalone benchmark tests. The Matlab implementation of WaveImpact is used throughout this report. More details on the implementation and functioning of the WaveImpact model in the Matlab environment can be found in Appendix B.

The general characteristics of the WaveImpact model are discussed in the subsequent sections. A detailed description is provided in Appendix B of the report by Kanning & Den Hengst (2013).

2.2 Failure mechanism model Golfklap

The failure mechanism “Failure of top layer revetment due to wave impact” of asphalt revetments is modelled in WaveImpact. Asphalt revetments are usually constructed directly on a sand dike body. In brief, the occurring stresses at the asphalt layer (point A in Figure 2.1) are compared to the resistance against fatigue of asphalt by calculating the so-called Miner sum. This is shown in Figure 2.1 for a fixed water level (h) and wave height (H_s). A hydraulic load model is chosen to generate a water level and wave height at each time step during a storm event. The WaveImpact model divides the asphalt revetment in discrete elements. For each time step, the occurring stresses and the fatigue resistance are determined. Finally, these are combined into a Miner sum for each discrete asphalt element for the storm event. The steps that are taken in WaveImpact are briefly summarized below, more information can be found in Appendix A:

- Determination of the hydraulic load model.
- Determination of occurring stresses in the asphalt layer.
- Determination of resisting fatigue stresses.
- Determination of the Miner sum per discrete element and the maximum of the Miner sum for the section considered.

For more background information about the use of Miner’s rule, please refer to KOAC NPC (2009a).

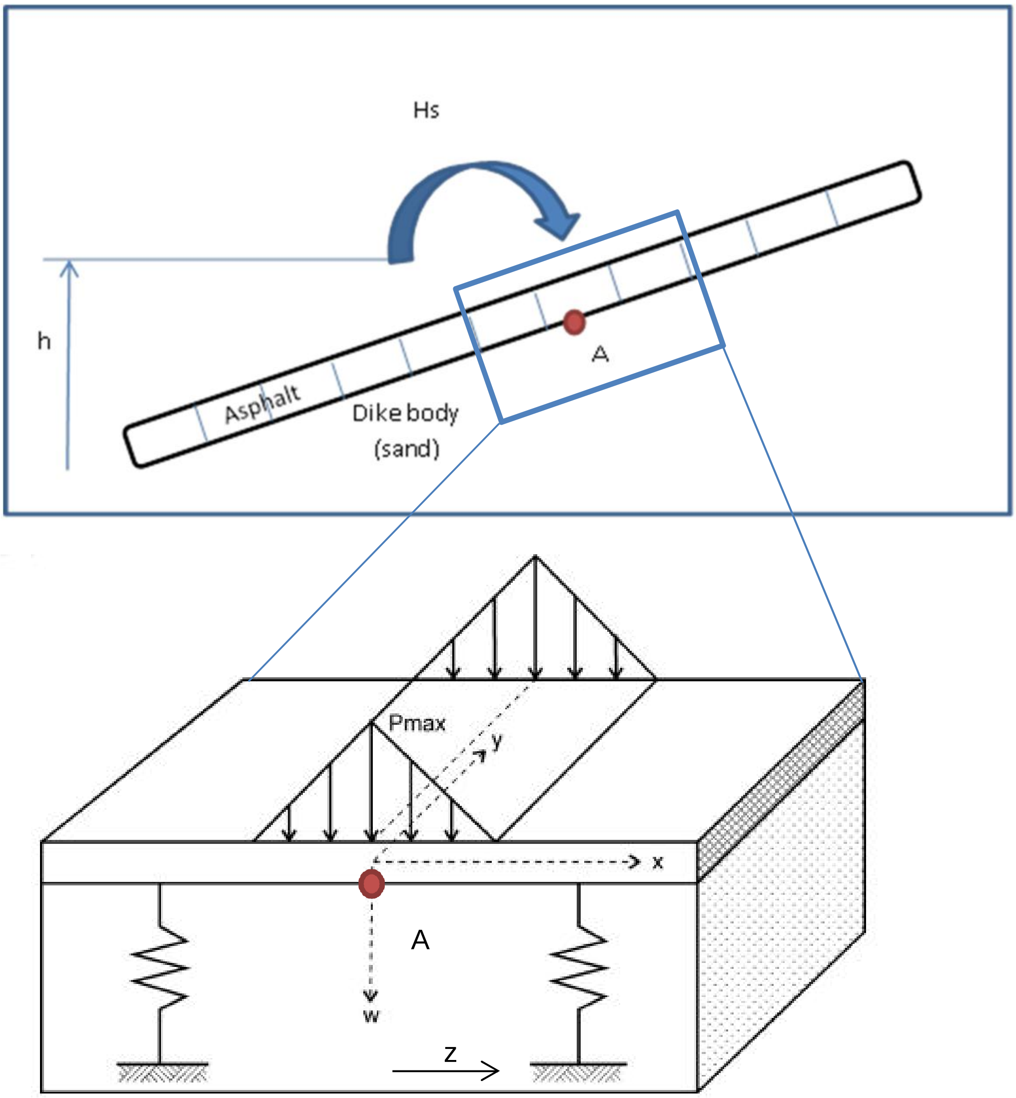


Figure 2.1 Schematic model of an asphalt layer under wave attack (above) and schematic model of the Golfklap model (below, source: KOAC NPC (2009a))

2.3 Current assessment practice

The general steps in a semi-probabilistic assessment are summarized below. For a more detailed description, please refer to KOAC NPC, (2009b):

- Determine the hydraulic regime (Westerschelde, Oosterschelde, North Sea, Wadden Sea or free input).
- Determine the design water level h (Toetspeil), significant wave height H_s and wave period T_m.
- Determine the dike geometry.

- Determine the representative values of the asphalt thickness, asphalt stiffness, soil modulus, and asphalt fatigue parameters based on in-situ and laboratory testing. These are all 5% lower limit values, except for the asphalt elasticity which is a 5% upper limit value. For the fatigue parameters α_a and β_a mean values are taken. For σ_b, the cracking strength, a 5% lower limit is used. The parameters α_a, β_a and σ_b are determined using a spreadsheet called 'grafiekenmaker'. This spreadsheet is outside the scope of this report (KOAC NPC, 2008).
- Calculate the Miner sum for the discretized parts of the asphalt revetment; the maximum Miner sum in the construction determines the total Miner sum
- If the Miner sum is larger than 1, the construction theoretically fails.
- The current assessment rules state (VTV, 2007), that because of implicit safety and residual strength, Miner sums between 1 and 5 should be passed to the advanced assessment (in WTI2011 this range was extended to Miner sums between 1 and 10).

There are also other failure mechanisms which are considered in the assessments. The fault tree in Figure 2.2 gives an overview of this. More details can be found in the assessment handbook for the assessment round of 2006-2011 (VTV, 2007).

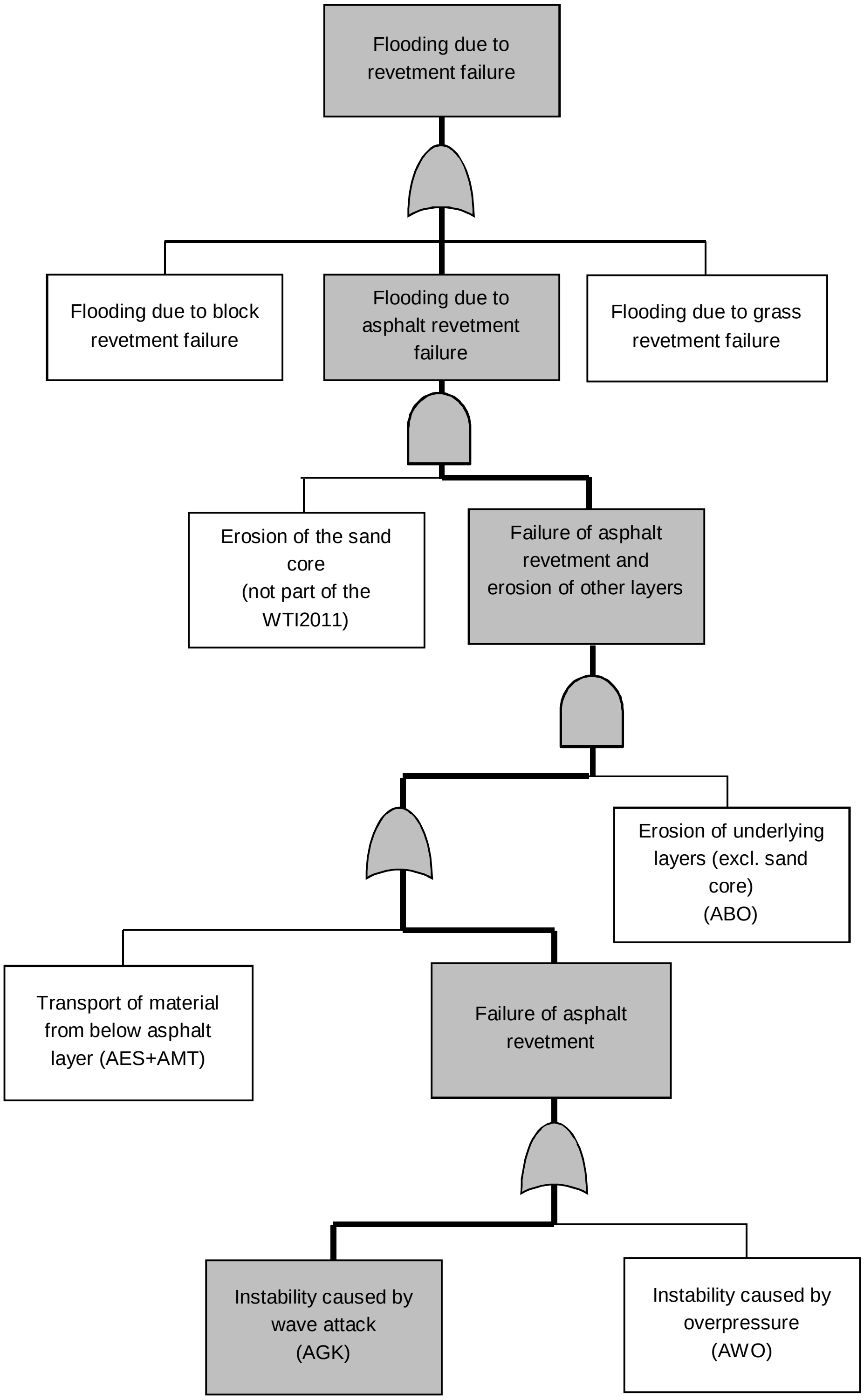


Figure 2.2 Fault tree for failures of asphalt revetments

3 Safety factor calibration procedure

A schematic overview of the procedure for calibrating a semi-probabilistic safety assessment rule for asphalt revetments under wave attack is given by Figure 3.1. It comprises the following steps (Jongejan, 2013):

1 Establish a reliability requirement. This requirement is defined as a maximum allowable probability of failure for the failure mechanism under consideration, i.e. a limit to the contribution of the mechanism to the probability of flooding. It is based on the maximum allowable probability of flooding. This step is dealt with in Chapter 4.

2 Establish the safety format. The safety format concerns how uncertainties are dealt with in order to obtain a sufficiently safe design in a semi-probabilistic computation. This step comprises the following activities:
  - 2.1 Establish a test set that covers a wide range of cases. The test set members may concern existing or fictitious cross-sections of levees.
  - 2.2 Calculate influence coefficients for each parameter for each test set member.
  - 2.3 Based on the outcomes of the previous activity and practical considerations, define representative values for all random variables and decide on the safety factors that are to be included in the semi-probabilistic assessment rule. This is done based on calculated influence coefficients (safety factors on most important variables) and practical considerations (not too many safety factors).

The first two substeps are dealt with in Chapter 5. Step 2.3 is dealt with in Chapter 1.

3 Establish safety factors. This step comprises the following activities:
  - 3.1 Establish, on the basis of representative influence coefficients and a target reliability index, the values of all safety factors, except for one β_T-dependent safety factor. These safety 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 layer thickness so that design resistance R_d equals design load 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.
  - 3.3 Calculate the probability of failure of each (modified) test set member.
  - 3.4 Apply calibration criteria 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.

These steps are dealt with in Chapter 7.

4 Compare the results of the calibrated semi-probabilistic assessment rule those of the present-day rule to give an indication of their impact on assessment scores. This comparison is given in Chapters 8 & 9.

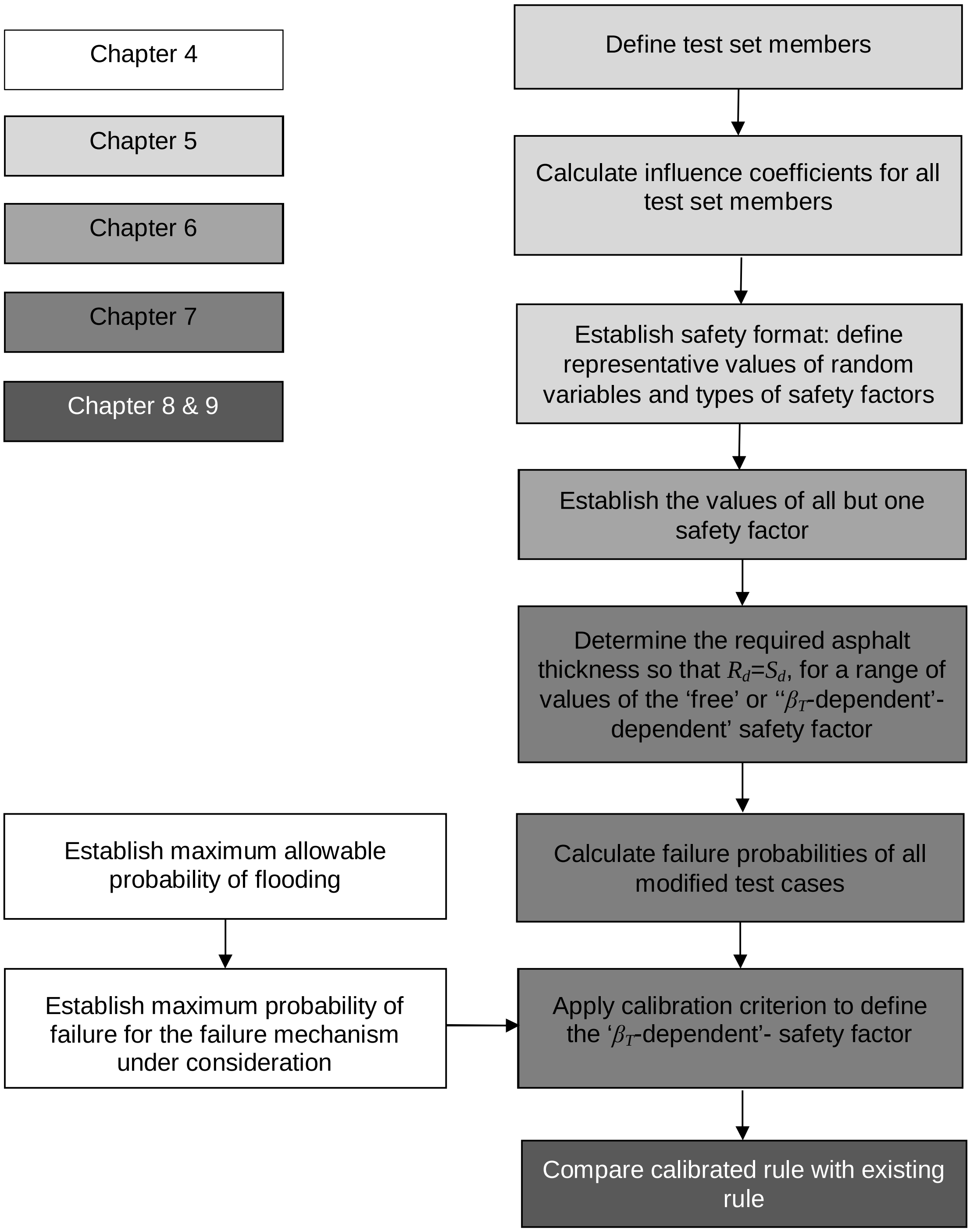


Figure 3.1 Flow chart for the calibration procedure

4 Establishing the reliability requirement

This chapter deals with translating maximum allowable probabilities of flooding to failure mechanism specific allowable failure probabilities. Section 4.1 introduces probabilities of flooding, after which the failure probability budget is introduced in Section 4.2. Section 4.3 further specifies this into a specific probability of flooding due to failure of asphalt revetments under wave attack. Section 4.4 gives some considerations on residual strength. Several parts of this Chapter are copied or modified from Jongejan (2014).

4.1 Maximum allowable probabilities of flooding

In late 2014 new safety standards will be defined in terms of maximum allowable probabilities of flooding. Without such standards, it would be impossible to decide on appropriate (partial) safety factors. Since the maximum allowable probabilities of flooding can vary per segment, the safety factors will be defined as a function of maximum allowable probabilities of flooding.
Because of the length effect (caused by imperfect spatial correlations), the same safety standard may lead to different cross-sectional reliability requirements for different dike sections, depending on its length. It is thus important to specify the spatial units to which the maximum allowable probabilities of flooding apply, in Section 6.1 this is discussed in further detail. The new safety standards will apply to so-called segments. A segment is (part of) a levee system and consists of multiple dike sections (Jongejan, 2014).
Differentiating flood safety standards within levee systems allows for a closer link between the stringency of safety standards and considerations regarding the acceptability of risks. In general dike 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).

4.2 Reliability requirements 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 way that their combined value does not exceed the maximum allowable probability of flooding.

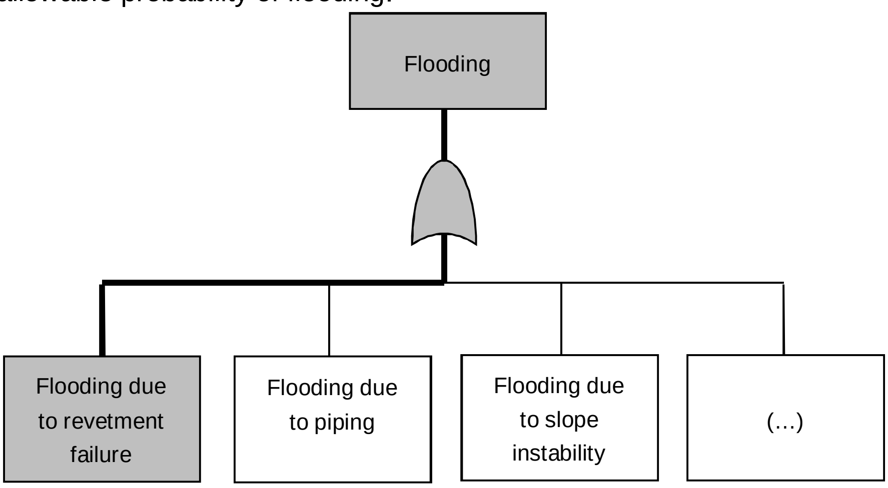


Figure 4.1. A fault tree with different failure mechanisms.

The maximum allowable contributions of the different failure mechanisms to the probability of flooding are shown in Table 4.2. These are based on the expected importance of the different failure mechanisms if all levee systems were to meet their (assumed) safety standards. Further details about the maximum allowable failure probabilities per failure mechanism, can be found in Jongejan (2013).

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

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

The choice for the term 'revetment failure and erosion' in Table 4.2 is deliberate, even though the failure mechanism is commonly referred to as revetment failure only. In levee safety assessments, the residual strength of levees can be ignored. This would be equivalent to assuming there is no residual strength. For now, residual strength of asphalt revetments is not considered. Given the definition of the failure mechanism (initiation of cracking) this might however be necessary as there is a large difference between a small crack in the asphalt layer and actual flooding.

4.3 Reliability requirements for asphalt revetments under wave attack

The 10%-value in Table 4.2 relates to all revetments, not only asphalt revetments, and to a range of (sub-)failure mechanisms, see Figure 4.2. This study is concerned solely with the reliability of asphalt revetments under wave attack.
A reliability requirement for asphalt revetments and this particular failure mechanism can, again, be derived from a fault tree analysis.

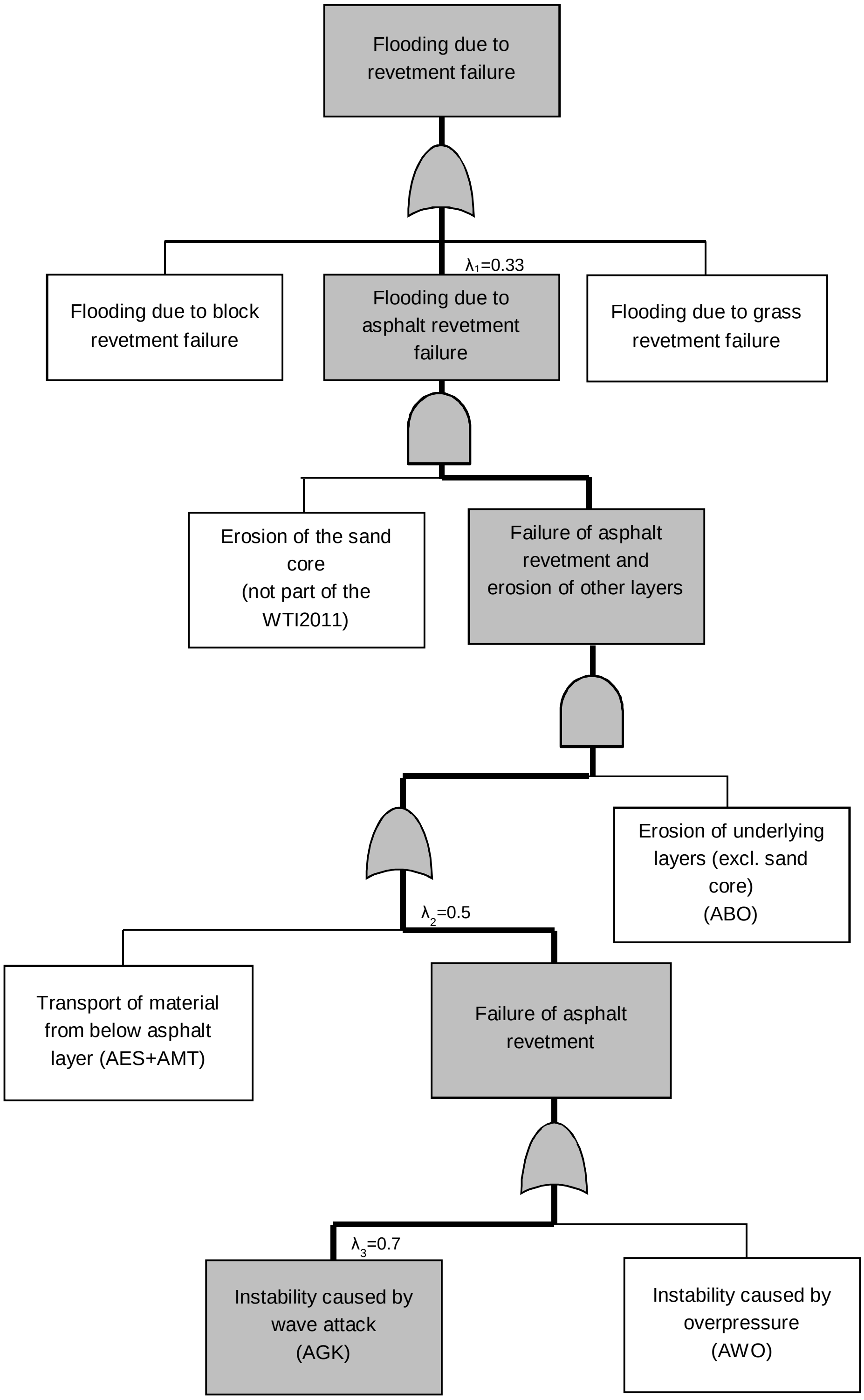


Figure 4.2. Fault tree for flooding due to revetment failure. The parts of the fault tree that this study is concerned with have been highlighted.

From the fault tree in Figure 4.2 it follows that the allowable probability of flooding caused by revetment failure is given by $P = f * P_{norm}$ with f=0.10. However there are several contributions to this failure probability, both by other failure mechanisms for asphalt revetments and failures of other revetment types. Let λ_1 be the contribution of *asphalt revetments* to the probability of flooding due to revetment failures (all types), λ_2 be the contribution of failures of asphalt revetments compared to failures due to material transport, and λ_3 the contribution of failures caused by *wave attack* to the overall probability of failure of an asphalt revetment. The reliability requirement for the stability of asphalt revetments under wave attack then becomes:

$$P_T = f\lambda_1\lambda_2\lambda_3 P_{norm} \tag{5.1}$$

where P_T is the maximum allowable probability of flooding due to the series of events triggered by the instability of an asphalt revetment under wave attack that lead to flooding. Note that the reliability of the sand core is ignored (see also see section 4.4). This is conceptually similar to assuming its conditional failure probability (failure of the levee given failure of the revetment) is equal to one.

The values for λ_1, λ_2 and λ_3 have been established as follows:

λ_1: For most dikes an asphalt revetment is often combined with other revetments. The other revetments are block and grass revetments. Therefore a sensible default value is λ_1 = 0.33.

λ_2: Asphalt revetments can either fail due to transport of material due to damage already present before the storm (AES+AMT), or failure of the revetment during the storm (AGK+AWO). From expert sessions in the past it has been concluded that initial damage is very important for failures of asphalt revetments ('t Hart, 2008), therefore a low value for λ_2 would be appropriate. However new rules as to the judgement of damages will be supplied by WTI-2017 and these have to be implemented in the "continu inzicht" (continuous insight) process. At what time this will be done is not clear yet, and there is also a discussion going on the assessment criteria. It is expected that regular maintenance measures should be sufficient to avoid material transport, which would imply a lowering of λ_2. As we expect that the water boards might have difficulties in (quickly) adapting the maintenance strategy to the new more severe requirements, a contribution of 0.5 to the probability of failure for material transport is suggested, which results in λ_2= 0.5.

λ_3: Failure of asphalt revetments during the storm can be caused by two failure mechanisms: overpressure (AWO) and failure due to wave impact (AGK). Wave impact is the most important of the two, therefore a relatively high value of λ_2 is chosen: λ_2 = 0.7.

The abovementioned values of *f*, λ_1, λ_2, λ_3 and the resulting maximum allowable failure probabilities for asphalt revetments under wave attack (P_T) are shown in Table 4.2. The reliability requirements are also expressed in terms of reliability indices (β_T).

Table 4.2. Reliability requirement for a range of arbitrarily selected safety standards (P_{norm}).

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	0.33	0.5	0.7	1/300	3.85E-05	3.95
				1/1000	1.16E-05	4.23
				1/3000	3.85E-06	4.47
				1/10000	1.16E-06	4.72
				1/30000	3.85E-07	4.94

It should be noted that the reliability requirements (P_T or β_T) in Table 4.2 apply to *entire* segments. These should *not* be confused with cross-sectional reliability requirements. Due to the length effect (i.e., imperfect spatial correlations in relation to a limited number of measurements), cross-sectional reliability requirements will have to be more stringent than reliability requirements for entire segments.

The difference between the reliability requirement for an entire segment and the reliability requirement for individual cross-sections will increase with decreasing spatial correlations and decrease with greater variability in cross-sectional reliabilities. The latter is because the failure probabilities of the weakest cross-sections will dominate the failure probability of the entire segment as the weakest cross-sections have relatively high probabilities of failure. 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 6.1.

4.4 Residual strength of asphalt dikes

Failure of asphalt revetments due to wave impact starts with cracking of the asphalt layer: the occurrence of a crack is defined as failure. However, there is a considerable difference between a crack and actual failure of the dike. As a crack in the asphalt cover causes a stress concentration at that location, it is likely that crack initiation also leads to further cracking and failure of the asphalt cover. When there is a gap in the order of 2 x 2 meter in the asphalt cover, the dike will quickly fail as there is little residual strength of the sandy core of the dike. The residual strength after the forming of a 2 x 2 meter gap is very small: the sandy core will erode quite fast. The process between crack initiation and the forming of an actual gap might take considerable time but there is insufficient knowledge to make sensible assumptions on this process. For now, in assessments, residual strength is therefore not considered.

5 Establishing the safety format

Rules on how to deal with uncertainty of different parameters in order to obtain a sufficiently safe design in a semi-probabilistic computation are defined in the safety format. The safety format entails the definition of representative values and the types of safety factors that are to be included in the semi-probabilistic assessment rule (see e.g. Jongejan, 2013). The safety format depends on the relative importance of the uncertainties related to the various random variables. To obtain insight into the relative importance of the uncertainties, probabilistic analyses are indispensable. Section 5.2 first discusses the test set members for which probabilistic analyses were carried out. The calculated influence coefficients are discussed in section 5.3. These lie at the heart of the safety format that is detailed in sections 5.4 and 5.5. A summary is provided in section 5.5.1.

5.1 General considerations of the safety format

The general goal of the safety format is to ensure sufficient safety of a revetment given that it fulfils the semi-probabilistic assessment rules. Hence, all the uncertainties should be covered. This is typically done by using design values (that are used as input) for the assessment, which are a product of a representative value and a partial safety factor. These can be chosen in many ways. A balance has to be found between simplicity and accuracy. The best accuracy (assessment is closest to fully probabilistic assessment) is achieved in case all random variable have design values equal to their design point. This will result in a lot of partial safety factors. In WTI an easier format is applied, in which most parameter uncertainty is covered by using representative values (5% upper/lower bound values), implying partial safety factors of 1; the model uncertainty is covered by a reliability independent model factor; and the remaining uncertainty is covered by a β-dependent safety factor that ensures sufficient safety for various safety standards. This may result in assessments that are slightly different from the targeted reliability, but the difference is not expected to have a very significant influence on the required asphalt thickness. For more information, please refer to Jongejan (2013).
It must be noted that not only the safety factors determine the safety, but also the representative (characteristic) values.

5.2 Establishing a test set

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

The test set members were defined systematically, see Appendix H. The bases were datasheets from a wide range of different asphalt revetments in the Netherlands, provided by KOAC-NPC. Based on this dataset average values were defined for the key parameters after which, based on the coefficient of variation, various combinations of probability density functions for the different parameters were defined.

5.3 Defining representative influence coefficients

The relative importance of the uncertainties of the different random variables is important in order to obtain a tailor-made safety format for assessments.

In the previous study by Kanning & Den Hengst (2013) FORM influence coefficients (usually referred to as α-values) for different cases were derived.
From this it appeared that the water level and cracking strength of the asphalt were the most influential random variables. Based on 6 cases delivered by KOAC-NPC the α-values of σ_b and the water level are indeed the largest. It has to be noted that due to the used relation for water level, wave height and wave period, the load is only dependent on the water level, so all influence of the load is accumulated in the TestLevel parameter.

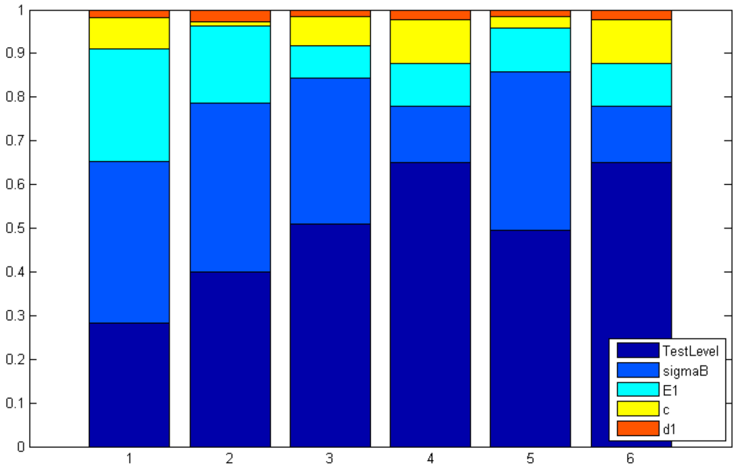


Figure 5.1 α-values for 6 cases in the Netherlands

The Young's modulus (E1) also has a considerable influence but the soil modulus and asphalt thickness are relatively unimportant. For the test set cases similar results can be found as is shown below for the different water systems. The α-values for the design water level (TestLevel) are between 0.25 and 0.8, depending on the water system and the type of test set (e.g. old/young asphalt). α-values of σ_b are typically between 0.1 and 0.4: this is also the range which is found in the test set. The Young's modulus also has some influence, but other parameters are less relevant and, due to their smaller variation and smaller contribution to the failure mechanism have α values smaller than 0.1.

Following from the influence coefficients in Figure 5.1, a test set can be defined based on the most influential parameters. The test set has different cases based on variation in design water levels, cracking strength and Young's modulus. For the mean a range of realistic values based on previous assessments was defined. For the soil modulus an average value representative for the whole range of asphalt revetments is used. Table 5.1 shows the different random variables in the test set and the coefficients of variation that have been used for these variables (note: the design water level is not shown in the graph although it is a random variable). For the design water level (maximum water level in storm) the conditional Weibull distributions as suggested in Kaste & Klein Breteler (2012) were used, where the cumulative distribution function of water level h is given by Equation 5.1:

$$F(h) = 1 - e^{\left[\left(\frac{\omega}{\alpha}\right)^{\kappa} - \left(\frac{h}{\alpha}\right)^{\kappa}\right]} \quad (5.1)$$

where α is the scale parameter, ω is the threshold value and κ is the shape parameter.
The test sets are applied to three water systems, for three safety standards, for revetments with a slope angle of 1:3 and 1:4 and for old and young asphalt quality.

After selecting relevant cases, this results in a total of 216 cases. More on the definition of the test set can be found in Appendix H.

Table 5.1 Ranges for mean and coefficient of variation for the different random variables.

Random variable	Range of mean	Variation coefficient
E_1	4000-10000	0.2 or 0.4
c	100	0.25
σ_B	5.0-7.6	0.2 or 0.35

Initially for the cracking strength, a coefficient of variation of 0.2 or 0.4 was assumed. In the test calibration using this test set a clear distinction was observed between cases with a coefficient of variation for σ_B of 0.2 and 0.4. As revetments with a coefficient of variation of 0.4 for the cracking strength are very rare a slightly less conservative value of 0.35 was used. As there is a correlation between coefficient of variation for cracking strength and the age of the revetment, the set with a coefficient of variation for σ_B of 0.2 is called 'young' and the set with a coefficient for σ_B of 0.35 is called 'old'. It has to be noted that there is not a one-on-one relation with the age, as construction quality also plays an important role.

Probabilistic computations are made for all test-set members. The resulting α-values are presented in Figure 5.2, Figure 5.3 and Figure 5.4 for the various water systems. When comparing the graphs for the different water systems it can be seen that Wadden and Kust are fairly similar in terms of the influence of the design water level (TestLevel), while for the IJsselmeer the design water level generally has more influence. This can be explained from the model used for the boundary conditions and the corresponding conditional Weibull distributions. It can also be observed that for the Kust 1/10000 case with old asphalt the influence coefficient of σ_B is equal to or bigger than for the other cases: thus for revetments with a high design standard and old asphalt (and thus a large coefficient of variation for the cracking strength) the variation of the cracking strength is more dominant than the variation of the load.
For the IJsselmeer small changes in design water level lead to relatively large changes in wave heights. Therefore the α-values for the design water level are larger. Furthermore, also the absence of tides, resulting in more concentrated wave attack on the revetment, results in a higher contribution of the design water level (TestLevel).
Please note that in these figures also parameter m is found, this is the model uncertainty factor which will be further introduced in Section 6.4.

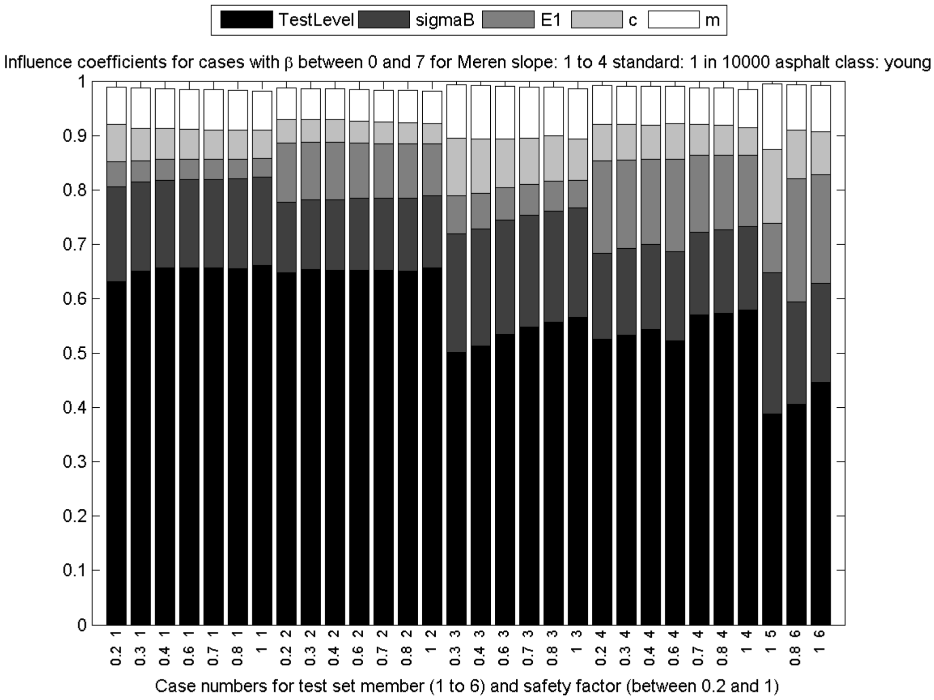


Figure 5.2 Influence coefficients (α-values) for water system Meren (Lakes) with slope 1:4, asphalt class 'young' and a 1/10000 year design level; with TestLevel being the maximum water level in the storm, sigmaB the crack-strength, E1 the Young's modulus, c the soil elasticity and m the model uncertainty

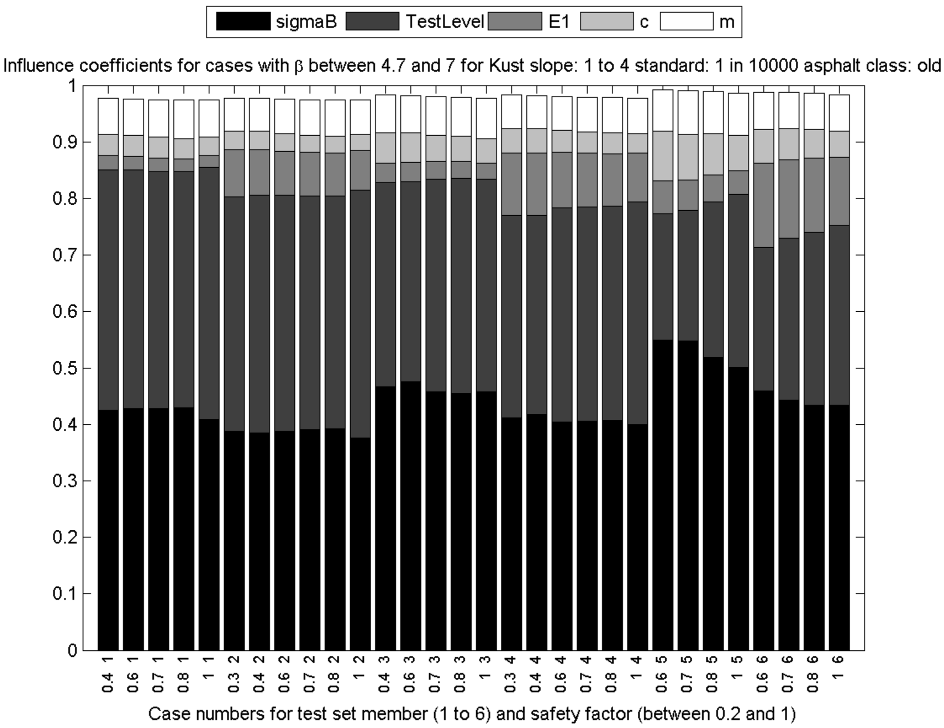


Figure 5.3 Influence coefficients (α-values) for water system Kust with slope 1:4, asphalt class 'old and a 1/10000 year design level; with TestLevel being the maximum water level in the storm, sigmaB the crack-strength, E1 the Young's modulus, c the soil elasticity and m the model uncertainty

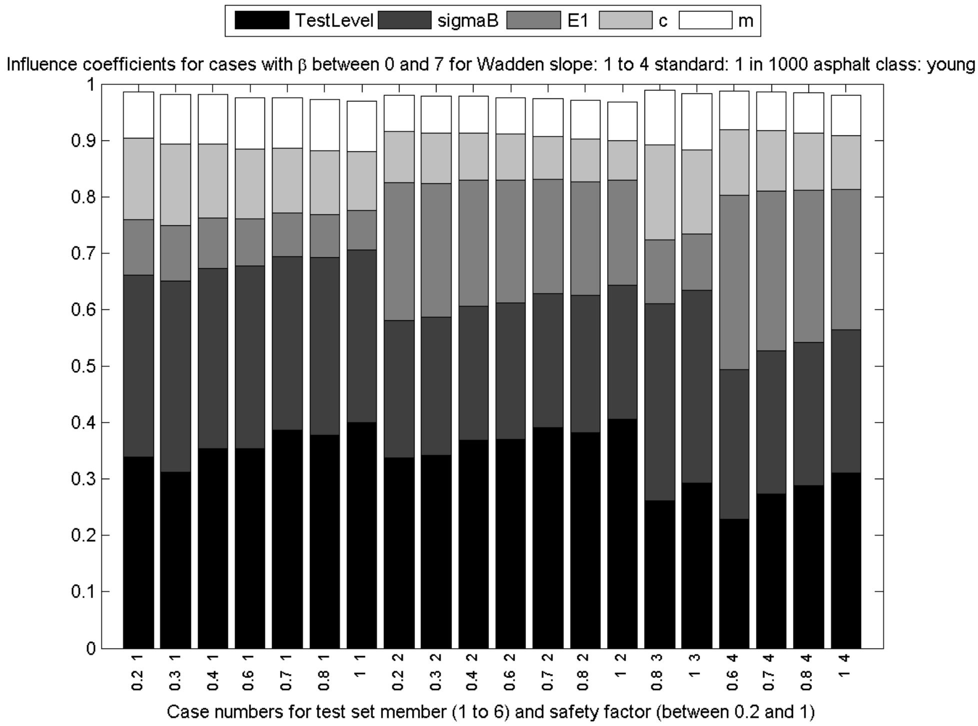


Figure 5.4 Influence coefficients (α-values) for water system Wadden with slope 1:4, asphalt class 'young' and a 1/1000 year design level; with TestLevel being the maximum water level in the storm, sigmaB the crack-strength, E1 the Young's modulus, c the soil elasticity and m the model uncertainty

5.4 Choice of representative values for the assessment

Current practice is to use 5/95% percentile values for all parameters considered as random variables in the calibration, according to the main rules shown in Section 2.3. There is no reason to change this as it is a uniform and consistent approach for the different variables. Only for the model uncertainty factor the average value is used, further explanation on this is given in Section 6.4. The distributions used in current practice, for instance the use of student-t distributions and normal distribution, sometimes combined with expert judgement, is not easily implemented in probabilistic calculations. Normal and student-t distributions can result in parameter values below zero, causing crashes or unreliable probabilistic calculations. Student-t distributions can be easily converted to normal distributions in the test set, under the assumption of perfect knowledge of the distribution (hence, a large number of measurements). Next to that, all distributions are changed to lognormal, for which Kanning & Den Hengst (2013) showed that this was a good fit to the actual data. Assuming different distributions can yield slightly different representative values, but the differences are minor, as is shown in Table 5.2, where the representative values from the assessment data provided by KOAC-NPC and representative values from the lognormal distribution are given for the case of Negenboerenpolder. In this case it can be seen that for *E1, d* and *c* the representative values are nearly identical. For the cracking strength a higher value is found due to the limited number of samples (N=8). This lowers the representative value from the data due to the use of a student-t distribution. However, in the test set perfect knowledge is assumed, so this is not a problem.

Table 5.2 Comparison between methods for determining representative values from data and from the lognormal distribution used in the calibration.

Location	**Parameter**	**Representative value from data**	**Representative value from lognormal distribution**
Negenboerenpolder	E1 [MPa]	12197	11763
	d [mm]	181	179
	c [MPa/m]	137	136
	σ_b [MPa]	3.24	3.89

Regarding the boundary conditions, within the WTI2017, the representative load is usually a load with an exceedance probability equal to the maximum allowable probability of flooding (Jongejan, 2013). For asphalt revetments, the representative values of the load parameters will therefore be obtained (with the so-called Q-variant) for target probabilities equal to the maximum allowable probabilities of flooding. This ensures consistency across failure mechanisms and facilitates comparisons between today's rules and the WTI2017. The same is done in the design step in the calibration, albeit with a simplified probabilistic model for the boundary conditions for the different water systems.

Theoretically, one can chose a safety factor for each random variable (resulting in design/assessment that is closest to the desired target reliability, but that is not very practical), or only one safety factor (that is simple to use but has higher deviations from the desired target) or anything in between. It is common practice in dike assessments to capture most uncertainty by the choice of representative values in combination with a few safety factors. The representative values are often characteristic values (5% upper or lower bounds), implying a safety factor of 1 (Jongejan, 2013). For asphalt these considerations resulted in the choice as shown in Table 5.3.

Table 5.3 Percentiles used for representative values of random variables

Random variable	**percentile value**
Water level (h)	based on target safety level
Soil modulus (c)	5%
Young's modulus (E1)	95%
Asphalt layer thickness (d1)	5%
Cracking strength (σ_b)	5%
Model factor (γ_m)	50%[1]

5.5 Safety format with safety factors

Given the ranges for Miner sums it is proposed to use the logarithm of the Miner-sum as starting point. This would give more homogeneous results, as Miner sums tend to vary between 0 and 30 for the cases considered in the report by Kanning & Den Hengst (2013), and even between 0 and 600 for the cases from the large dataset provided by KOAC-NPC, which is shown in Appendix A. The proposed *limit state function* for wave impact on asphalt dikes is therefore:

[1] *The model factor, or model safety factor, is the partial safety factor to cover the model uncertainty (random variable) m. In the semi-probabilistic assessment, the representative value of m is equal to 1 and the design value of the model factor (representative value times safety factor) is the 50%-value of the statistical distribution of m; see also section 6.4.*

$$Z_A = -\log_{10}(\mathrm{m} \bullet Miner) \qquad (5.2)$$

With:

m the lognormally distributed model uncertainty.

Another reason to use the logarithm in the limit state function is that FORM-calculations become more stable.

As the Miner sum has shown to give values in a large range, mainly due to its very non-linear definitions, a conventional safety factor on the Miner sum would result in very large safety factors. Even though expected, given the definition of the Miner sum, this is not in line with safety factors of other failure mechanisms. Therefore a semi-probabilistic safety format of the form:

$$\gamma_m \bullet \gamma_s^* \bullet Miner < 1$$

Where γ_m is the model factor and γ_s^* the beta-dependent safety factor is not feasible.

The first step in the calibration is a semi-probabilistic design based on the representative values of the different parameters, see Chapter 2. As the logarithm of the Miner sum is used in the limit state function, it is also necessary to use a logarithmic parameter as safety factor, otherwise safety factors would still range between 1 and 30 or more, while safety factors between 1 and 2 are common. The *proposed safety format* for the semi-probabilistic design is therefore:

$$\log_{10}(\gamma_m Miner) < -\gamma_s \qquad (5.3)$$

With $\gamma_s = \log_{10}(\gamma_s^*)$.

This safety factor is β-dependent and differs per safety standard, also length effects are accounted for in this factor. This safety factor should be derived from the β-γ relations following from the calibration.

The model factor γ_m is the partial safety factor of the model uncertainty of the WaveImpact model. γ_m is equal to the mean of the underlying distribution and the representative value for m is assumed to be 1, so γ_m equals the mean value of m. The derivation of m is further discussed in Section 6.4.

5.5.1 Summary of the safety format

The safety format is defined as follows:

1. All representative values for random variables in the calibration are 5/95% values as defined in WTI2011. Only the model uncertainty is treated differently.
2. Boundary conditions are derived from the Q-variant model for the exceedence probability equal to the probability of flooding for the dike segment.
3. The model safety factor consists of a representative value times a partial safety factor. For simplicity, this product is equal to the mean of the model uncertainty distribution (thus assuming the representative value is the mean of the model uncertainty and the partial safety factor is 1). Further derivation of this is given in Section 6.4.
4. The model factor γ_m is β-invariant, while the β-dependent safety factor γ_s follows from a β-γ relation for the different water systems.

Considerations on the impact and on alternative forms of the safety format can be found in Chapter 8 and Appendix G respectively.

6 Establishing representative values and the model safety factor

This chapter discusses the derivation of the model safety factor and the representative values for semi-probabilistic assessments of asphalt revetments 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 6.1. The length effect and how this is dealt with is discussed in 6.2. Section 6.3 deals with correlations with overtopping. Section 6.4 then deals with the β_T-invariant model uncertainty factor. The used representative values for the input parameters are shown in Section 6.5. The derivation of γ_s is subsequently discussed in Chapter 7.

6.1 Defining the calibration criterion

According to the WTI2017 calibration criteria, the failure probability of a segment should, on average, be smaller than its safety standard (Jongejan et al., 2013). The average failure probability is therefore used for calibrating safety factours. This value roughly corresponds to the 20th quantile values of the calculated reliability indices for each value of the overall safety factor, based on modelled normal distributions. Both metrics may be used in calibration exercises to relate cross-sectional reliability requirements to the results of probabilistic analyses (see Jongejan, 2013). Both metrics are also shown in the results of this calibration study.

6.2 Length-effects and spatial averaging

To be able to relate the cross-sectional failure probabilities of the test set members to the probability of flooding of a segment, length effects have to be accounted for.

Due to uncertainties in probability distributions derived from point measurements for a longer section of a dike, the cross sectional failure probabilities can differ from the failure probability of the segment. When doing measurements of uncertain parameters it is possible and likely that a distribution based on measurements is different from the real distribution. An example of this for the piping failure mechanism is shown in Figure 6.1.

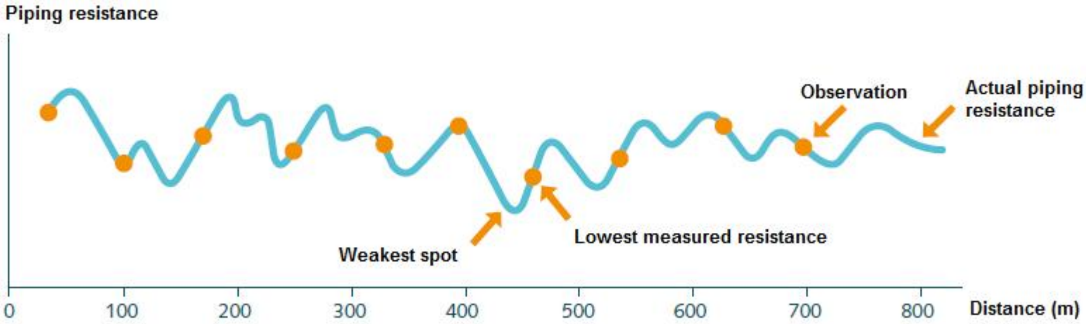


Figure 6.1 Length effect for piping resistance

As the length effect can cause a considerable difference between cross sectional and segment failure probability, it also has to be accounted for in the translation from safety standard for a dike segment to a cross sectional β-dependent safety factor.

In principle, when translating requirements for failure probabilities for a dike section or dike ring to a cross section representing a smaller segment of the dike ring, this can be done using the following formula:

$$P_{CS} \approx \frac{P_T}{(1+\frac{a \cdot L_{segment}}{b})} \tag{6.1}$$

Where:

- a the part of the ring or segment where the considered failure mechanism can occur (usually only 2 or 3 revetments contribute in one dike ring) [-]
- b is a representative scale for the length effect [m]
- $L_{segment}$ is the length of the dike segment [m]
- P_T the target probability of failure of a segment for the considered failure mechanism

b is determined based on average α-values and correlation lengths for the different random variables. In WTI2017 a standard one-dimensional autocorrelation function is assumed for every random variable, which has the form:

$$\rho(\Delta x) = (1-\rho_k) \cdot \exp\left(-\left(\frac{\Delta x}{d_k}\right)^2\right) + \rho_k \tag{6.2}$$

Where:

- Δx the distance between two cross sections [m]
- ρ_k a constant (rest)correlation of random variable X_k (1 for model parameters, 0 for parameters with spatial variation [-]
- d_k a correlation distance for random variable X_k [-]
- α_k the influence coefficient for random variable X_k [-]

Under the assumption that the autocorrelation function of the limit state function has the same form this leads to (Jongejan, 2013):

$$\rho(\Delta x) = (1-\rho_Z) \cdot \exp\left(-\left(\frac{\Delta x}{d_Z}\right)^2\right) + \rho_Z \tag{6.3}$$

With:

$$\rho_Z = \sum_{k=1}^{n} \alpha_k^2 \cdot \rho_k \tag{6.4}$$

By setting the second derivative of equation (6.3) for Δx=0 equal to the second derivatives of equation (6.2), weighed by their respective values for α_k^2, this leads to the following relation for the correlation length of the limit state function:

$$d_Z = \left(\frac{1}{(1-\rho_Z)} \sum_{k=1}^{n} \alpha_k^2 \cdot (1-\rho_k) \frac{1}{d_k^2}\right)^{-1/2} \tag{6.5}$$

Where for the length of equivalent independent reaches (*ΔL)* it holds (for cases with $\beta_{CS} > 2$):

$$\Delta L \approx \frac{\sqrt{\pi} \cdot d_Z}{\beta_{CS} \cdot \sqrt{1-\rho_Z}} \tag{6.6}$$

This ΔL is the length of an independent reach, as shown in Figure 6.2 where each element has a width of ΔL. This ΔL thus determines how many independent elements a dike section consists of, and it is therefore a very important parameter for the relation between failure probabilities of a dike segment, dike section and dike cross section. ΔL is approximately equal to b in equation (6.1).

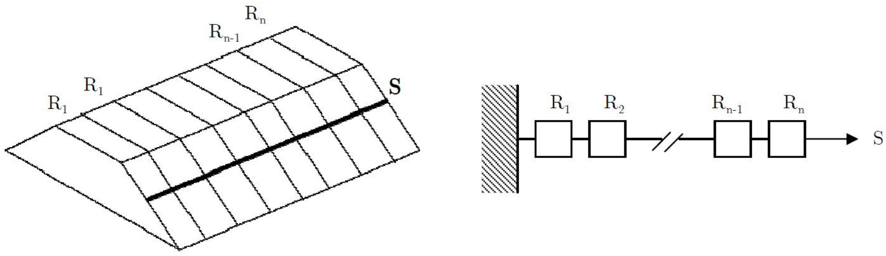


Figure 6.2 Length effects and the meaning of independent reaches (Kanning, 2012)

In general the failure probability of a cross section can be approximated by the following formula:

$$P_{CS} \approx \frac{P_T}{N_{avg}\,(1+\frac{L_{avg}}{\Delta L})} \tag{6.7}$$

Where:

N_{avg} the average number of dike sections per segment [-]
L_{avg} the average length of a dike section [m]
ΔL the average length of independent reaches for the failure mechanism considered [m]

6.2.1 Length-effects for asphalt revetment failures due to wave impact

In order to derive the length of equivalent independent reaches for failure of asphalt revetments due to wave impact the values for α, ρ_k and d_k are needed. Correlation lengths are estimated to be small for this case (Kanning & Den Hengst, 2013). Values for ρ_k are always 0 for spatially distributed random variables and values for α have been derived from a set of cases. However the problem is that there is a significant knowledge gap regarding values for d_k, ’t Hart (2014) gives some estimates as well as Kanning & Den Hengst (2013), but the relation of this knowledge to the WaveImpact calculations is not clear. In previous assessments it is very likely that a large part of the uncertainties regarding spatially varying random variables is implicitly covered in conservative assumptions in the model. Therefore it cannot be stated that the correlation lengths derived from measurements can be directly applied to WaveImpact calculations, as this can potentially lead to serious double counting of uncertainties, resulting in extremely stringent safety factors.

However due to the nature of the new safety standards, where probability of flooding of a segment is considered instead of exceedence probability of a dike section, some kind of length effect has to be taken into account.

Table 6.1 shows the effect of ΔL on failure probabilities for a dike segment with a safety standard of 1/10000 and a length of 25 kilometres.

Table 6.1 Relation between ΔL and the cross sectional failure probability

Value of ΔL *[m]*	**Failure probability for the dike segment [-]**	**Failure probability for wave impact failures for the dike segment [-]**	**Target cross sectional reliability index β [-]**	**Target cross sectional failure probability [-]**
4	10^{-4}	10^{-5}	5.96	1.23e-9
10	10^{-4}	10^{-5}	5.80	3.33e-9
25	10^{-4}	10^{-5}	5.64	8.33e-9
50	10^{-4}	10^{-5}	5.52	1.67e-8
100	10^{-4}	10^{-5}	5.40	3.33e-8
1000	10^{-4}	10^{-5}	4.97	3.33e-7

From this table the magnitude of the influence of length effects on failure probabilities is quite clear. A ΔL of 4 meters corresponds to correlation lengths of 10 meters for all asphalt parameters. This leads to a 300 times lower target failure probability than a ΔL of 1000 meters. Based on analysis of real cases, a ΔL of 4 meters would lead to disapproving a considerable amount of dike sections which are considered sufficiently safe by experts. Therefore it seems that use of a ΔL of 4 meters does not correspond to the spatial meaning of the WaveImpact calculations, as dike sections with asphalt which are considered sufficiently safe are disapproved. Therefore it has been decided, based on expert consultation (see `t Hart, 2014, reproduced in Appendix F) and analysis of old assessments to use a $\Delta L$ of 1000 meters, which is a variation to the basic option suggested in the memo by `t Hart (2014). This corresponds to the average length of a dike section while it also deals with differences between larger and smaller dike sections. In previous assessments and also in PC-Ring (although PC-Ring uses a different failure model) these values have been used without raising eyebrows. It was not feasible to implement the approach suggested in the memo by `t Hart in this stage, however this memo provides some useful suggestions for the future.
The current assumption is a pragmatic choice and research into the spatial meaning of the WaveImpact model, as well as the correlation lengths of the asphalt parameters, is strongly advised (Teixeira & Kanning, 2014).

6.2.2 Spatial averaging

In the description of the parameters according to WTI2011 spatial averaging is not taken into account. In future research, also in relation to the length effects spatial averaging also has to be considered. For instance the number of point measurements for the cracking strength are small. ’t Hart (2014) provides an indication of how these measurements can be translated to values which are relevant for the failure mechanism.

6.2.3 Implementation of length effects in Ringtoets

In assessments for asphalt revetments Formula 6.1 is used as a starting point. However the ‘1+’ in the denominator introduces additional conservatism. This value was added to exclude the possibility that the target failure probability for a cross section would be higher than for a segment. If this part of the denominator is dropped this results in the following formula:

$$P_{norm,CS} = \frac{P_T}{a * L_{segment} / \Delta L} \qquad (6.8)$$

Where $a{*}L_{segment}$ is calculated as $N_{mech}{*}L_{avg}$ with N_{mech} the number of sections where the failure mechanism can occur and L_{avg} is the average length of a dike section. This is a more accurate approximation of the length effect, but with this formula cases can occur where the cross sectional target failure probability is larger than the target failure probability of the segment. Therefore in Ringtoets the length effect is implemented in the following form:

$$P_{norm,CS} = \min\left(\frac{P_T}{a * L_{segment} / \Delta L}, P_T\right) \tag{6.9}$$

This formulation prevents unnecessary conservatism whilst assuring that target cross sectional failure probabilities can never exceed target failure probabilities of the segment. For length effect calculations this format will be used in the remainder of this report.

6.3 Correction for overtopping

For block revetments, Jongejan (2014) showed that due to correlations with overtopping the demands for the cross sectional reliability could be lowered. However, the influence coefficient of the load α_s is in the order of 0.95 for block revetments but only in the range of 0.5 to 0.6 for asphalt revetments. Hence, correction for overtopping is not incorporated.

6.4 Derivation of the model factor

The model uncertainty factor has been derived in Wichman (2014). In this report partial factors have been derived for different uncertainties in the Miner sum calculations (e.g. by comparison with finite element calculations). The following table summarizes the different partial model factors for different aspects of model uncertainty.

Table 6.2 Overview of the partial factors contributing to model factor γ_m for the respective safety aspects.

Aspect	Partial factor (range)	Notes
1a. subsoil schematisation	5.45 (in case Miner = 1.054) to 1.461 (in case Miner = 0.2486)	correction with respect to linear elastic subsoil
1b. schematisation of wave impact	0.5 (in case Miner is smaller than 1)	a mean distribution for the factor of wave impact is taken
1c. number of significant wave loadings	0.78	based on Delta flume experiments
1d. uniform material parameters in vertical	1	In agreement with choice for horizontal independent section of 1000 m
1e. changes in slope angle	1	Will be treated in schematisation guideline: take separate slope sections
2 uncertainty Miner sum calculation	1	bended fatigue line increases safety as to sequence in strength wave attacks
3 irregularities in structure	1	
4 degree of saturation of dike body	2.6 to 5.2	will not be taken into account for standard detailed assessment, so not to be implemented in γ_m.
5 input parameter determination	1	No systematic errors
6 effect of higher temperature	1.42 to 1.54	
7 residual strength	1	will cover negative effect of aspect 3

Based on the ranges defined in the table a range for the model uncertainty factor can be derived. Table 6.3 shows the derivation of minimum and maximum values for the model uncertainty factor.

Table 6.3 Overview of the partial factors contributing to model factor γ_m for the respective safety aspects.

Aspect	Partial factor (minimum)	Partial factor (maximum)
1a. subsoil schematisation	1.461 (in case Miner = 0.2486)	5.45 (in case Miner = 1.054)
1b. schematisation of wave impact	0.5 (in case Miner smaller than 1)	0.5 (in case Miner smaller than 1)
1c. number of significant wave loadings	0.78	0.78
1d. uniform material parameters in vertical	1	1
1e. changes in slope angle	1	1
2 uncertainty Minersum calculation	1	1
3 irregularities in structure	1	1
4 degree of saturation of dike body	not included	not included
5 input parameter determination	1	1
6 effect of higher temperature	1.42	1.54
7 residual strength	1	1
MULTIPLICATION γ_m *min*	**0.809**	**3.27**

Based on the minimum and maximum values given in the table a distribution was fitted. Due to the logarithmic behaviour of Miner sums a lognormal distribution for the Miner sum is used. The upper and lower bounds are assumed to be 5/95% values of the lognormal distribution. From this the mean and standard deviation of the model uncertainty factor *m* can be derived which results in μ = 1.77 and σ = 0.784.

Figure 6.3 shows the lognormal distribution for the model uncertainty factor. For the design step in the calibration a representative model uncertainty factor has to be chosen. This value is assumed to be equal to 1, with a partial safety factor equal to the expected value of the distribution of the model uncertainty factor (1.77). Further considerations on the model uncertainty can be found in Wichman (2014).

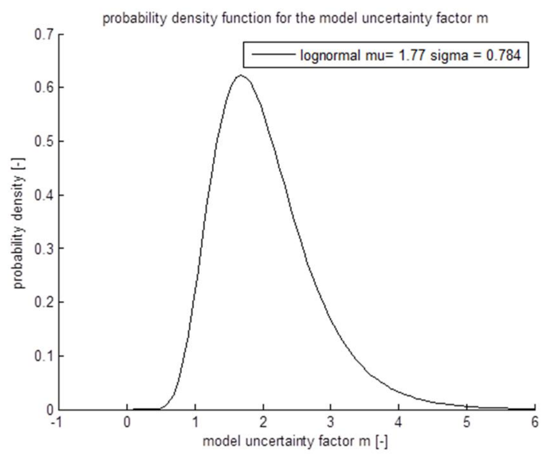


Figure 6.3 Probability density function for the model uncertainty factor m

6.5 Overview of safety factors and representative values

In the preceding sections different safety factors and representative values have been defined. Table 6.4 gives an overview of these values. The last section shows the values to be used in a semi-probabilistic assessment. The ‘value in semi-probabilistic assessment’ is also referred to as design value and is the product of the representative value times the partial safety factor.

Table 6.4 Overview of the representative values and safety factors to be used

Parameter	representative value	partial safety factor	value in semi-probabilistic assessment
Young’s modulus (E)	95%-value	1	equal to repr. value
Soil modulus (c)	5%-value	1	equal to repr. value
Test level (H)	based on safety standard	1	equal to repr. value
Cracking strength (σ_b)	5%-value	1	equal to repr. value
Model uncertainty (m)	1.00	1.77	1.77
Overall safety factor (γ_s)			Safety-dependent, see Chapter 8

7 Calibration of the overall safety factor γ_s

This chapter presents the results of the calibration of overall safety factor γ_s for failure of asphalt revetments due to wave impact. Section 7.1 presents the results of the calibration for different sub areas. Section 7.2 discusses the differences between old and young asphalt. Section 7.3 presents the safety factors following from the calibration.

7.1 Calibrating β-dependent safety factors

To calibrate the β-dependent overall safety factor (γ_s), first the test set members are modified such that their limit state function would equal 0 in a semi-probabilistic assessment given a certain safety factor. The layer thickness is used as design variable while other values were taken as representative values according to the WTI2011 guidelines for asphalt revetments (see Section 6.5). The required layer thickness to satisfy the criterion is subsequently determined (including its back-calculated distribution, assuming the required thickness is the 5% lower bound).

Next, a reliability index is calculated for each test case given the new distribution for the layer thickness. This results in a (increasing) relation between reliability indices and safety factors for the whole test set as the assessment criterion becomes more stringent. Finally, a relation through the cloud of safety factors (γ_s) and reliability indices is fitted, see also Appendix F. For more information about the calibration process, see Jongejan (2013).

7.2 Classification of old and young asphalt

Based on the data from field observations obtained from previous assessments, there is a clear distinction between old (>40 years) and young asphalt, especially regarding the coefficient of variation for the cracking strength. Therefore for the resulting β-γ_s relations, a distinction is made between old and young asphalt, as these result in significantly different safety factors (see e.g. Figure 7.3 and Figure 7.4). The difference between the two is the coefficient of variation for the cracking strength, which is 0.2 for young, and 0.35 for old asphalt. Due to the large difference between the two cases, clear rules have to be set on which safety factor to use in which case. Although the classes are called ‘old’ and ‘young’, the coefficient of variation for the cracking strength is not entirely age dependent, but it also strongly depends on the construction quality of the revetment. Therefore it is proposed to determine safety factors based on the coefficient of variation for the cracking strength that is measured rather than on age. For coefficients of variation < 0.2 the safety factors for young asphalt are used, for coefficients of variation >0.35 a detailed assessment is required. For coefficients of variation between 0.2-0.35 a linear interpolation of the safety factors is used. For example: if for a certain case a γ_s of 0.72 is found for old, and 0.55 for young asphalt, a CoV for σ_B of 0.28 would result in a γ_s of ((0.72-0.55)/0.15) *0.08+0.55 = 0.64. Figure 7.1 shows the relation between γ_s and the coefficient of variation of σ_B.

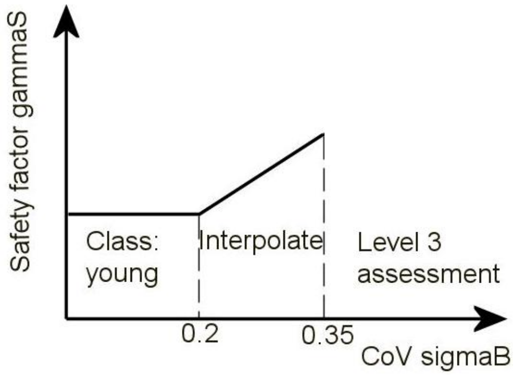


Figure 7.1 Relation between γ_s and coefficient of variation of the cracking strength

7.3 Calibration results

This section shows the result of the calibration. First the relation between the reliability index and beta- dependent safety factor γ_s is shown in Section 7.3.1. This relation is subsequently fit to an analytical relation between the target reliability of a cross section and the beta-dependent safety factor γ_s in Section 7.3.2. This relation is transferred to the relation between target reliability of the system and the safety factor γ_s by including the length-effect. Finally, the effects of the safety factors on the required asphalt thickness are shown in Section 7.3.4.

7.3.1 General relation between reliability index and safety factor γ_s

The results of the calibration of γ_s as function of reliability index β_{cs} of the cross-section are shown in Figure 7.3 to Figure 7.8 on the next pages for the different water systems and different asphalt classes (black dashed lines are fitted β-γ relations). The presented results are the fits through the cloud of realizations for γ_s, β_{cs} points that are based on the test set, an example for a single case is shown in Figure 7.2. Two criterions are presented, the 20th percentile fit and the average failure probability fit (see Appendix F), based on these two the black dashed line has been fitted. Three water level exceedence frequencies are used according to the test set, see Appendix H.

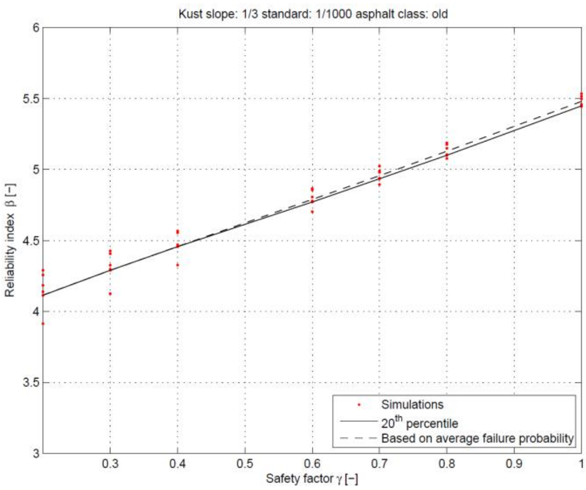


Figure 7.2 20th percentile of β and average failure probability fit

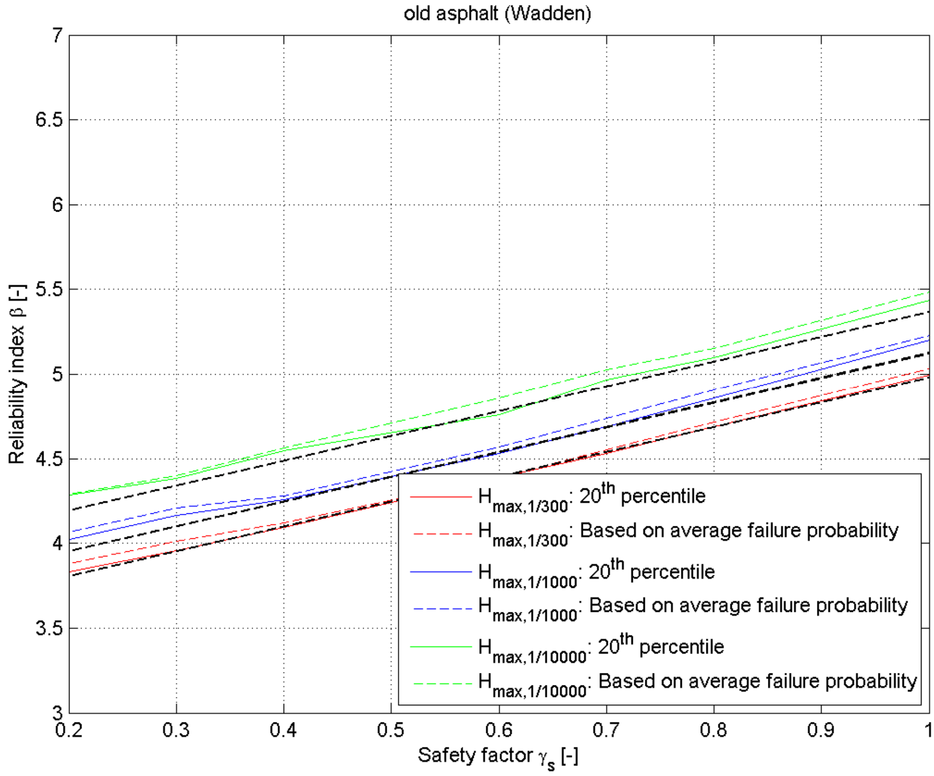


Figure 7.3 β_{cross} -γ_s *relation for old asphalt for the Wadden sea*

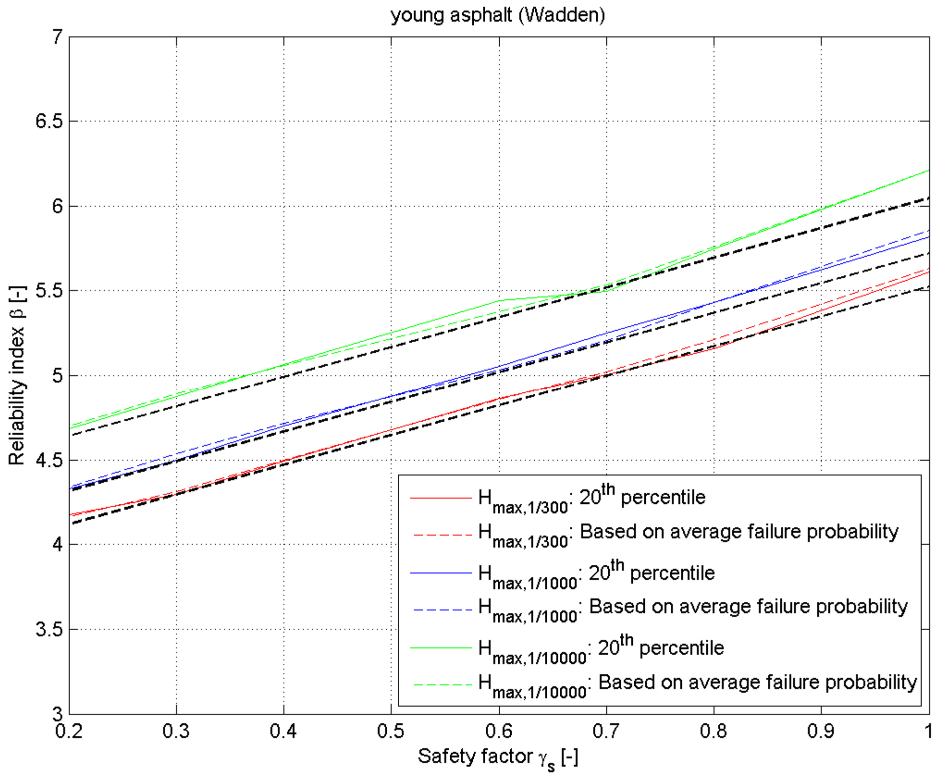


Figure 7.4 β_{cross} -γ_s *relation relation for young asphalt for the Wadden sea*

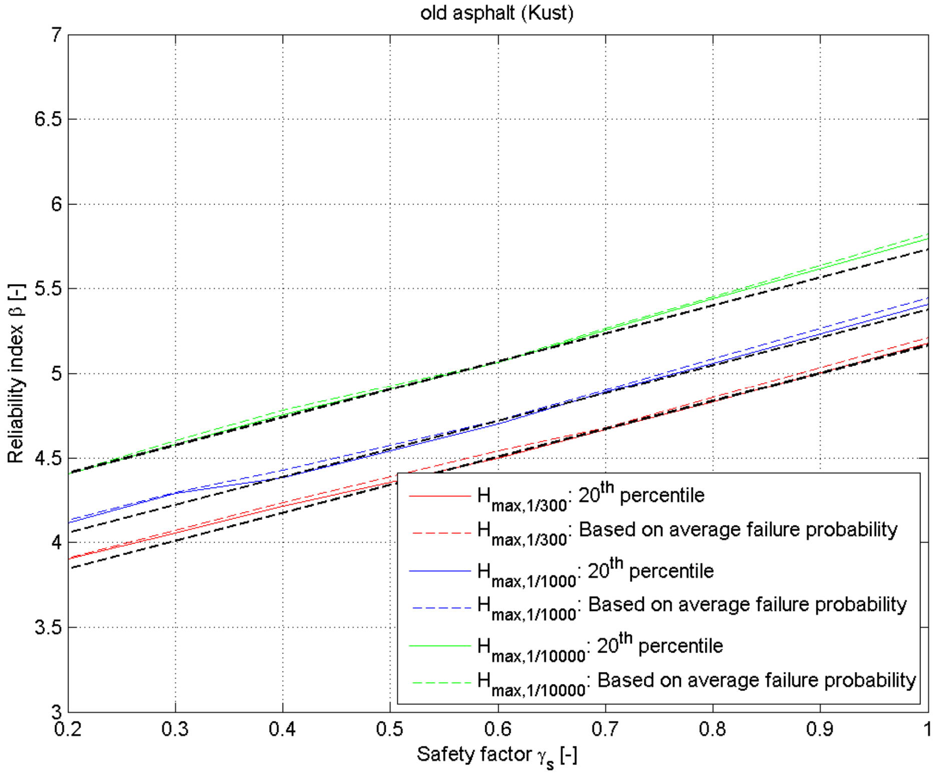


Figure 7.5 β_{cross} -γ_s *relation for old asphalt for the Coast/Western Scheldt*

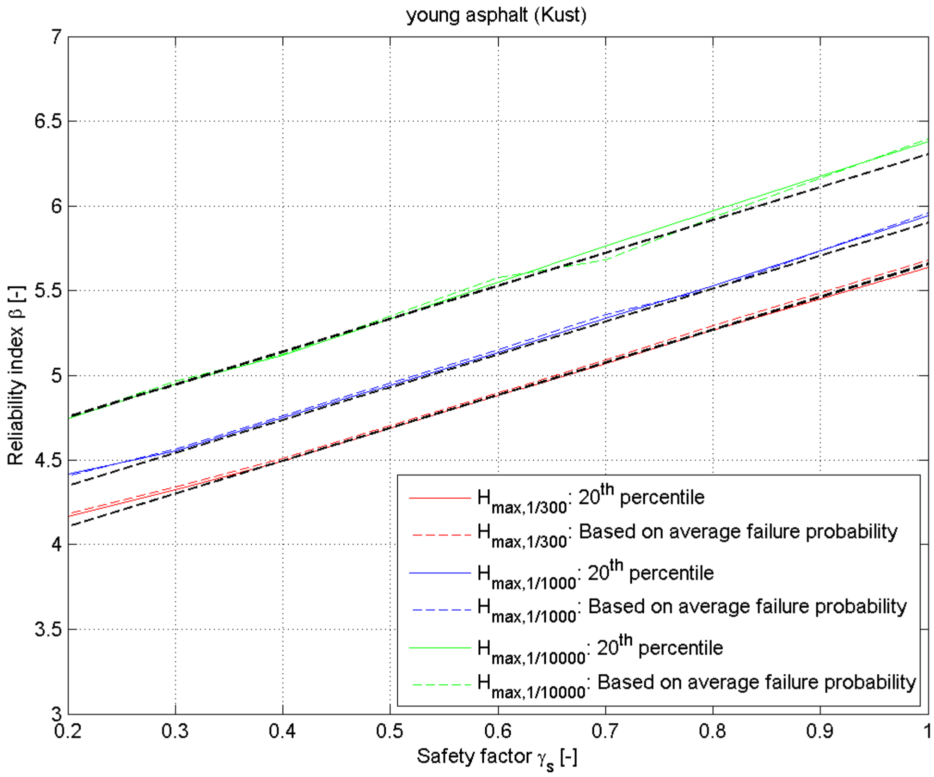


Figure 7.6 β_{cross} -γ_s *relation for young asphalt for the Coast/Western Scheldt*

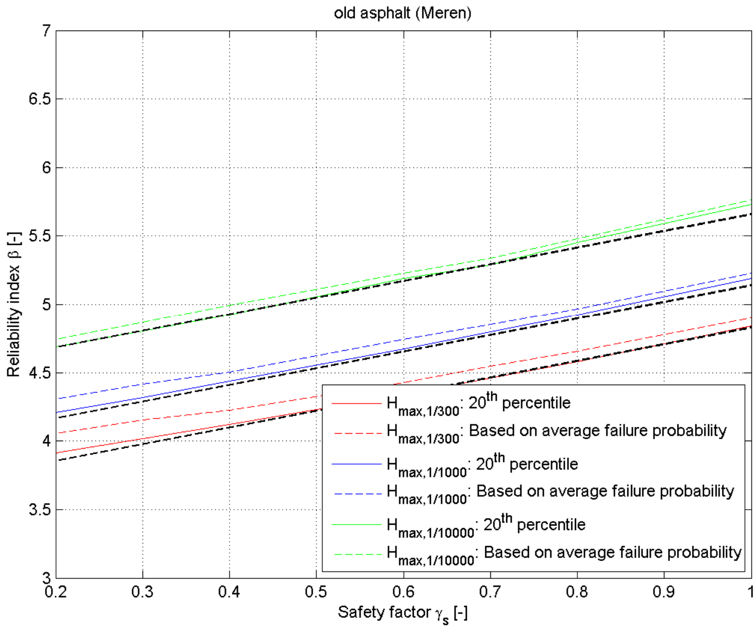


Figure 7.7 *β_{cross} -γ_s relation for old asphalt for the IJssel Lake*

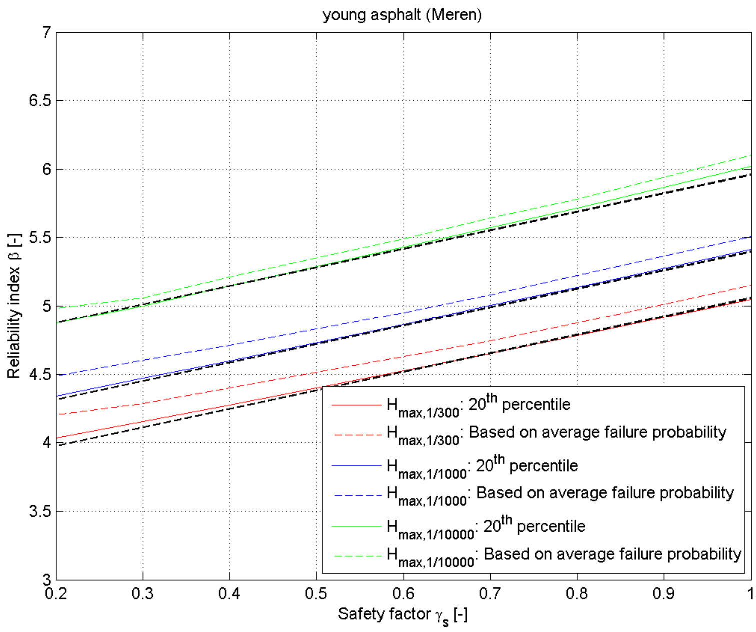


Figure 7.8 *β_{cross} -γ_s relation for young asphalt for the IJssel Lake*

For the different subsystems the β_{cross}-γ_s-relationships give a similar impression, although the curves for the Western Scheldt and Wadden Sea are slightly steeper than for Lake IJssel. This is due to a different relation between wave height and design water level for the different water systems. For Lake IJssel a small increase in water level leads to a relatively larger increase in significant wave height compared to the other water systems. This is the reason that the for Lake IJssel the lines are further apart than for the other water systems.

7.3.2 Analytical relation between target reliability of a cross-section and the β dependent safety factor

For a given cross-sectional reliability requirement β_T, the values of the β-dependent safety factors (γ_s) can be obtained from the figures on previous pages. Note that a more stringent safety factor corresponds to a more extreme representative load and thus a more stringent design parameter.

The β_T-dependent safety factor for an asphalt revetment has the following format, see also Chapter 4:

$$\gamma_s = c_a \cdot (\beta_{T,cs} - c_b) + c_{norm} \quad \text{with:} \quad \beta_{T,cs} = -\Phi^{-1}\left(min(\frac{f\lambda_1\lambda_2\lambda_3 P_{segment}}{a * L_{segment} / \Delta L}, f\lambda_1\lambda_2\lambda_3 P_{segment}) \right) \qquad (7.1)$$

Where:

c_a,c_b	constants
c_{norm}	a coefficient that depends on the safety standard
f	the maximum allowable contribution of revetment failure to the probability of flooding (f =0.1)
λ_1	the contribution of *asphalt revetments* to the probability of flooding due to revetment failures (all types) (λ_1=0.33)
λ_2	the contribution of failure of the asphalt layer to the overall probability of failure of an asphalt revetment (λ_2=0.5)
λ_3	the contribution of failures of asphalt revetments caused by *wave attack* to the probability of failure of an asphalt layer (λ_3=0.7)
$P_{segment}$	failure probability of the dike segment
$L_{segment}$	length of the dike segment [m]
ΔL	the average length of independent reaches for the failure mechanism considered [m]
a	Part of the dike segment where the failure mechanism can occur [-]
$\beta_{T,cs}$	Cross sectional target reliability index [-]

To derive the formulas in Table 7.1, a least squares fit was made for the calibration results. The fitted lines are represented by the black dashed lines in Figure 7.3 - Figure 7.8. To obtain conservative values an additional penalty was applied for points where the fitted line overestimated β_T. More information on the fitting method of the β-γ relations can be found in Appendix F.

Table 7.1 $\beta_{,cross}$ -γ_s relations for the different subsystems and asphalt classes

Water system	Age class	β_T-dependent safety factor
Western Scheldt/Coast	young	$\gamma_s = 0.52\left(\beta_{T,cs} - 1.97\right) - 0.33\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
	old	$\gamma_s = 0.61\left(\beta_{T,cs} - 1.99\right) - 0.34\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
Wadden sea	young	$\gamma_s = 0.57\left(\beta_{T,cs} - 2.37\right) - 0.29\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
	old	$\gamma_s = 0.68\left(\beta_{T,cs} - 2.47\right) - 0.26\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
IJssel lake	young	$\gamma_s = 0.74\left(\beta_{T,cs} - 1.28\right) - 0.66\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$
	old	$\gamma_s = 0.82\left(\beta_{T,cs} - 1.37\right) - 0.68\beta_{Norm}$ with $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$

7.3.3 Relation between system target reliability and safety factors; including the length-effect.

In order to derive the relation between the safety standard (system target reliability) and the safety factors, the length-effect has to be included, see Section 6.2.

The formulas of Table 7.1 lead to different safety factors for different probabilities of flooding in the considered water systems. Table 7.2 shows possible values for different segment lengths (under the assumption that for the whole length of the segment asphalt revetments are assessed). N, the number of independent equivalent reaches is calculated by dividing this length by a ΔL of 1000 meters.

Table 7.2 Safety factors γ_s for different safety standards for the different water systems and asphalt classes

Watersystem	Safety standard	Old			Young		
		5 km	*10 km*	*25 km*	*5 km*	*10 km*	*25 km*
Wadden Sea	*1/300*	0.55	0.65	0.78	0.32	0.41	0.52
	1/1000	0.63	0.73	0.85	0.36	0.44	0.55
	1/3000	0.70	0.79	0.91	0.40	0.48	0.58
	1/10000	0.78	0.87	0.98	0.44	0.52	0.61
	1/30000	0.85	0.93	1.04	0.48	0.55	0.65
Western Scheldt & Coast	*1/300*	0.50	0.60	0.71	0.33	0.41	0.51
	1/1000	0.53	0.62	0.73	0.34	0.41	0.51
	1/3000	0.56	0.65	0.76	0.35	0.42	0.51
	1/10000	0.60	0.68	0.78	0.37	0.44	0.53
	1/30000	0.64	0.71	0.81	0.39	0.45	0.54
IJssel Lake	*1/300*	0.57	0.69	0.85	0.46	0.58	0.72
	1/1000	0.53	0.64	0.79	0.41	0.51	0.65
	1/3000	0.50	0.61	0.75	0.37	0.47	0.60
	1/10000	0.48	0.58	0.72	0.33	0.43	0.55
	1/30000	0.46	0.57	0.70	0.31	0.40	0.52

One of the things that appears from this table is that for the IJssel Lake the safety factors decrease with an increasing safety standard. This is due to a combination of the relatively mild slope of the resulting lines in Figure 7.7 and Figure 7.8 and the distance between these lines. As was mentioned before, this is mainly caused by the different relation between wave height and water level and the different boundary condition model in general. In the probabilistic calculations for the IJssel Lake much higher α-values for the load are found, which implies that the influence of an increase in the load is higher than for the other areas.

An increase in safety standard can lead to a lower safety factor, in cases where the influence of the increase in load (due to taking a higher percentile value from the load distribution) is larger than the influence of the increase in safety factor due to the more stringent safety standard. This is the case at the IJssel Lake.
Due to the large differences between the different subareas the distinction between different water systems is necessary, as taking the highest value for each combination of safety standards would yield extremely conservative values for 2 of the 3 subareas.

The cases result in a cloud of safety factor – beta relations, through which a mean value is fitted (see Jongejan, 2013). These are the safety factors for segments, that might not be conservative enough for individual sections, but on average should be conservative enough for segments.

7.3.4 Effects of safety factors on required asphalt thickness

A good sanity check on whether the results found are realistic is to look at the resulting representative thickness. Table 7.3 gives thicknesses for the range of safety factors given in Table 7.2.

Table 7.3 Resulting layer thickness (in m) for different safety standards for the different water systems and asphalt classes

Watersystem	**Safety standard**	**Young**		**Old**	
		lower	*upper*	*lower*	*upper*
Wadden Sea	*1/300*	0.07	0.08	0.11	0.15
	1/1000	0.09	0.1	0.16	0.19
	1/10000	0.1	0.12	0.2	0.26
Western Scheldt & Coast	*1/300*	0.1	0.12	0.17	0.21
	1/1000	0.11	0.15	0.2	0.25
	1/10000	0.16	0.19	0.28	0.36
IJssel Lake	*1/300*	0.06	0.09	0.1	0.13
	1/1000	0.07	0.11	0.13	0.17
	1/10000	0.12	0.15	0.2	0.25

It can be observed that for young asphalt the thickness is roughly in the range of 7-20 cm. This is a common range also found in the assessment data available. When looking at old asphalt it can be observed that thicknesses are higher especially for the Western Scheldt & Coast. This is due to the fact that for the Western Scheldt, wave heights and thus impacts are significantly higher than at for instance the Wadden Sea. This results in very high values for the layer thickness for old asphalt at the Western Scheldt. This means that asphalt layers with a common thickness (say 25 cm) but with a bad quality will rarely pass assessments in Coastal areas, as loads are higher than at for instance the Wadden Sea. Concluding it can be said that results from the calibration are in line with thicknesses found in reality.

8 Implications of the new safety factors

This chapter presents considerations regarding the used safety format, why it was chosen and what its implications for the assessment are. Section 8.1 presents some considerations regarding the current safety format, Section 8.2 deals with the implications of the current safety format and Section 8.3 gives a brief description of the way the safety format was chosen. Most choices of safety formats are cosmetic and do not influence calibration results, only the way they are represented in e.g. assessment manuals.

8.1 Considerations on the proposed safety format

At the time the safety format was defined, correlation lengths were assumed to be much smaller than in the current and final situation. While the safety format led to very realistic values for safety factors at that time (i.e. in the range of most safety factors for levee assessment; order of magnitude between 1 and 2), the change in length effect definition resulted in safety factors which are generally smaller than 1. In engineering practice this can be perceived as counter-intuitive, as safety factors are generally expected to be above 1. On the other hand, the origin of this range of values is the logarithm in the safety format: if the logarithm would be dropped γ_s would range between 2 and 12.
It has to be noted that changes in safety format are cosmetic and do not influence the β-γ relation, therefore different options are presented in Appendix G.

8.2 Implications of the proposed safety format

To assess the implications of the proposed safety factors it should be compared to current assessment practice. If the logarithmic form of the safety format is dropped, and one general safety factor γ is used the safety format becomes $\gamma{*}Miner < 1$. In this form the safety format is best comparable to current practice. The resulting values for the safety factor γ are shown in Table 8.1, please note that required Miner sums can be calculated from the table by taking the reciprocal of γ.

*Table 8.1 Representative safety factor γ in case the safety format γ*Miner < 1 is used for different segment length, under the assumption that for the whole length of the segment asphalt revetments are assessed*

Watersystem	Safety standard	Old			Young		
		5 km	*10 km*	*25 km*	*5 km*	*10 km*	*25 km*
Wadden Sea	*1/300*	6.3	7.9	10.7	3.7	4.5	5.9
	1/1000	7.6	9.5	12.5	4.1	4.9	6.3
	1/3000	8.9	10.9	14.4	4.4	5.3	6.7
	1/10000	10.7	13.1	16.9	4.9	5.9	7.2
	1/30000	12.5	15.1	19.4	5.3	6.3	7.9
Western Scheldt & Coast	*1/300*	5.6	7.0	9.1	3.8	4.5	5.7
	1/1000	6.0	7.4	9.5	3.9	4.5	5.7
	1/3000	6.4	7.9	10.2	4.0	4.7	5.7
	1/10000	7.0	8.5	10.7	4.1	4.9	6.0
	1/30000	7.7	9.1	11.4	4.3	5.0	6.1
IJssel Lake	*1/300*	6.6	8.7	12.5	5.1	6.7	9.3
	1/1000	6.0	7.7	10.9	4.5	5.7	7.9
	1/3000	5.6	7.2	10.0	4.1	5.2	7.0
	1/10000	5.3	6.7	9.3	3.8	4.8	6.3
	1/30000	5.1	6.6	8.9	3.6	4.4	5.9

This shows that the safety factors are quite large, especially compared to previous assessments. This means that the assessment is more stringent than in the past. There are several reasons for the more stringent factors, some of the more important are:

- The safety factors are explicitly derived based on a safety consideration. The old method (implicitly) assumed the safety format resulted in sufficient safety. However, this was not directly linked to a safety standard. In fact, the only difference in the old situation between areas with a different safety standard was the water level and load used in the assessment. Hence, the explicit derivation of safety factor is for a large part due to a change in definition of what is assessed as there is now also a distinction of required Miner sums for different safety standards. Thus a large part of the change in safety factors is due to the change from probability of exceedence to probability of failure.
- Model uncertainty is now taken into account explicitly, which introduces a safety factor of 1.77.
- Length effects are now taken into account.

Given these considerations, and that results from the old situation which are not explicitly based on a safety levels, the differences between the old and the proposed situation are explainable. This gives confidence in the old and in the proposed method.

8.3 Choice of safety format

The choice for the applied safety format was made following multiple expert meetings with WTI Clusters Asphalt and Uncertainties, together with representatives of KOAC-NPC. The format of choice was considered to have the best advantages and most manageable disadvantages (e.g. by some clear examples). The main consideration might therefore be the formulation of the safety format, different safety formats and their (dis)advantages are discussed in Appendix G.

9 Example: application of safety factors for assessment of dike ring 5

9.1 Introduction

To assess the performance of the safety factors they are applied to an actual case. In this case dike ring area 5 Texel is chosen, specifically dike segment 5-2. The former dike ring has been split up into two segments as is shown in Figure 9.1. This dike segment is approximately 25 km long, of which around 16 km of the revetments is (partially) covered by an asphalt layer. Data for the different dike sections is given in Appendix C.

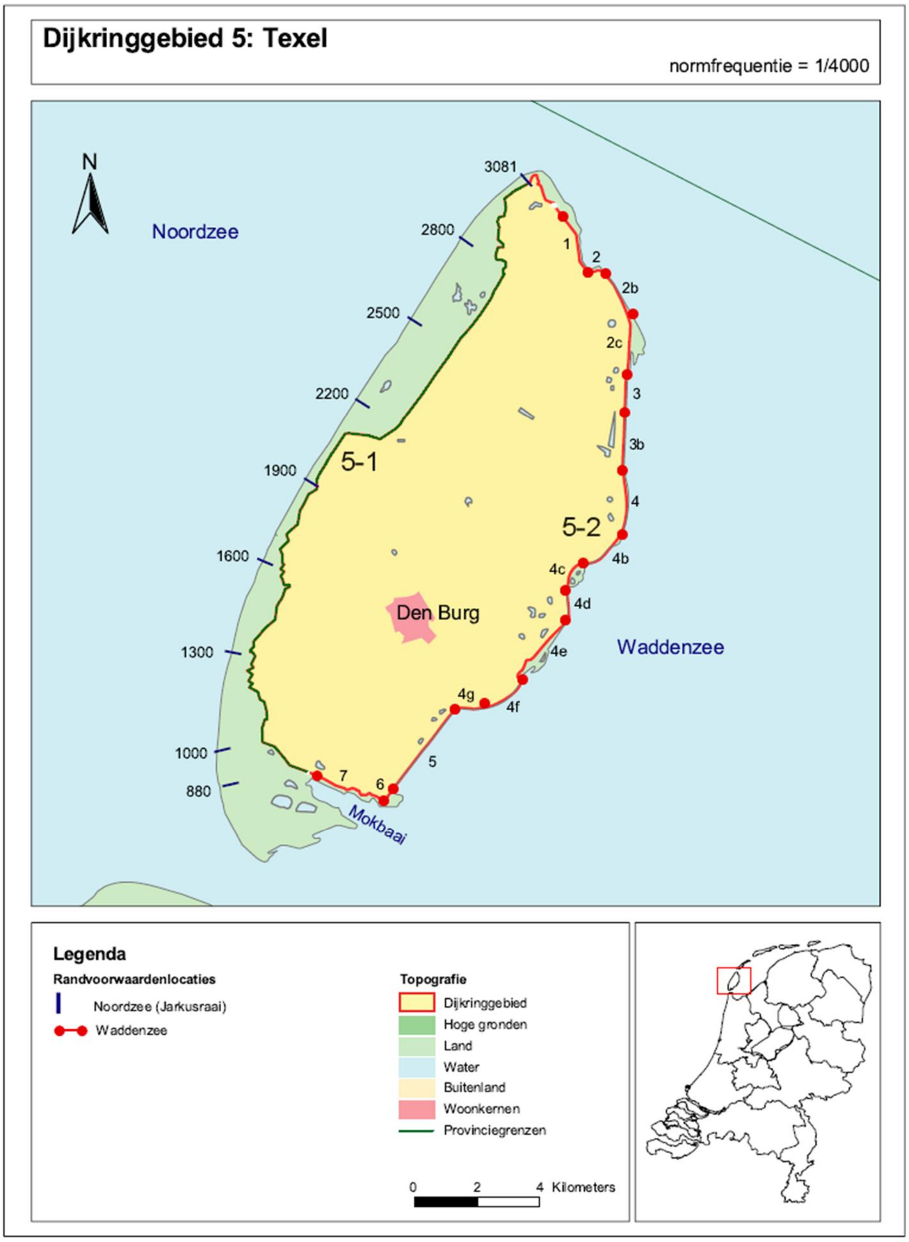


Figure 9.1 Dike ring area 5: Texel, with the two sections as defined in the new safety standards. The green section (5-1) consists of sandy coast, the red section of dikes, partially covered with asphalt.

9.2 Results of the last assessment

In 2009 KOAC-NPC carried out the assessment for the asphalt revetments in this area. Table 9.1 shows the results of this assessment. Initially 4 sections were disapproved (dike sections 1, 4f, 4g and 5), however a reanalysis of cracking strength data for sections 4f-5 led to approval of these sections as Miner sums are now below 1. Thus, in the end only dike section 1 was not approved.

Table 9.1 Assessment results for different dike sections according to last assessment in 2009, from assessment data provided by KOAC-NPC.

	Dike name	Dike section	Miner sum	Assessment result:
1	Prins Hendrikpolder	5	0,868[2]	OK
2	Gemeenschappelijke Polders	4g	0,647[2]	OK
3	Gemeenschappelijke Polders	4f	0,405[2]	OK
4	Gemeenschappelijke Polders	4e - 4c	0.006	OK
5	Gemeenschappelijke Polders	4b	0.006	OK
6	Gemeenschappelijke Polders	4	0.015	OK
7	Het Noorden	3b	0.322	OK
8	Het Noorden	3	0.104	OK
9	Eendracht	2c	0.036	OK
10	Eendracht	2b - 2	0.110	OK
11	Eijerland	1	467.829	not OK

9.3 Assessment using the semi-probabilistic assessment rules

When using the semi-probabilistic assessment rule, length effects have to be taken into account. Next to that, the assessment has to be done using boundary conditions for the water level at an exceedence probability equal to the probability of flooding as determined within WTI2017 (Jongejan, 2013). Normally this would be done using the Q-variant, but in this case it is done by using the model for the boundary conditions for the Wadden Sea that was also used in the calibration. Although this is not completely correct local differences are very small.

9.3.1 Step 1: Translate probability of flooding for the dike trajectory to target cross sectional failure probability

The first step in the assessment is to translate the probability of flooding for the dike trajectory to a target cross sectional failure probability. The probability of flooding of dike segment 5-2 is 1/1000 year. The relation between the probability of flooding of the trajectory and the target cross sectional failure probabilities can be approached by the following formula (Jongejan, 2013):

$$P_{CS} = \min\left(\frac{P_T}{a * L_{segment} / \Delta L}, P_T\right) \quad (9.1)$$

Where:

P_{CS} the averaged failure probability for the considered failure mechanism for cross sections [-]

a the fraction of the segment where wave impact failures of asphalt can occur [-]

---

[2] *For Prins Hendrik polder and sections 4g and 4f the first result was that they were disapproved. However after further study of the cracking strengths new Miner sums were determined which satisfied the criterion (Weijers, 2010)*

$L_{segment}$ the total length of the segment[m]
ΔL the average length of independent reaches [m]

With $P_T = P_{norm}*f*\lambda_1*\lambda_2*\lambda_3 = 1/1000*0.1*0.33*0.5*0.7 = 1.16*10^{-5}$ *and a*L*$_{segment}$ = *(L*$_{asphalt}$*/L*$_{segment}$*)*L*$_{segment}$ = *(18750/26100)*26100 = 18750 m* this leads to a target cross sectional failure probability:

$$P_{CS} = min(\frac{P_T}{a*L_{segment} / \Delta L}, P_T) = min(\frac{1.16*10^{-5}}{18750/1000}, 1.16*10^{-5}) = 6.16*10^{-7}$$

for ΔL = 1000m which is the default length of independent reaches.

9.3.2 Step 2: Derive γ_s from β-γ relation and check assessment criterion

The β-γ relation for respectively young and old asphalt for the Wadden Sea, are given by:

$$\gamma_s = 0.57(\beta_{T,cs} - 2.37) - 0.29\beta_{Norm}$$

$$\gamma_s = 0.68(\beta_{T,cs} - 2.47) - 0.26\beta_{Norm}$$

This leads to safety factors, which, when using the semi-probabilistic assessment criterion lead to the assessment scores in Table 9.2. The safety factors are determined based on the coefficient of variation in cracking strength.

Table 9.2 Assessment results for dike segment 5-2 Texel using the semi-probabilistic assessment rule

	Section name	Dike section	CoV of σ_b	γ_s	γ_m Miner (-)	log(γ_m Miner) + γ_s	Assess-ment
1	Prins Hendrikpolder	5	0.35	0.81	0.49	0.50	not OK
2	Gem. Polders	4g	0.35	0.81	0.88	0.76	not OK
3	Gem. Polders	4f	0.35	0.81	0.33	0.33	not OK
4	Gem. Polders	4e - 4c	0.15	0.50	0.0039	-1.90	OK
5	Gem. Polders	4b	0.15	0.50	0.0027	-2.07	OK
6	Gem. Polders	4	0.15	0.50	0.0067	-1.67	OK
7	Het Noorden	3b	0.11	0.50	0.21	-0.17	OK
8	Het Noorden	3	0.11	0.50	0.0903	-0.54	OK
9	Eendracht	2c	0.11	0.50	0.14	-0.35	OK
10	Eendracht	2b - 2	0.11	0.50	0.14	-0.34	OK
11	Eijerland	1	0.35	0.81	1015	3.82	not OK

Compared to the previous assessment 3 additional sections are now disapproved. However, in the last assessment, sections 4f-5 were only approved after reanalysis of cracking strength measurements and still had relatively high Miner sums. Due to the introduction of the model factor (which is essentially a multiplication of the Miner sum), and the change in safety standard these sections are now disapproved.

9.3.3 Comparison with probabilistic calculation

To get a view of the relation between the semi-probabilistic and probabilistic calculation it is useful to compare the assessment results to the probabilistic calculation. The results of this comparison are shown in Table 9.3. The failure probabilities per dike section are obtained from a probabilistic calculation for the representative cross section and accounting for the length effect in each section.

Table 9.3 Assessment results obtained from the probabilistic analysis

	Dike name	Dike section	β_{CS}	P_{CS}	$P_{T,CS}$	Assess-ment
1	Prins Hendrikpolder	5	5.15	1.28E-07	6.16E-07	OK
2	Gemeensch. Polders	4g	4.71	1.27E-06	6.16E-07	not OK
3	Gemeensch. Polders	4f	5.22	9.04E-08	6.16E-07	OK
4	Gemeensch. Polders	4e - 4c	7.38	7.92E-14	6.16E-07	OK
5	Gemeensch. Polders	4b	8.66	1.11E-16	6.16E-07	OK
6	Gemeensch. Polders	4	8.20	1.11E-16	6.16E-07	OK
7	Het Noorden	3b	5.55	1.43E-08	6.16E-07	OK
8	Het Noorden	3	6.74	7.81E-12	6.16E-07	OK
9	Eendracht	2c	6.44	6.11E-11	6.16E-07	OK
10	Eendracht	2b - 2	6.20	2.76E-10	6.16E-07	OK
11	Eijerland	1	3.99	3.35E-05	6.16E-07	not OK

The total target probability of failure due to asphalt failures caused by wave attack is calculated by using the following formula:

$$P_T = P_{segment} * f * \lambda_1 * \lambda_2 * \lambda_3 = (1/1000) * 0.1 * 0.33 * 0.5 * 0.7 \approx \frac{1}{90000}$$

The current failure probability for wave impact ($P_{seg,AGK}$) can be obtained by summing up P_{CS} and accounting for the lengths of the sections using the following formula:

$$P_{seg,AGK} = \sum_i \frac{L_i}{\Delta L} * P_{CS}$$ where L_i is the length of section i.

This results in a failure probability of around 1/22000 per year for the current situation, which is too high. However this is mainly caused by the section at Eijerland. If this section is not taken into account the failure probability is reduced to 1/670000 (=$1.5*10^{-6}$) per year. If we also remove section 2 which was also disapproved the failure probability decreases to $4*10^{-7}$ per year, which is way below the required safety level ($1.1*10^{-5}$=1/90000).
Therefore it can be said that the semi-probabilistic assessment is a bit stringent for this test case.

9.4 Conclusion

Based on the results for the test case it can be concluded that the semi-probabilistic assessment rule is sufficiently safe, maybe even a bit stringent for this case. Due to the addition of a model factor and a change in safety standards a few extra dike sections are disapproved compared to the previous assessment. To draw more definitive conclusions on the validity of the safety factors the same checks should be done for other cases (in other water systems).

10 Summary of semi-probabilistic assessment and comparison with WTI 2011

10.1 Comparison with WTI2011

In WTI2011 (and VTV 2006) no safety factor is used for the Golfklap mechanism and all uncertainties are assumed to be covered by the use of characteristic values of the main input parameters. The safety factors found now are a considerable change compared to previous practice. There are several reasons for this:

- Accounting for length effects: in WTI2011 length effects were not taken into account. In this study these have been taken into account, albeit in a relatively simplified manner. However, accounting for length effects automatically leads to considerably higher safety factors. In most cases this will also lead to higher safety levels.
- Accounting for model uncertainty: in WTI2011 no model uncertainty was quantified. As it is accounted for now, it is plausible that the assessment criteria will be more stringent.
- Change in safety standards: As failure probabilities are now explicitly quantified and the demands are different from previous assessments this will lead to higher safety factors for assessments.

The values for γ_s found in the calibration range between 0.3 and 1.1. This is a wide range especially as it is on a logarithmic scale. A more 'conventional' safety factor of the form $\gamma_s \bullet \gamma_m \bullet Miner < 1$ would lead to values between 2 and 12 (times the model factor).
The difference between these values for the safety factor originates mainly from the change in safety standards: due to the application of a failure probability budget with a limited space for failures due to wave impact, the criterion for assessment is more stringent.

10.2 Summary of the semi-probabilistic assessment format

This section outlines the different steps to be taken when doing a semi-probabilistic assessment for an asphalt revetment under wave attack.

1. Determine the geometry and hydraulic boundary conditions
2. Determine the asphalt and soil parameters (5% lower bound for soil modulus, and asphalt thickness, 95% upper bound for the asphalt Young's modulus)
3. Determine the 5% lower bound for asphalt cracking strength and mean values for fatigue parameter alfa and beta using the Grafiekenmaker
4. Compute the maximum Miner sum with WaveImpact using the results of steps 1 to 3; possibly for multiple slope angles

5. Based on the safety standard and length-effect for the revetment determine the required safety factor.
   a. If no better or more case-specific information is available use the following default values:

ΔL	f	λ_1	λ_2	λ_3
1000 m	0.1	0.333	0.5	0.7

   b. Calculate the required cross sectional failure probability by using:

$$P_{norm,CS} = \min\left(\frac{P_T}{a * L_{segment} / \Delta L}, P_T\right)$$ with:

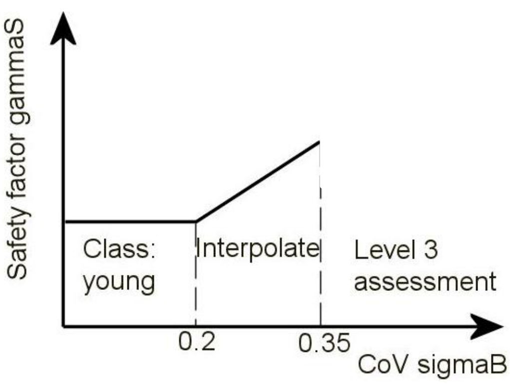
P_T	$= P_{norm}\, f\, \lambda_1 \lambda_2 \lambda_3$
$a{*}L_{segment}$	$= N_{mech}{*}L_{avg}$
N_{mech}	Number of sections where the mechanism is assessed
L_{avg}	the average length of these sections [m]

   c. Determine the safety factors for the considered water system using following relations:

Water system	Age class	β_T-dependent safety factor
Western Scheldt/Coast	young	$\gamma_s = 0.52\left(\beta_{T,cs} - 1.97\right) - 0.33\beta_{Norm}$
	old	$\gamma_s = 0.61\left(\beta_{T,cs} - 1.99\right) - 0.34\beta_{Norm}$
Wadden sea	young	$\gamma_s = 0.57\left(\beta_{T,cs} - 2.37\right) - 0.29\beta_{Norm}$
	old	$\gamma_s = 0.68\left(\beta_{T,cs} - 2.47\right) - 0.26\beta_{Norm}$
IJssel lake	young	$\gamma_s = 0.74\left(\beta_{T,cs} - 1.28\right) - 0.66\beta_{Norm}$
	old	$\gamma_s = 0.82\left(\beta_{T,cs} - 1.37\right) - 0.68\beta_{Norm}$

Where $\beta_{Norm} = -\Phi^{-1}(P_{Norm})$ and $\beta_{T,cs} = -\Phi^{-1}(P_{Norm,cs})$

   d. Determine the safety factor to be used based on the coefficient of variation of the cracking strength in accordance with the following graph:



6. The revetment complies to the safety standard if the logarithm of the product of the Miner sum times the model factor is smaller than the safety factor: $\log_{10}(\gamma_m \text{ Miner}) < -\gamma_s$ with γ_m = 1.77

11 Conclusions

11.1 Conclusions

The new WaveImpact kernel has been successfully implemented in the previously developed Matlab environment and gives reliable results for the different benchmarks and references available. However as some parameter limits were introduced, running probabilistic calculations caused some issues. These limits were removed and reliable FORM and MCIS calculations have been executed using the probabilistic failure model.

The goal of the calibration of safety factors is to ensure that an asphalt revetment has a sufficient reliability level if it passes the semi-probabilistic calculation, which includes the safety factors. For this calibration, the following safety format is chosen that should result in sufficient safety. Applied to a semi-probabilistic assessment, it involves the following steps:

1. Determine the geometry and hydraulic boundary conditions
2. Determine the asphalt and soil parameters (5% lower bound for soil modulus, and asphalt thickness, 95% upper bound for the asphalt Young's modulus)
3. Determine the 5% lower bound for asphalt cracking strength and mean values for fatigue parameter alfa and beta using the Grafiekenmaker.
4. Compute the maximum Miner sum with WaveImpact using the results of steps 1 to 3; possibly for multiple slope angles
5. Determine the safety standard and length-effect for the revetment and determine the required safety factor.
6. Check if the logarithm of the product of the Miner sum times the model factor is smaller than the safety factor
7. If this is the case, the revetment complies to the safety standard

The calibration of semi-probabilistic safety factors has been successful. However the range of safety factors poses a problem, as values of 0.5 for safety factors are quite uncommon. A possible solution could be to apply the safety factor to a certain random variable rather than the Miner sum, or rename the safety factor to safety threshold. In general however the safety factors result in realistic values for for instance the layer thickness. Next to that it has to be noted that, if the safety factors are considered in a different (more conventional) format, this will result in safety factors between 2 and 12.
It has to be noted that the safety factors are interacting: choices on the lay-out of the safety format do not influence the calculations itself, as long as the β-dependent safety factor is a multiplier of the Miner sum.

Also the magnitude of the length effects is quite uncertain, now it has been determined based on expert judgement and comparison with old assessments, however a more fundamental approach to length effects is definitely needed. To improve this, the spatial context of the WaveImpact model also has to be defined, this should be one of the main targets for further research on asphalt revetments. In order to further substantiate the validity of the model full scale tests seem necessary, as these are the best way to link the theoretical model to a real case aside from real failures.

In terms of uncertainty it is doubtful whether for instance the number of measurements (8 per dike section) for the cracking strength is sufficient.

As it turns out from the probabilistic calculations the cracking strength is one of the most important variables, therefore it is advised to further investigate the measuring methods for the cracking strength, as reducing standard deviations for cracking strength will result in a significantly better estimation of safety levels.

11.2 Recommendations

The calibration procedure is recommended to be tested on old assessments, both for the input parameters and the required asphalt thickness. Already several old cases have been assessed for this report, though mainly the Texel case is presented. Cases for the Coastal/ Western Scheldt and IJssel Lake hydraulic boundary conditions are especially recommended to be tested more elaborately by or together with e.g. the parties who executed asphalt assessments.

The model factor is newly introduced in the safety format. The values are based on the current knowledge. It is recommended to use recent and new research in asphalt fatigue behaviour to reduce the model uncertainty and thus the model factor.
This is also a good option if a dike section is disapproved: reduction of the model factor by reducing soil schematization uncertainty is a good option. Otherwise uncertainty reduction by taking more measurements may reduce uncertainty and thus increase representative values.

The derivation of the asphalt strength parameters α, β, σ_B (based on the Grafiekenmaker) is recommended to be evaluated in relation to the probabilistic assessment.

Measurement methods of for instance the cracking strength and Young's modulus should be further investigated, as large uncertainties in strength parameters currently have a large contribution to the failure probability.

It is recommended to investigate spatial fluctuations and length effects for asphalt revetments. Also, the spatial relation between correlation lengths found for different parameters and the WaveImpact model should be defined. Length effects have a significant contribution to failure probabilities and as these are now based on a pragmatic choice, it is necessary to further substantiate the relations between measurements, distributions and model results.

The safety factors found are not directly applicable to other types of asphalt such as open stone asphalt, due to a difference in material parameters. As the failure mechanism is the same the same calibration method can be applied using a different test set.

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A Failure mechanism model Golfklap

The failure mechanism “Failure of top layer revetment due to wave impact” of asphalt revetments is modelled in WaveImpact. Asphalt revetments are usually constructed directly on a sand dike body. In brief, the occurring stresses at the asphalt layer (point A in Figure 2.1) are compared to the resistance against fatigue of asphalt by calculating the so-called Miner sum. This is shown in Figure 2.1 for a fixed water level (h) and wave height (H_s). A hydraulic load model is chosen to generate a water level and wave height at each time step during a storm event. The WaveImpact model divides the asphalt revetment in discrete elements. For each time step, the occurring stresses and the fatigue resistance are determined. Finally, these are combined into a Miner sum for each discrete asphalt element for the storm event. The steps that are taken in WaveImpact are briefly summarized in the subsequent sections:

- Determination of the hydraulic load model (Section A.1.1).
- Determination of occurring stresses in the asphalt layer.
- Determination of resisting fatigue stresses.
- Determination of the Miner sum per discrete element and the maximum of the Miner sum for the section considered.

For more background information about the use of Miner’s rule, please refer to KOAC NPC (2009a).

A.1.1 Hydraulic loads

The starting point for the WaveImpact calculation is the hydraulic load, which is a storm event with a specified duration. It is possible to model different hydraulic loads (development of the water level and waves in time) for different areas in WaveImpact: North Sea, Wadden Sea, Eastern Scheldt and the IJssel Lake. Additionally there is a free input option. The most common load model consists of a storm surge with a superimposed tide, see Figure 2.2.

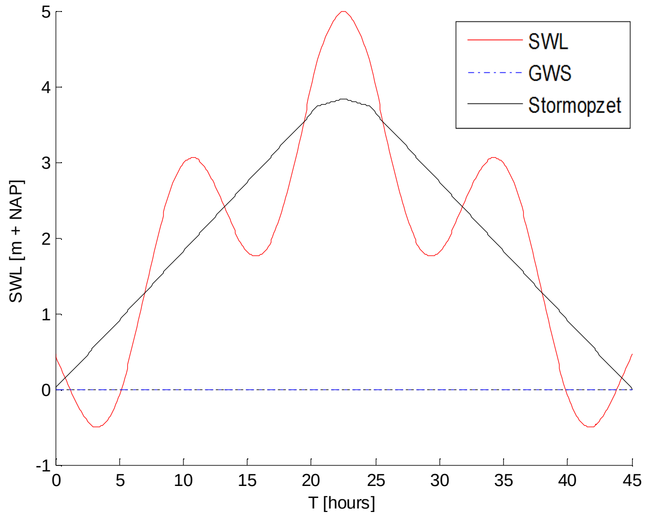

Figure 0.1 Standard storm (Wadden Sea), with SWL = Sea Water Level, Stormopzet = Storm Surge, GWS = mean sea level (0 in this figure).

A.1.2 Limits to parameters

There are several parameter limits mentioned in the WaveImpact documentation. For the purpose of calibrating safety factors however these parameter limits can lead to errors as input values can be close to the physical possibilities/boundaries. Important parameter limits are:

- The slope may range between 1:2 and 1:7.
- Maximum wave height H_s is 3 m.
- Asphalt thickness(first layer) minimum: 0.10 m.
- Maximum Young’s Modulus: 25000 MPa.

As the limits for wave height, asphalt thickness and Young’s Modulus cause crashing of the probabilistic routines, these limits are not used in the version of the kernel which is used for the calibration. Instead exceedences of the deleted limits are logged by the Matlab interface. The limits will be upheld in the final WTI version of WaveImpact.

A.1.3 Main parameters

The main parameters used in the WaveImpact model are summarized in the table below, for the complete required input of the WaveImpact model, please refer to Appendix D. In the list in the appendix parameters for double layer systems are also given. The double layer systems are not yet implemented in the Matlab environment and calibration, as they do not occur often, but may be incorporated in a later stage. The input parameters are almost the same as in the previously used Golfklap kernel, albeit that there is now also the CorrectionImpactFactor which enables accounting for load reduction due to the presence of a berm. This parameter is not used in the calibration. There is a distinction between general and representative parameters. The general parameters concern geometry and for instance the shape of the storm surge. The representative parameters are parameters which are considered as representative values, based on their respective probability distributions.

Most of them are random variables in the probabilistic assessment, although some of them are also considered deterministic, more details on this can be found in the report by Kanning & Den Hengst (2013).

Table A.1 Input parameters for WaveImpact

#	Used symbol	Description
General, load and geometry		
1	TideOption	Determines tide or no tide
2	SWLSteps	Discretization of high water
6	DurationTide	Tidal period
7	gga	Tidal amplitude
12	SurchargeType	Type of stormsurge (different for Wadden and North Sea)
13	WaveHeigthVector	Wave height
14	WavePeriodVector	Wave period
15	LevelForeshore	Foreshore level
16	ProfileY	Geometry points in y (horizontal) direction
17	ProfileZ	Geometry points in z (vertical) direction
33	NumberSlopeParts	Number of discrete elements
34	MinLevelAsphalt	Min level of considered asphalt layer
35	MaxLevelAsphalt	Max level of considered asphalt layer
36	NumberOfImpactPoints	Number of discrete wave contributions

Representative values:		
1	h	maximal water level in the storm [m+NAP]
3	c	Soil modulus[MPa/m]
5	d	Thickness asphalt [m]
7	E1	Stiffness asphalt [MPa]
9	ν	Poisson's Ratio [-]
10	α_a	Curved fatigue parameter [-]
11	β_a	Curved fatigue parameter [-]
12	σ_b	Crack strength asphalt [MPa]

A.2 Functioning of the new kernel

The new WaveImpact kernel is compared to Golfklap 1.3 and to the benchmark studies that were carried out to test the kernel. The benchmark tests were successful (see

Table A.3) and provided good results, except for benchmark test 6 which could not be executed using the Matlab environment, however this was a test with manually entered impact factors, which is an option not used in the calibration.

The 5 cases studied in 2013 give Miner sums very close to the Golfklap 1.3 dll, see Table A.3, except for case 4, which is a peculiar case as it has free input for the boundary conditions. This is an option that is not used in the calibration, so it will not pose a problem. The deviation of case 1 is known by the developers, there are deviations for high Miner sums, which is not very important as mainly Miner sums around 1 or lower than one count for assessing the revetment.

Table A.2 Comparison of Miner sums of Golfklap 1.3 and the new WaveImpact kernel

	Golfklap 1.3	**WaveImpact 14.1.1.900**
1. Lauwersmeer	30.35	32.52
2. Negenboerenpolder	0.0496	0.0514
3. Helderse	0.0029	0.0029
4. Zuiderdiep	0.8428	0.0739
5. Balgzand	0.0916	0.0945

Furthermore, the Matlab shell implementation of WaveImpact is also compared to the benchmarks in Excel. These are simplified cases, with less discretization steps, designed to test wave impact. For more information, please refer to Trompille (2013). The results are shown in Table A.3. It can be seen that the Matlab shell implementation gives exactly the same results as the benchmarks.

Table A.3 Comparison of Miner sums of WaveImpact kernel with the benchmarks

case	WaveImpact 14.1.1.900	Benchmark Excel
1	7.003	7.003
2	3.34	3.34
3	5.10	5.10
4	12.6	12.6
5a	10.159	10.159
5b	5.59	5.59
6	error: more impact factors in benchmark than is allowed in kernel	42.46
7	3.70	3.70
8	0.149	0.149
9	7.003	7.003
10	1.211	1.211
11	7.184	7.184

The only problems found with the new kernel were parameter limits or for instance the maximum slope being exceeded. Possible problems with this will be further addressed in Appendix B.

A.3 Differences between the WaveImpact kernel and Golfklap

Compared to the previous model there have not been any changes to the calculation method. However there are some changes in the structure of the program. The new kernel has not been built for standalone calculations so it does not include a user interface. In general the WaveImpact kernel gives less feedback on the performance of calculations, although some intermediate results are given as output. The new kernel also doesn't calculate for different slopes, it uses 1 representative slope for the entire revetment given as input. This interpolation can influence results, especially for revetments with a varying slope. This may be solved in a user interface that splits the revetment in various parts with constant slope and later combines the result. The old Golfklap model also had an option for making designs, this is not included anymore but may also be incorporated in a user interface.

A.4 Conclusions

Results from benchmark tests provide sufficient confidence to execute the calibration of safety factors with the new WaveImpact Kernel. However, the new version has several parameter boundaries which might cause crashes when the kernel is used in a probabilistic context. Therefore an adapted version of the kernel has been made, where these parameter boundaries are deleted and replaced by warnings in the Matlab environment, this will be discussed in Appendix B.

B Probabilistic implementation of WaveImpact

This Appendix considers the probabilistic implementation of the WaveImpact model. First the coupling with Matlab is discussed (B.1), then the adaptations necessary for using WaveImpact in a probabilistic context (B.2). After that some other aspects of probabilistic calculations are discussed (B.3).

B.1 Probabilistic coupling of WaveImpact and Matlab

In order to do a probabilistic analysis of the wave impact failure mechanism an additional shell is necessary to be able to use the WaveImpact Kernel in a probabilistic context. This is done using Open Earth Tools. Compared to the previous study by Kanning & Den Hengst (2013) the probabilistic Matlab environment uses the same procedures for deriving boundary conditions (Kaste & Klein Breteler, 2012) and generating random variables and deterministic input for the Kernel, as well as the same limit state function (albeit in a slightly different form). The procedure is similar to the one followed in the previous report, although the new kernel has some differences in programming structure. This also causes some issues, which are discussed below.

B.2 The change from deterministic to probabilistic use

Doing a probabilistic analysis with a tool primarily intended for deterministic use can cause problems, as extreme values of parameters might exceed (physical) bounds. In this case parameter limits in the original version of the WaveImpact Kernel can cause problems in some cases. Therefore these are deleted/changed in order to be able to do stable probabilistic analysis. Instead of generating an error in the Kernel, exceedences of the parameter limits are logged in Matlab. This prevents (often unnecessary) crashing, as most exceedences are at most a few iterations of the FORM-procedure (First Order Reliability Method) used for determining failure probabilities which exceed the limits. The removed boundaries are shown in Table B.1.

Table B.1 Parameter limits in the WaveImpact Kernel and their values for the standard and adapted version

Parameter limit	Parameter	Old value	New value
Min Thickness Layer 1	d1	0.10 m	0 m
Max Young's modulus	E1	25000 MPa	$1*10^6$ MPa
Min Young's modulus	E1	500 MPa	1 MPa
Max Soil Modulus	c	300 MPa	1000 MPa
Max Significant Wave Height	H_s	3 m	10 m

The limit for the thickness of the asphalt layer caused problems in cases with low characteristic values for the thickness. Due to the iterative process of FORM this caused problems with the 0.10 m limit (also for cases with a resultant thickness > 0.10 m). The same holds for the maximum value of the Young's modulus. No problems were encountered with the minimum value for the Young's modulus and the maximum value for the soil modulus, but these boundaries were removed as they might prove a potential problem in the future.
The maximum wave height of 3 meters is set due to the possible occurrence of other unknown failure mechanisms in case of such high wave heights (STOWA, 2010). However, as this limit causes problems in probabilistic calculations, it was decided, based on expert consultation, that this limit could be removed. It has been replaced by a warning in the Matlab shell. The limits will remain in the WaveImpact kernel for the end-users, in order to prevent unrealistic calculations.

B.3 Probabilistic calculations with the WaveImpact Kernel

B.3.1 Definition of boundary conditions

There is no fully probabilistic model available that gives both water levels and wave conditions. Therefore, the definition of the boundary conditions as given by Kaste & Klein Breteler (2012) is used. This model gives relations for water levels and wave conditions for three different subareas: Kust (Western Scheldt and coastal areas), Wadden (Wadden sea) and Meren (IJssel Lake). These relations are also used for the calibration of safety factors for block revetments and are shown in Appendix E. In general, the model uses a water level probability distribution (conditional Weibull) to determine the maximum water level in a storm. The development of the water level in a storm is assumed to be constant, with a fixed storm duration, a trapezoid water level development and the maximum of the trapezoid being described by the conditional Weibull distribution). The wave height and wave period are modelled as functions of the water level development in time.
The wave height is assumed dependent on the water level, which should also be the case for the wave period, otherwise this could lead to unrealistically steep waves (steeper than the breaker criterion). Therefore the wave period is calculated using a fixed wave steepness (s) of 0.05, by using the following formula:

$$T_m = \frac{T_p}{1.28} \text{ with } T_p = \sqrt{\frac{H_s}{1.56 * s}}$$

With H_S the significant wave height. The factor 1.28 to transform the peak period to the mean period. For the commonly used JONSWAP spectrum this is between 1.1 and 1.3, in assessments 1.28 is a commonly used value.

For the tidal amplitude a choice has to be made per subsystem. However, tidal amplitudes can vary along the coast. A simple sensitivity analysis however showed that the tidal amplitude has very little influence on the resulting safety factors. For the tidal amplitudes, the values from Table B.2 are used as defaults.

Table B.2 Tidal amplitudes for the different water systems

Watersystem	Average tidal amplitude ‘gga’ (m)
Meren	0
Waddenzee	1.16
Kust/Westerschelde	1.16

B.3.2 Definition of random variables

For the random input variables, the distributions are taken the same as in the study by Kanning & Den Hengst (2013). These parameters are presented in AppendixD, the used distributions for boundary conditions are presented in Appendix E.

B.3.3 Limit state function

The limit state function used in probabilistic calculations is of the form:

$Z = -\log_{10}(\mathrm{m}^* Miner)$ with m the distribution of the model uncertainty. Further details on the limit state function and its relation to the safety format used for assessments are given in Section 5.5.

B.3.4 Probabilistic calculation methods

For the probabilistic calculations in principle FORM is used, as this is the fastest method and it is also quite reliable. Kanning & Den Hengst (2013) already showed through a comparison with Monte Carlo that FORM performed quite well. The FORM method was also tested during the calibration using WaveImpact and gave the same results as Monte Carlo Importance Sampling (MCIS). However it might be that cases do not converge and FORM is not suitable. This especially holds for cases close to the valid boundaries of the model (e.g. very small/big layer thickness). In those cases MCIS (Monte Carlo with Importance Sampling) can provide a solution (normal Monte Carlo would require several days and is not considered an option). However when using MCIS, a proper sampling strategy is required for the method to be efficient. To be able to apply this on different cases a very simple strategy for Adaptive Importance Sampling was applied (Dimov et al, 2003). In this case a smaller set of calculations is executed from which a proper sampling strategy is defined. After this the real calculation is executed using an adapted sampling strategy based on the results of the initial run. In general the results of the calibration and for the real cases are stable and there is no reason to assume that FORM is converging to local minima or maxima. Figure B.1 shows a comparison of results for both MCIS and FORM for a set of cases from the calibration. The values are not equal but are close and from analysis of the results it seems that most of the deviation comes from the far from optimal sampling strategy (e.g. some cases have 15000 failures in 20000 samples, which gives a high probability of ‘missing’ a part of the failure space). However the range of results of the two methods is similar.

Hence, Monte Carlo (MC) or MCIS should only be used in case FORM does not converge. If this is the case, a full MC is accurate but time-consuming, making MCIS more attractive. However, a person with sufficient experience should perform the MCIS calculations as inaccurate sampling strategies may result in inaccurate computations.

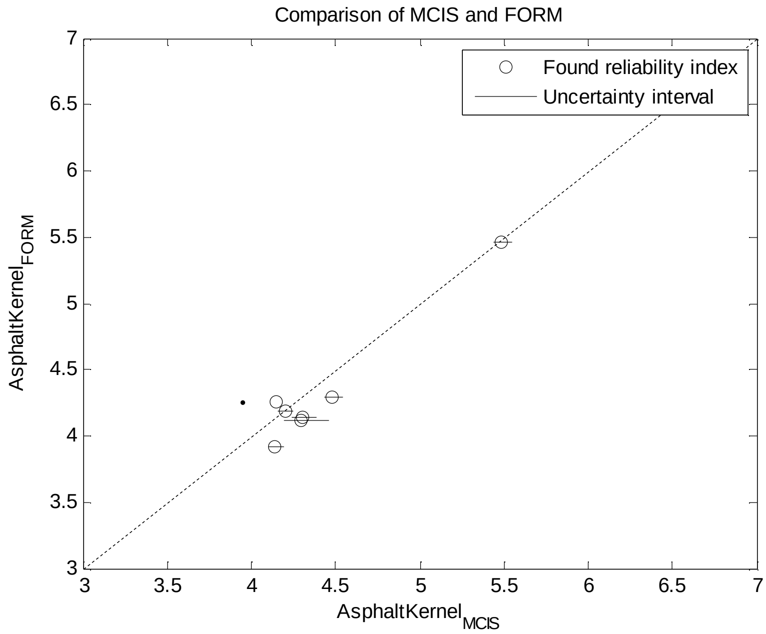

Figure B.1 Comparison of resulting reliability indices β for FORM and MCIS for a selection of cases

C KOAC-NPC assessment data of asphalt revetments

Table C.1 Deterministic input parameters of reference cases

		Lauwersmee	Negenboere	Helderse	Zuiderdiep	Balgzand	Prins Hendr	Gem polder(	Gem polder(	Gem polder	Gem polder	Gem polder	Noorden(1)	Noorden(2)	Eendracht (1	Eendracht (2	Eijerland
Input		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	TideOption	3	3	3	5	3	3	3	3	3	3	3	3	3	3	3	3
2	SwlSteps	30	30	30	30	30	50	50	50	50	50	50	50	50	50	50	50
3	GWS	0	0	0	0	0.03	0	0	0	0	0	0	0	0	0	0	0
6	DurationTide	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4
7	gga	1.16	1.57	0.7		0.7	0.7	0.73	0.73	0.73	0.73	0.73	0.75	0.75	0.78	0.78	2
12	SurchargeType	2	2	1		2	2	2	2	2	2	2	2	2	2	2	2
15	LevelForeshor	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10	-10
16	ProfileY	[-18.29,-14.24,0]	[-18.28,-16.49,-3	[0, 11.8, 16.8, 27.	[-17.84, -11.63, -5	[0, 10.65, 14.65,	[0, 9.5]'	[0, 8, 9, 12]'	[0, 8.5, 9.5, 13.5]	[0, 9, 11, 17]'	[0, 9.5, 10.5, 13]'	[0, 9.5, 11, 14]'	[0, 8.5, 10, 13]'	[0, 8, 11.5, 14.5]'	[0, 8.5, 11, 14.5]'	[0, 9, 12, 14.5]'	[0 12.5]'
17	ProfileZ	[1.73,2.14,5.99]'	[2.51,3.02,6.27,6	[2, 4.7, 5, 7.1]'	[1.11, 2.86, 4.58,	[2, 5.55, 5.75, 7]	[1.17,4.66]'	[1.67,4.49,4.72,4	[1.73,4.58,4.76,4	[1.57,4.28,4.69,4	[1.63,4.91,5.11,5.2	[1.43,4.72,5.06,5	[1.75,4.71,5.05,5	[1.7,4.25,5.12,5.2	[1.65,4.58,5.15,5	[1.6,4.99,5.27,5.	[1.42 5.5]'
30	CorrectionImp	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
31	WaterDensity	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025	1025
32	GravitationCor	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81	9.81
33	NumberSlopeP	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50	50
34	MinLevelAsph	174	2.51	2	2	2	117	167	173	157	163	143	175	17	165	16	142
35	MaxLevelAspa	5.9	6.44	7.1	7.1	7	4.66	4.9	4.92	4.99	5.27	5.29	5.23	5.26	5.34	5.45	5.5
36	NumberImpact	40	40	40	40	40	50	50	50	50	50	50	50	50	50	50	50
41	TestLevel	4.8	5.2	4.5		4.8	4.56	4.56	4.56	4.56	4.56	4.56	4.56	4.56	4.56	4.56	4.56
43	c	71	137	80	82.4	86	58.2	55.6	79.2	51.5	80.6	50.7	46.6	52.9	42.2	52.3	61.3
45	d1	0.16	0.18	0.29	0.16	0.28	0.22	0.22	0.22	0.23	0.23	0.25	0.25	0.25	0.24	0.25	0.18
47	E1	18214	12197	10676	16016	12571	7964	10575	11475	8067	6847	9205	11534	7992	7145	7370	8709
49	nu	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
50	alfa	0.42	0.5	0.63	0.25	0.3	0.5	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.23
51	beta	4.76	5.08	6.09	4.1	4.75	5.6	5.6	5.6	5.9	5.9	5.9	4.3	4.3	4.3	4.3	3.83
52	sigmaB	156	3.24	4.54	155	5.05	2.15	2.15	2.15	5.32	5.32	5.32	6.22	6.22	6.22	6.22	0.57

Table C.2 Probabilistic input parameters of reference cases

	Thickness d1		**Young's Modulus E1**		**Soil modulus c**		**Cracking strength σ_b**	
	mean	std	mean	std	mean	std	mean	std
1	204	23	6368	5927	112	50	6.40	3.01
2	226	29	8090	2240	155	12	5.85	2.48
3	330	24	7741	1636	116	24	6.24	1.00
4	185	14	8140	3921	142	34	5.35	2.22
5	312	21	9641	1701	125	24	6.20	0.62
6	no data	no data	no data	no data	86	20	5.28	1.85
7	no data	no data	9136	1714	100	42	5.28	1.85
8	241	26	7901	2692	116	30	5.28	1.85
9	258	23	5425	2158	90	39	7.21	1.05
10	259	30	5053	1231	110	29	7.21	1.05
11	282	21	7254	1387	80	19	7.21	1.05
12	276	16	7668	2508	82	33	7.63	0.83
13	273	19	6422	1115	80	22	7.63	0.83
14	277	20	5949	851	62	14	7.63	0.83
15	278	17	5385	1108	82	26	7.63	0.83
16	216	21	6868	2248	77	15	6.49	2.30

D Input Golfklap model

The table below shows the input fields as used for the old Golfklap and the new Kernel. Some of them are not used as they were only relevant for the old kernel.

Input	English term	Value	Description
1	TideOption [-]	3	1 = Overig; 2 = Oosterschelde, 3 and SurchargeType 2 = Waddenzee, 3 and SurchargeType 1 = Kust, 4 = Meren; 5 = Overig
2	SwlSteps [-]	50	Discretization of sea water level
3	GWS [m]	0	Ground water level
5	OffsetTide [h]	0	
6	DurationTide [h]	12.4	Tidal period in hours
7	Gga [m]	1.16	Tidal amplitude
8	T_free [h]	NaN	Not used in new Kernel
9	Surcharge_free [m]	NaN	Free_SWLinput
10	Start_T_free [h]	NaN	Not used in new Kernel
11	End_T_free [h]	NaN	Not used in new Kernel
12	SurchargeType [-]	2	Type of surcharge
13	WaveHeigthVector [m]	[2 1.75]	Wave height vector for deterministic calc
14	WavePeriodVector [s]	[2 4.22]	Wave period vector for deterministic calc
15	LevelForeshore [m]	-10	Level of the foreshore
16	ProfileY [m]	[0, 9.5]'	Definition of profile in Y direction
17	ProfileZ [m]	[1.17,4.66]'	Definition of profile in Z direction
18	ProfStart [-]	1	Not used in new Kernel
19	ProfEind [-]	2	Not used in new Kernel
20	LogOutputFilePath [-]	D:\temp\Test.txt'	Not used in new Kernel
21	LogAppendToFile [-]	1	Not used in new Kernel
22	LogRVW [-]	0	Not used in new Kernel
23	LogTaluddeel [-]	0	Not used in new Kernel
24	LogLaagdikte [-]	0	Not used in new Kernel
25	LogMiner [-]	0	Not used in new Kernel
26	LogInslag [-]	0	Not used in new Kernel
27	LogNMax [-]	0	Not used in new Kernel
28	LogToFile [-]	0	Not used in new Kernel
29	Berekenmodus [-]	0	Not used in new Kernel
30	CorrectionImpactFactor [-]	1	New in this sheet; to correct for berm. Default is 1
31	WaterDensity [kg/m3]	1025	Density of the water
32	GravitationConstant [m/s2]	9.81	
33	NumberSlopeParts [-]	50	Discretization of the slope

34	MinLevelAsphalt [m]	1.17	Min level at which the asphalt is considered
35	MaxLevelAspalt [m]	4.66	Max level at which the asphalt is considered
36	InslVerdAant [-]	50	Discretization of the ImpactPoints
37	ClogN [-]	0	Not used in new Kernel
38	ClogS [-]	0	Not used in new Kernel
39	TwoLayerSystem [-]	0	0 for 1Layer, 1 for 2Layer system
40	Vermoeiing [-]	0	Not used in new Kernel
41	TestLevel [m]	random variable	Design water level
42	a	NaN	parameter for old linear fatigue line
43	c [MPa]	random variable	Soil modulus
44	k	NaN	parameter for old linear fatigue line
45	d1 [m]	random variable	Layer thickness of layer 1
46	d2	NaN	Layer thickness of layer 2
47	E1 [MPa]	random variable	Young’s modulus of layer 1
48	E2	NaN	Young’s modulus of layer 2
49	nu	0.35	Poisson ratio
50	alfa [-]	determinist	fatigue line parameter
51	beta [-]	determinist	fatigue line parameter
52	sigmaB [N/mm^2]	random variable	cracking strength
53	m [-]	random variable	model uncertainty (not used in WaveImpact but in Matlab shell)

E Model for boundary conditions

Simplified load models
Because of the absence of a load model suitable for probabilistic computations with the Steentoets model, simplified load models were used (after Roskam et al., 2000). These models apply to locations in three different water systems: Vlissingen (Western Scheldt), Harlingen (Wadden Sea) and Urk (Lake IJssel). They have previously been used by Kaste and Klein Breteler (2012).

The fact that the simplified load models give inaccurate predictions of the hydraulic loads at selected locations is acceptable for the purpose of calibrating safety factors. The number of test set members is limited and the actual hydraulic loads are different for each test set member. Yet the selection of different test set members should not lead to different safety factors. The only truly important thing is therefore that the test set members cover a broad range of possible load (and strength) characteristics. And this exactly what the simplified load models do.

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 C.1 for the three selected locations.

Table E.1 Parameters of the conditional Weibull distribution (Roskam et al., 2000).

Location	**Parameters of the Conditional Weibull distribution**			
	Threshold relative to NAP (m)	Annual exceedance frequency of threshold	Shape	Scale (m)
Western Scheldt	2.900	3.907	1.040	0.2793
Wadden Sea	2.00	5.715	2.17	1.55
Lake IJssel	0.0386	7.023	0.9117	0.1137

The water level distribution is based on the HR2006. It includes 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 Western Scheldt:

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

For the Wadden Sea:

$$H_{s,\max} = 0.913 \cdot \left(0.60 \cdot h_{max} - 0.75\right)$$

For Lake IJssel:

$$H_{s,\max} = 1.115 \cdot (1.05 \cdot h_{max} - 0.23)$$

where

$H_{s,\max}$ Significant wave height at the top of the storm [m]

$h_{\max}$ Water level at the top of the storm, measured relative to NAP [m]

The corresponding exceedance frequencies of $H_{s,\max}$ are shown in Figure C.1.

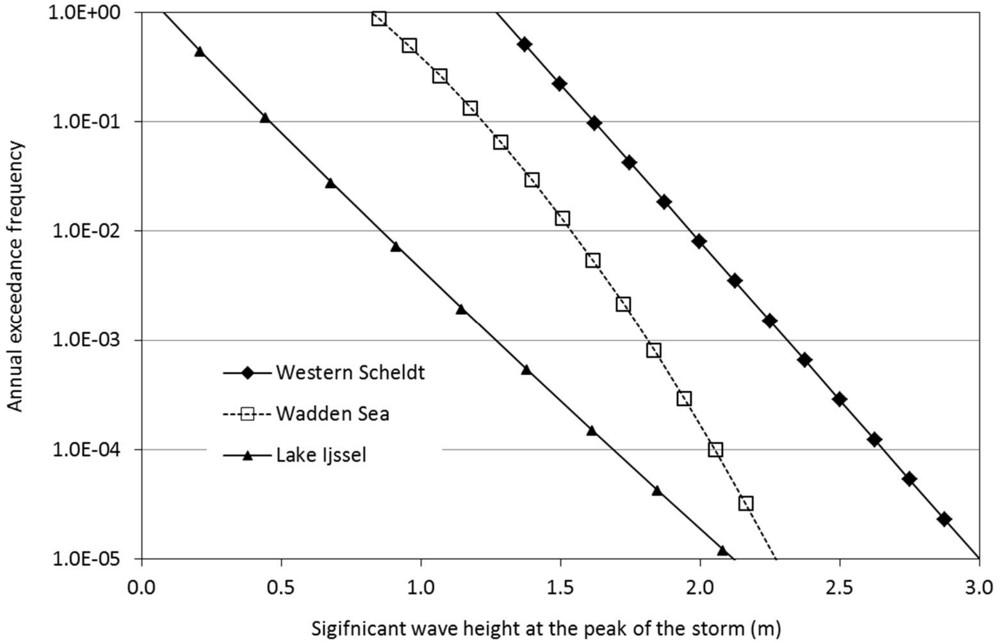

Figure E.1 Annual exceedance frequencies of the significant wave heights at the peak of the storm.

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

For the Western Scheldt:

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

For the Wadden Sea:

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

For Lake IJssel:

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

where

$H_{s,\max}$ Significant wave height at the top of the storm [m]

$h_{\max}$ Water level at the top of the storm, measured relative to NAP [m]

H_s Significant wave height at a particular moment during the storm [m]

h Water level at a particular moment during the storm, measured relative to NAP [m]

The simplified breaker criterion was used:

$$H_s \le 0.5 \cdot (h - z_{bottom})$$

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 C.2 for the Western Scheldt, in Figure C.3 for the Wadden Sea and in Figure C.4 for Lake IJssel. The lower limits on the y-axes correspond to the foreshore levels.

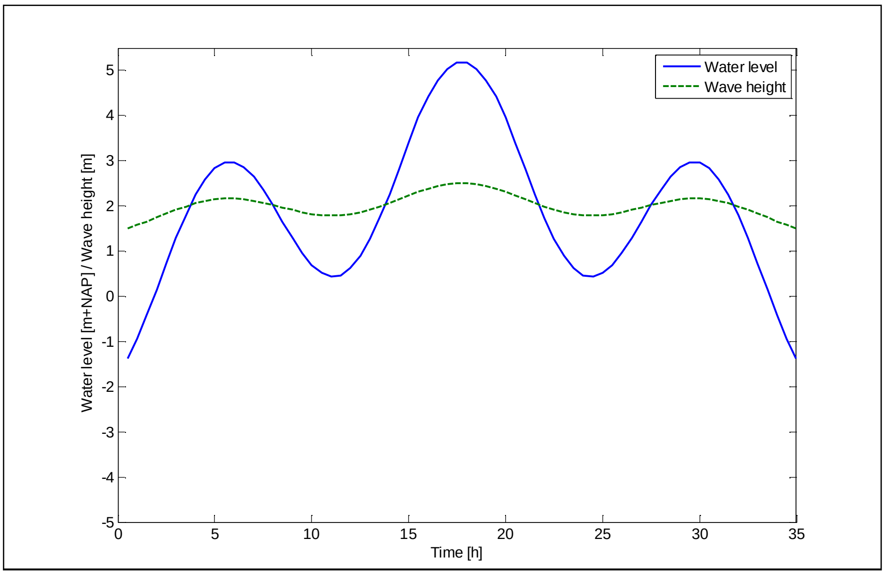

Figure E.2 Example of the development of the water level and significant wave height during a storm event in the Western Scheldt.

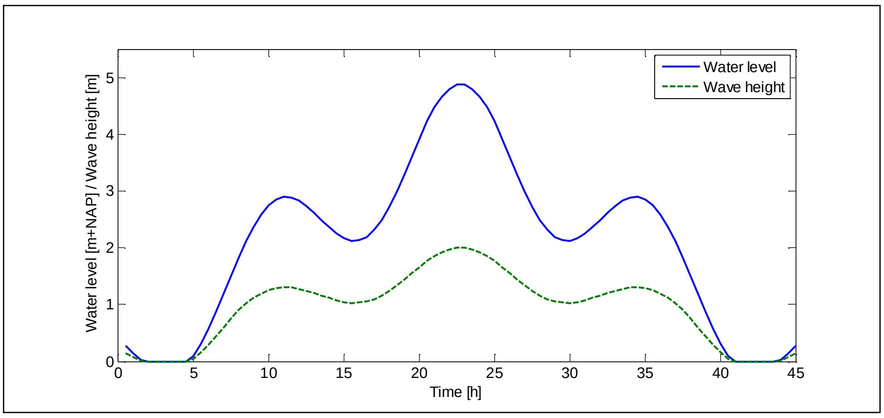

Figure E.3 Example of the development of the water level and significant wave height during a storm event in the Wadden Sea.

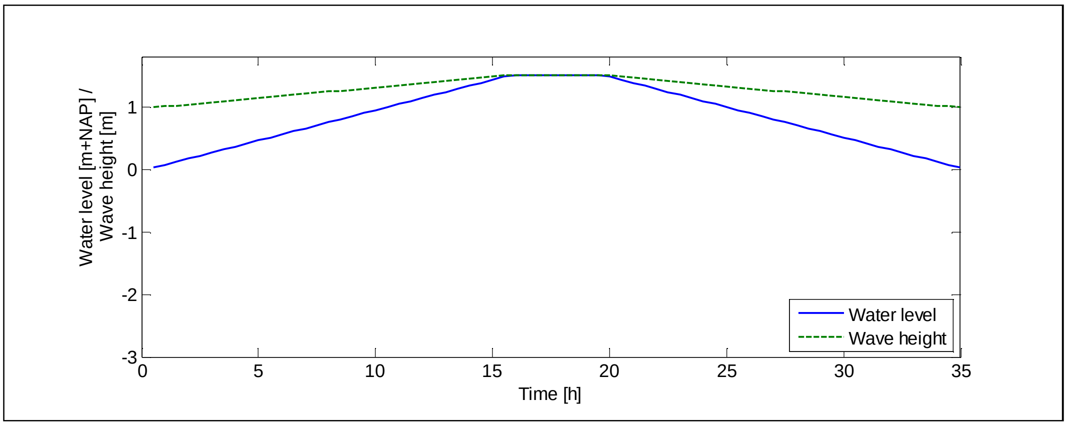

Figure E.4 Example of the development of the water level and significant wave height during a storm event in Lake IJssel.

F Fitting of β-γ relations

Method of least squares

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

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

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

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

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

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

Penalty function

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

$$R^2 = R_1^{\ 2} + R_2^{\ 2}$$

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

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

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

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

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

The factor 1000 in this function is based on visual inspection of the fitting results and provided a nice slightly conservative fit for all cases as shown in Figure F.1Figure F.1 F (see next page).

As an optimization method the standard GRG2 method as implemented in the Microsoft Excel Solver was used.

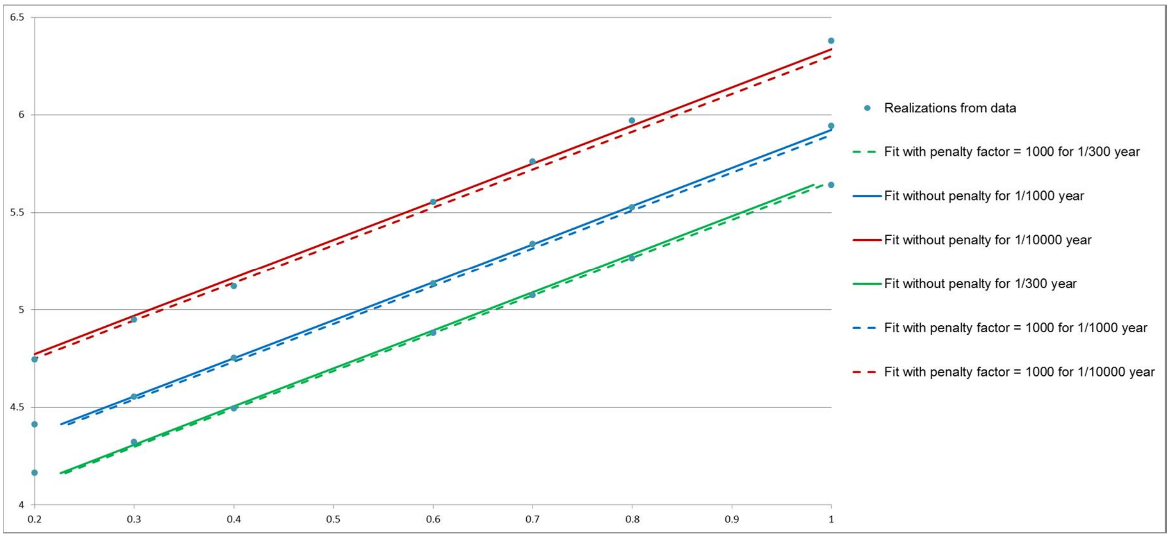

Figure F.1   Fitting of β-γ relations with and without penalty function

G Alternative safety formats: advantages & disadvantages

In this report a choice for the safety format has been made, based on expert judgment, current practice and analysis of calibration results. The choice for a safety format for a semi-probabilistic assessment consists of three parts:

- Choice of failure model.
- Choice of representative values for semi-probabilistic assessment.
- Definition of a safety rule that has to be satisfied.

For this calibration study the first two points can only be adapted at significant efforts and new computations, as these influence the calibration results. Furthermore, these follow the old assessment method exactly. The definition of the safety rule however does not influence the calibration results as long as the same output parameter (in this case the Miner sum) is considered.
The choice of a safety rule depends on the preferences of the users, therefore this paragraph aims to give an overview of possible formats with their advantages and disadvantages.

Option 1: currently suggested safety format

The currently suggested safety format is described by the following formula's
$\log_{10}(\gamma_m Miner) < -\gamma_s$ with $\gamma_s = \log_{10}(\gamma_s^*)$

Advantages:

- Both parameters are in a relatively realistic range: 1.77 for γ_m and 0.3 to 1 for $\gamma_{s.}$ although safety factors below 1 can also be perceived as counter-intuitive.
- The model uncertainty is treated as it is derived: no transformation using a logarithm is needed.
- The logarithm of Miner converges better in probabilistic calculations. Thus use of this format results in consistent formats for the limit state function used in probabilistic calculation and the semi-probabilistic assessment rule.

Disadvantages:
- γ_m and γ_s are not treated in the same way. Therefore it is more laborious to combine them to one safety factor and more difficult to assess their magnitude of influence.

Option 2: safety format without logarithm

In this safety format the logarithm is dropped resulting in the following safety format:

$\gamma_m \gamma^*_s \text{Miner} < 1$
where γ_s^* follows directly from the calibration.

Advantages:
- Very straightforward due to simple multiplication
- γ_s^* and γ_m are treated consistently

Disadvantages:

- Due to the range of Miner results the resulting safety factors will be very variable (order of magnitude between 2 and 12) and values are outside the range dike safety assessment professionals are used to.

Option 2b: safety format without logarithm and general safety factor
This safety format is an adaptation of Option 2:

$$\gamma * \text{Miner} < 1$$

where γ is the multiplication of γ_s and γ_m

Advantages:

- Even more straight-forward than Option 2.
- Single value for safety factor gives more intuitive feeling of effects of applying the safety factor.

Disadvantages:

- Due to the range of Miner results and the additional multiplication of the resulting safety factors, the final safety factor will be very variable (order of magnitude between 4 and 20) and values are outside the range dike safety assessment professionals are used to.
- Due to implicit multiplication of safety factors model uncertainty is not explicitly shown in the safety format, which might give the impression that there is no model uncertainty. Furthermore, the model factor is less straightforward to adapt in case of new research.

Option 3: safety format with both safety factors in 1 logarithm
This safety format is an adaptation of Option 1:

$$\log_{10}(Miner) < -\gamma \quad \text{with } \gamma = \log_{10}(\gamma_s^* * \gamma_m)$$

where γ_s^* and γ_m follow from the calibration.

Advantages:

- More straight-forward than Option 1, due to single safety factor. Solves the problem of inconsistently used safety factors.
- Single value for safety factor gives more intuitive feeling of effects of applying the safety factor.
- Range of safety factors is in a relatively realistic range (approximately between 0.6 and 1.3).

Disadvantages:

- Due to implicit multiplication of safety factors model uncertainty is not explicitly shown in the safety format, which gives the impression that there is no model uncertainty.
- Furthermore, the model factor is less straightforward to adapt in case of new research.

As the safety factors are applied to the Miner sum, and the variation of the Miner sum is large and non-linear, it is a disadvantage of all aforementioned safety formats that there is no intuitive relation to design parameters of the revetment. A possible solution could be to apply the safety factor to a design parameter.
Exploration for the layer thickness has shown that applying the safety factor to this value only increases the range of safety factors due to its relatively small influence on the Miner sum. A

possible solution would be to apply the safety factor to the cracking strength, but this is also not easily translated into safety in terms of revetment properties.

H Establishing the test set

Memo

To
Bernadette Wichman, Robert 't Hart

Date
7 August 2014

Number of pages
9

From
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Subject
Definition of test set for deriving safety factors for failure of asphalt revetments under wave impact

1 Introduction

For calibration safety factors for the assessment of asphalt revetments for failure under wave impact a test set is necessary. This test set should cover a wide scale of types and qualities of possible asphalt revetments. A few parameters will be dealt with as random variables and some as determinist. This memo presents an approach for setting up this test set.

2 Available data

Currently available is a spreadsheet with assessment data of most of the asphalt revetments in the Netherlands. This data is not always complete, especially mean and standard deviation of the random variables are not always available. In most cases a representative value is available. For 16 dike sections detailed assessment reports with measurement data are available.

3 Approach

It is proposed to define a test set based on the range of parameters of these 16 dike sections, if it appears from the larger dataset that certain parameters are not covered sufficiently the parameter boundaries can then be adapted accordingly.

Based on the influence coefficient obtained from probabilistic calculations 2 or 3 parameters are found to be most important. These parameters are the water level (and with it significant wave height and wave period), cracking strength (σ_b) and Youngs modulus (E1). For a number of dike sections the influence coefficients are shown in Figure 3.1.

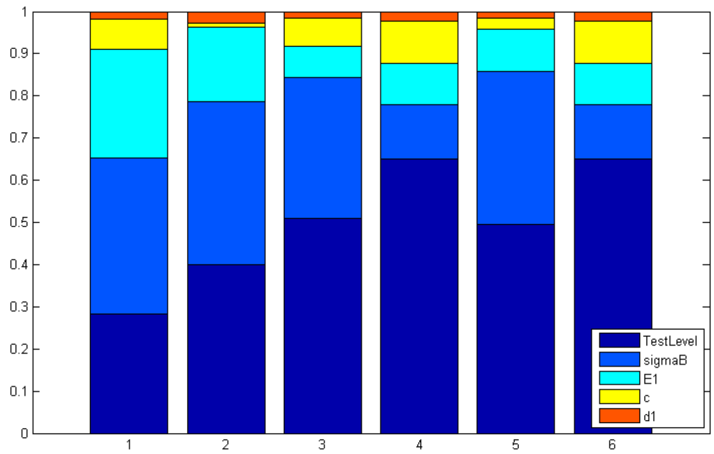

Figure 3.1 Influence coefficients for a set of cases

The influence coefficients for a test calibration run for water system “Kust”, slope ¼ and safety standard 1/10.000 show similar ranges as is shown in

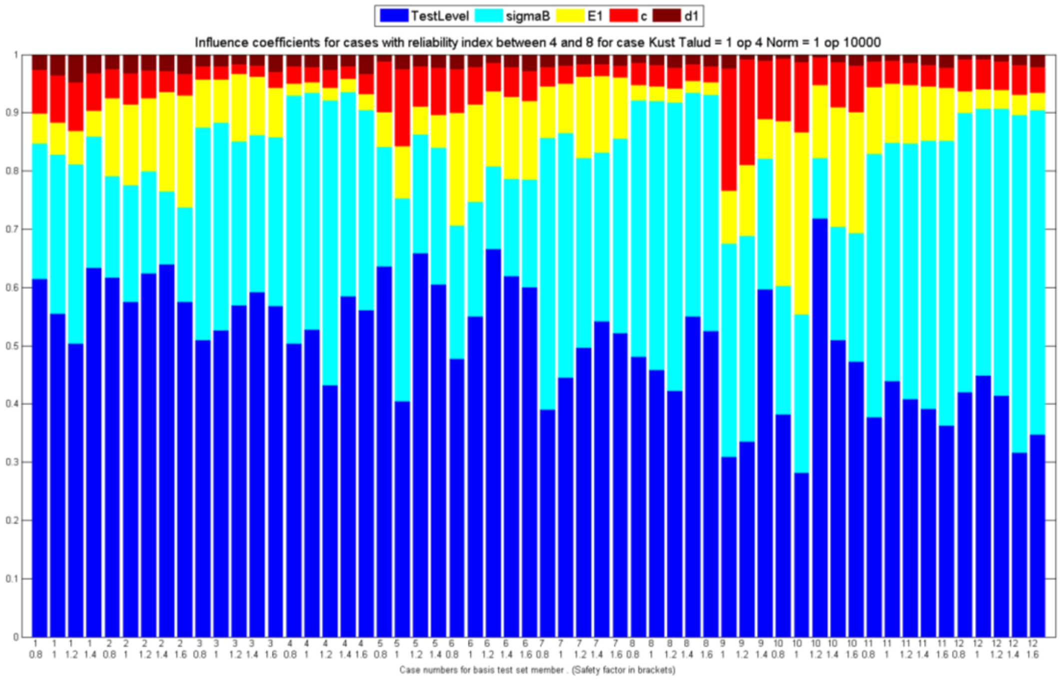

Figure 3.2 Influence coefficients for a test calibration run

It can be observed that for most cases the water level has the biggest influence, followed by the cracking strength and Young's modulus. The other parameters are generally quite unimportant.

3.1 Approach for defining cases

The first step in defining cases is to define cases for slope, water system and safety standard. This results in 3 x 3 x 3 cases:

- Different water systems (3 cases)
- Different safety standards (3 cases)
- Different slopes (3 cases)

The next step is to define parameter ranges for the different random variables. This is done based on average values and variation coefficients:

- Define lower and upper boundary and an average value for the mean value of the parameter, this results in 3 cases (bad, good, average).
- Determine the coefficient of variation of the parameter. In case there is a large variation: define 2 separate cases with for instance a high and low coefficient of variation.

4 Definition of parameters

4.1 Loads

The influence of water levels is covered by taking different water systems into account, as well as different design water levels. For the calibration conditional Weibull distributions are used for the water systems, as shown in Table XX.

Table 4.1 Weibull parameters for the boundary conditions

on	Parameters of the Conditional Weibull distribution			
	Threshold	Exceedance frequency of threshold	Shape	Scale
Western Scheldt	2.900	3.907	1.040	0.2793
Wadden Sea	2.00	5.715	2.17	1.55
Lake IJssel	0.0386	7.023	0.9117	0.1137

No calibration is done for aspahlt revetments at the Eastern Scheldt. The reason for this is twofold: there is no model for determining boundary conditions at this location and the very large majority of asphalt revetments at the Eastern Scheldt is not ‘waterbouwasfaltbeton’ but ‘open steen asfalt’, for which no calibration has to be executed.
The safety standards/design levels to be used are 1/300, 1/1000 and 1/10000. These levels are of importance for the first step in the calibration, which is the determination of the required thickness to just satisfy the safety requirement.
For the significant wave height H_s and wave period T_m a wave steepness of 0.05 is used, as in a probabilistic calculation not adapting the wave period leads to physically impossible waves.

4.2 Geometry

For the geometry of the slope the boundaries are chosen quite broad, ProfileZ = [0 10], which means a revetment of 10 meters in vertical direction. Eventually only the maximum Miner sum is of interest.
For the slope two values are used: 1/3 and 1/4. Previously also 1/6 slopes were considered, but these are dropped as it is often difficult to make a design for which Miner = 1, as mild slopes result in low wave impact. Another argument for not calculating cases with slopes 1/6 is that the design often has a thickness for which the WaveImpact model is not validated. Lastly, the slope has only a very minor influence on the resulting beta-gamma relations.
Furthermore no berm and two-layer systems are considered.

4.3 Asphalt properties

4.3.1 Approach

For random variables the calibration of safety factors for block revetments uses a method where, based on averages from data, a realistic range of mean values for the different parameters is defined. After that, based on the data, a coefficient of variation is determined which is representative for this parameter. This method can also be applied for asphalt revetments, aside from the fact that it is sometimes necessary to use 2 coefficients of variation for 1 parameter (see next paragraphs). For the averages a mean value and lower and upper boundary are calculated using a vector [0 ½ 1] in the calibration scripts (with 0 the lower boundary and 1 the upper boundary.

4.3.2 Random variables

Thickness (d1)
The thickness is determined in the design step in the calibration. For the thickness a coefficient of variation of 0.1 is used.

Soil modulus (c)

The soil modulus is assumed to be distributed lognormally with average 100 and a coefficient of variation of 0.25. Given the low influence coefficient in probabilistic calculations of the soil modulus it is not necessary to vary the coefficient of variation, as this would have a marginal effect on the failure probabilities.

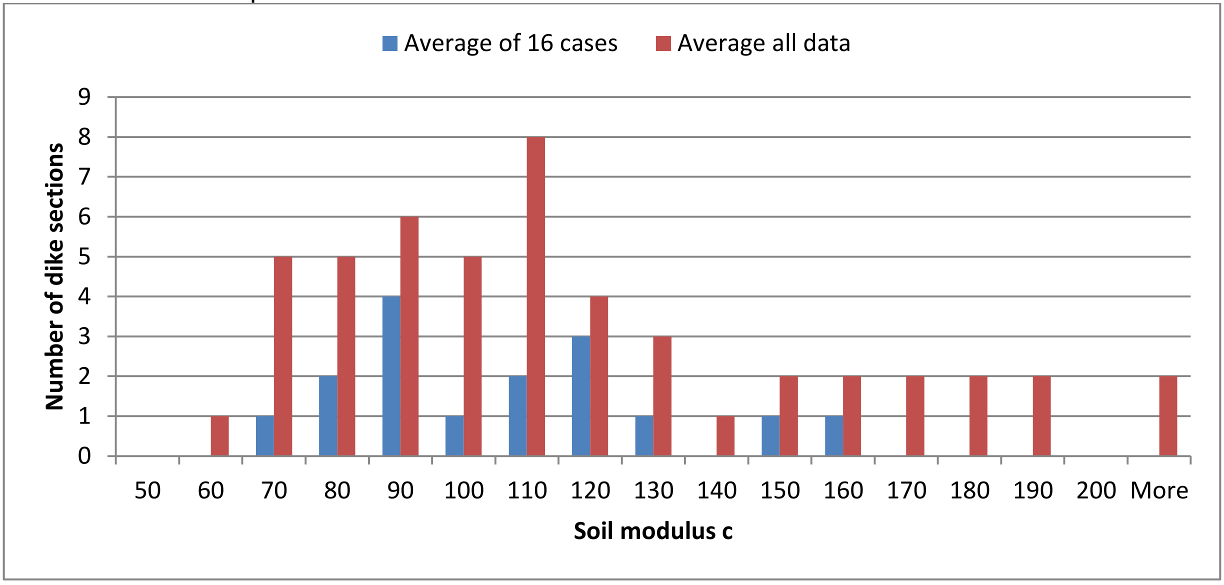

Figure 4.1 Average soil modulus values from the data set from KOAC-NPC

Youngs modulus (E1)

The Youngs modulus is an important parameter for the asphalt strength. The histogram in Figure 4.2 shows that the set of 16 cases is a good representation of the possible range, but there is no extremely high value like in the dataset for all the cases. The extreme value in the large data set is found at the Hondsbossche and Pettemer sea dike, which is currently under reconstruction. Taking this case into account is possible but it would have a large influence on the dataset, which might cause the set of cases to be an improper representation of the possible cases. A possible solution might be to do an advanced probabilistic assessment for cases with an extremely high Youngs modulus.

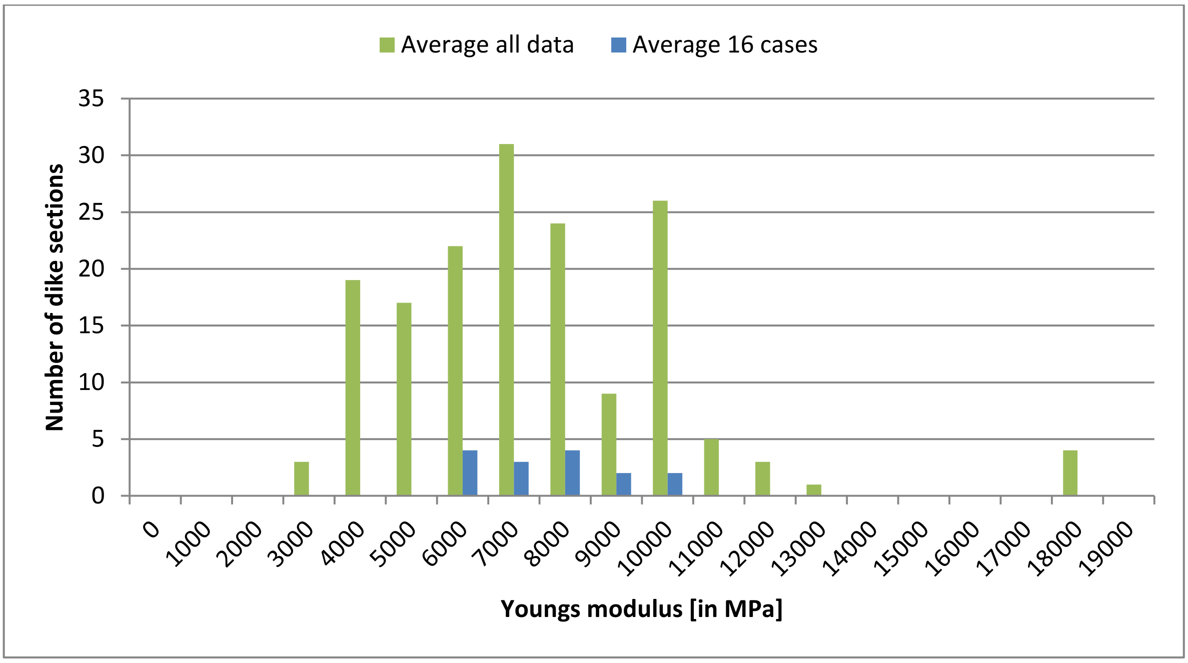

Figure 4.2 *Average Young's modulus values from the KOAC-NPC dataset*

For the Young's modulus the mean values are taken in a range between 4000 and 10000. Coefficients of variation of 0.2 and 0.4 are assumed, resulting in two cases: one with a large variation and one with a smaller variation. This covers the observed range of characteristic values, only the extreme cases with a representative value of >20000 MPa are not taken into account in the test set.

Cracking strength (σ_b)

Rewrite after discussion with Robert

Based on α-values from the test calibration and test cases the cracking strength is the most important parameter. The average cracking strength varies between 5.0 and 7.6 N/mm^2 with a few exceptions where values of 3.8 N/mm^2 are found. These exceptions however are dike sections with 44 year old asphalt which is most likely disapproved during the last assessment(van Pallandt en Martina Corneliadijk). As the safety factor is valid for revetments which could be approved in an assessment the data from these sections should not be in the dataset. It is also not sensible to take these cases into account, as they would increase safety factors for other revetments. Based on the other data the coefficient of variation varies between 0.1 and 0.5, therefore the approach used for the Youngs modulus also seems a good solution here. Therefore the average is assumed to be varying between 5 and 7.5 N/mm^2 and two separate cases for old (large variation) and young (small variation) are defined. The first with a coefficient of variation of 0.35 and the latter with a coefficient of variation of 0.2.
As the influence of the coefficient of variation of the cracking strength seems to be quite large (based on the test calibration), this leads to separate safety factors for young and old asphalt.

4.3.3 Other properties

Poisson ratio (v)
Fixed at 0.35

Fatigue parameters α and β
The fatigue parameters α and β have a considerable influence on the Miner sum. α varies between 0.23 and 0.63, an increase of α from 0.23 to 0.63 leads to an increase of the Miner sum by a factor 10. β varies between 3,8 and 7, resulting in a factor 20 difference in Miner sum for both cases. These parameters have a considerable influence on the final result. Based on these fatigue relations the parameters are coupled to values for $\sigma_{b.}$ α and β are not random variables, but uncertainty of these parameters is accounted for in $\sigma_{b.}$ To determine standard values for α and β the assessment line as suggested by (Opstellen nieuwe ontwerp- en toetsgrafiekenKOAC@) is used.

Model uncertainty factor (m)
The model uncertainty factor covers uncertainties in the model schematization of WaveImpact. It is provided by Cluster 5. It covers a set of uncertainties which could be quantified with the given time and money. This results in a model uncertainty factor between 0.809 and 3.27. In the probabilistic calculation this is covered by a lognormal distribution with mu = 1.77 and sigma = 0.784.

5 Test cases

It is proposed to use the following cases for the calibration.
Base cases for water systems:

Location	Parameters of the Conditional Weibull distribution			
	Threshold	Exceedance frequency of threshold	Shape	Scale
Western Scheldt	2.900	3.907	1.040	0.2793
Wadden Sea	2.00	5.715	2.17	1.55
Lake IJssel	0.0386	7.023	0.9117	0.1137

<u>3 design standards:</u>
- 1/300
- 1/1000
- 1/10000

<u>2 slope angles[1]:</u>
- 1/3
- 1/4

[1] Initially also a 1/6 slope was taken into account. Due to the low wave impact caused by the mild slope, it was very difficult to find realistic values for the thickness in the design step. As the slope has only a very small influence on the beta-gamma relation this case is dropped.

For the random variables various cases are used. A range is defined for which for lower and upper boundary as well as the median a case is defined. For all stochasts the distributions are transformed to lognormal distributions.

For the Youngs modulus
2x3 cases

Case	**Mean**			**Coefficient of variation**
	Lower boundary	Upper boundary	Median	
E_CoVHigh	4000	10000	7000	0.2
E_CoVLow	4000	10000	7000	0.4

For the cracking strength
2x3 cases

Case	**Mean**			**Coefficient of variation**
	Lower boundary	Upper boundary	Median	
σ_b_old	5.0	7.6	6.3	0.2
σ_b_young	5.0	7.6	6.3	0.35

For the cracking strength a distinction is made between the cases with high and low coefficient of variation. This leads to separate safety factors for 'young'(small CoV) and 'old'(asphalt).

For the random variables, for the thickness and soil modulus mean values with a standard coefficient of variation are assumed. *v, α* and *β* are assumed deterministic, with *α* and *β* based on the assessment fatigue line as defined in @Ref

Stochast	Distribution type	Mean	Coefficient of variation
c	L	100	0.25
d1	L	Variable	0.1
v	D	0.35	-
α	D	0.5	-
β	D	4.8	-

The cases are defined in the following way, for example for old asphalt at the Wadden sea with a slope of 1/3 the following would be used:

Case:	**mean E**	**CoV E**	**mean sigmaB**	**CoV sigmaB**
1	10000	0.2	5.0	0.2
2	10000	0.4	7.6	0.2
3	7000	0.2	6.3	0.2
4	7000	0.4	6.3	0.2
5	4000	0.2	5.0	0.2
6	4000	0.4	7.6	0.2

For young asphalt this would be the same except that the CoV for sigmaB is 0.35.
The same holds for all cases for all water systems and all slopes.

I Dealing with asphalt length-effects in WTI2017

Memo

Aan
Mark Klein Breteler, Ruben Jongejan, Wouter Jan klerk, PG-Asfalt

Datum	**Aantal pagina's**	
7 augustus 2014	4	
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Onderwerp
Omgaan met lengte-effecten in WTI2017

Inleiding

Voor het WTI2017 moet er rekening worden gehouden met de variatie van eigenschappen van de bekleding. Hoe langer een dijkvak des te extremer waarden mogen worden verwacht op d zwakste plek. Hier moet op de één of andere wijze rekening mee worden gehouden bij de beoordeling van asfaltbekledingen op golfklappen.

In de toetsmethode volgens het VTV2006 werd de veiligheid op impliciete wijze gerealiseerd door een deterministische berekening uit te voeren zonder een modelfactor te gebruiken (in feite dus γ_m = 1,0) en door voor alle relevante parameters die onzekerheid kennen veilige waarden te hanteren op basis van de gemeten parameterwaarden. Dat wil zeggen dat uitgegaan werd van bijvoorbeeld voor de sterkte een lage waarde (5% onderschrijdingswaarde voor de sterkte), voor de vermoeiing een veilige vermoeiingskarakteristiek en voor de asfaltstijfheid een hoge waarde (95% overschrijdingswaarde voor de sterkte). Die hoge waarde voor de stijfheid is een waarde aan de veilige kant, omdat een hoge asfaltstijfheid tot relatief hoge spanningen in het asfalt leidt, waardoor bij een hoge stijfheid er eerder sprake zal zijn van bezwijken van het asfalt.
Nul optie voor WTI2017 is de methodiek als bij de VTV2006 handhaven.

In kader van WTI2017 wordt echter een semi-probabilistische aanpak (niveau I) nagestreefd, waarbij de veiligheidscoëfficiënt is afgeregeld met prob-sommen op niveau II of hoger.
Een punt van discussie is echter hoe om te gaan met de ruimtelijke spreiding van de materiaalparameters.

Analyse
Uit onderzoek, veldmetingen, blijkt dat materiaaleigenschappen op zeer korte afstand kunnen variëren. Zelfs boorkernen die direct naast elkaar uit de bekleding zijn genomen, kunnen extreme verschillen in materiaalsterkte laten zien. En de rand van een boorkern toont op de onderrand veelal al een verloop in asfaltdikte. Oftewel het lijkt erop dat veel constructie-eigenschappen in het veld gekenmerkt kunnen worden met een zeer korte correlatielengte.

Voor de beoordeling gebruiken we een rekenmodel wat in feite het gedrag beschrijft van een strook asfalt (tegen het talud op) die wordt belast door inkomende golven. De breedte van de strook waarvoor de berekening wordt uitgevoerd is ter discussie gesteld. Deze breedte in relatie tot de vaklengte bepaalt namelijk hoe rekening te houden met de variatie van de constructie-eigenschappen.

In het verleden (VTV2006) werd één berekening per dijkvak uitgevoerd. Men zou dus kunnen veronderstellen dat de strook van het rekenmodel de breedte heeft van de dijkvaklengte. Dat betrof echter de deterministische berekening met voor alle parameters een waarde aan de veilige kant. Als we nu in de probabilistische referentiesommen voor alle stochastische parameters de statistisch kenmerkende grootheden opgeven (verwachtingswaarde en spreiding) dan zal, als de strookbreedte gelijk wordt genomen aan de dijkvaklengte, er geen rekening (meer) worden gehouden met een lengte-effect: met de variatie in materiaaleigenschappen langs de dijkas. De afregeling van de semi-probabilistische berekening zal dan dus, afhankelijk van de vaklengte, aan de veilige (kort dijkvak) of juist onveilige kant (lang dijkvak) zijn.

Aan de andere kan is geconstateerd dat in het veld een constructie-parameter elke 10 cm een andere waarde kan hebben. Dat suggereert dat de verwachtingswaarde voor de laagste parameterwaarde voor een bekleding gelijk is aan de waarde die hoort bij een kans 1/(10*vaklengte), waarbij de vaklengte gegeven is in meters. Aangezien de vaklengte bij asfaltbekledingen meestal in de orde van één tot enkele km's is, zou de laagste parameterwaarde dus volgen uit een kans van orde 1/10.000: een extreem lage waarde.
Het constructiegedrag wat door het rekenmodel wordt beschreven, zal door ruimtelijke middeling niet allesoverheersend bepaald worden door een enkel slecht stukje met een lengte van 10 cm. Daarom moet voor de probabilistische referentie-berekeningen geen rekening worden gehouden met een correlatieschaal in de orde van 10 cm.

De vraag is welke orde van grootte er wel een correct beeld oplevert van de veiligheid. Daarvoor moet naar het gedrag van de constructie en de belasting worden gekeken. Asfaltbekledingen zijn plaatbekledingen die ca. 0,2 tot 0,3 m dik zijn. Deze bekleding rust in principe op een zandbed. De maatgevende belasting wordt gevormd door golfklappen, die worden geschematiseerd tot drukverdelingen op het bekledingsoppervlak, waarbij de lengte (gemeten in de richting van de dijkas) van de drukverdeling groot is, orde 10 m, en waarbij de breedte (gemeten in de richting tegen het talud op) in de orde van de golfhoogte is. Dat heeft als consequentie dat vervormingen in de lengterichting (van de dijk) min of meer dezelfde zullen zijn. De gehanteerde modellering in GOLFKLAP is danook een plane-strain-modellering van een strook bekleding tegen het talud op.

Een plane strain-model, terwijl er in werkelijkheid in de richting in de richting van de dijkas wel stijfheidsverschillen op kleine schaal (10 cm) zijn, maakt dat de effectieve stijfheid waarmee moet worden gerekend in feite een over enige afstand gemiddelde stijfheid moet zijn. Dat is één van de redenen waarom voor de beoordeling de asfaltstijfheid niet wordt ontleend aan de kleine proefstukken (50*50*230mm^3) aan de hand waarvan de sterkte wordt bepaald, maar aan de VGD-metingen. Bij die metingen wordt immers de constructie als geheel aangesproken en daardoor een over een groter oppervlak gemiddelde stijfheidswaarde gevonden. Al is in [2009] al eens geconstateerd dat proefresultaten een dergelijke redenering niet per se lijken te bevestigen.

De **(breuk)sterkte** wordt echter wel aan de hand van de kleine proefstukken bepaald. Dat maakt dat voor de breuksterkte er wellicht ook een uitmiddeling moet plaatsvinden. Een simpele verkenning op basis van theoretisch gedrag heeft in [2009] geleerd dat de plaatwerking niet leidt tot een sterkte die het gemiddelde is van de gemeten breuksterkten, maar tot een zeer lage waarde voor de effectieve sterkte: μ-1,5σ à μ-2σ. Dit resultaat is niet afhankelijk van toevallig erg slechte segmentjes in de bekleding. De zwakste elementjes bezwijken namelijk toch wel voordat de maximale sterkte van de plaat is bereikt. Daarom kan die effectieve sterkte worden aangehouden voor het gehele dijkvak.

Gezien het grofstoffelijk karakter van de verkenning [2009] en de geconstateerde onzekerheid in de aannamen wordt voorgesteld om voor de effectieve breuksterkte in de probabilistische referentie-berekeningen uit te gaan van een verwachtingswaarde op basis van de aan proefstukken gemeten sterktes: $\mu = \mu_{metingen}-1{,}75\sigma_{metingen}$; en een spreiding $\sigma = 0{,}15\ \sigma_{metingen}$.
Voor de semi-probabilistische berekening wordt voorgesteld om de werkwijze te handhaven die ook voor het VTV2006 werd gebruikt: rekenen met de 5%-onderschrijdingswaarde van de breuksterkte op basis van de gemeten waarden. In beide gevallen heeft de geadviseerde sterkte dus betrekking op het gehele dijkvak.
Voorgesteld wordt om de **vermoeiingslijn** op de gebruikelijke manier vast te stellen, gekoppeld aan de voornoemde breuksterkte.

Voor de **stijfheid van het asfalt** wordt geadviseerd uit te gaan van de verdeling van de met de VGD-apparatuur gemeten waarden. Op grond van eerder uitgevoerd heterogeniteitsonderzoek lijkt het realistisch om de gemeten waarden representatief te stellen voor stroken van orde 10 m breedte.
Het lijkt echter niet reëel om van deze 10 meter uit te gaan voor het in rekening brengen van het lengte-effect. De spreiding in de meetwaarden moet namelijk waarschijnlijk voor een deel worden toegeschreven aan toevallige onvolkomenheden in de bekleding in de nabijheid van een meetpunt. Te denken valt aan een scheur of naad in de bekleding nabij het punt waar het valgewicht op neerkomt. Een dergelijke onvolkomenheid zal leiden tot grotere doorbuigingen van de plaat en dus tot een relatief lage bijbehorende asfaltstijfheid. Nb. dit is wellicht ook de verklaring voor de in [2009] gegeven constatering dat VGD-metingen niet per se kleinere spreidingen in stijfheid laten zien dan de metingen aan kleinere proefstukken. Alhoewel dit kwantitatief vast niet helemaal correct is, wordt bij gebrek aan betere gegevens verondersteld dat de extra spreiding die in VGD-metingen aanwezig is, juist voldoende is om het lengte-effect voor dijkvakken van één km lengte af te dekken. Dat betekent dat als er sprake is van duidelijk afwijkende vaklengtes hier wel rekening mee moet worden gehouden. Als de afsluitdijk als één dijkvak wordt beoordeeld, dan levert dat dus bijvoorbeeld 30 onafhankelijke strekkingen. Omgekeerd krijg je bij korte dijkvakken van bijvoorbeeld 500 m een “positief” lengte-effect.

De **stijfheid van de ondergrond** wordt eveneens afgeleid uit VGD-metingen. Op grond van de wijze van aanleg van de bekleding (verdichting), kan worden gesteld dat orde 10 m een redelijke maat lijkt voor de correlatie-lengte. Geadviseerd wordt om die lengte te gebruiken voor de probabilistische set referentiesommen.

De **asfaltdikte** varieert op korte afstanden, zoals blijkt uit de onderzijde van boorkernen die worden genomen. Doordat de dikte wordt gemeten in een raai met radar zijn met betrekking tot het verloop van deze dikte ook relatief veel gegevens beschikbaar. Vanuit de fysica van de buigende plaat wordt ingeschat dat rekenen met een over ca. 5 meter gemiddelde asfaltdikte leidt tot een reëel inzicht in de spanningen en dus de veiligheid.
Daarom wordt geadviseerd om de gemeten asfaltdiktes over 5 m te middelen en de kansverdeling (μ en σ) van deze waarden te gebruiken als input voor de rekenmodellen.
Nb. het effect van lokaal dunnere bekledingen die specifiek bij overgangen van dun naar dik aanleiding kunnen zijn tot lokaal hogere spanningen, is een effect wat in de modelfactor moet zijn verrekend.

Ten besluit
De variatie van de materiaaleigenschappen langs de strook tegen het talud op is buiten beschouwing gelaten. Zuiver statistisch gezien zou, als die variatie bekend was, hier nog rekening mee moeten worden gehouden. In de praktijk worden de materiaaleigenschappen voor een beoordeling echter verzameld op een niveau wat vrijwel zeker maatgevend is. Laagdikten, sterkte en stijfheid worden namelijk bepaald langs een raai op een niet al te grote afstand langs de onderrand van de bekleding. Daarbij moet rekening worden gehouden met een eventueel aanwezige asfaltwig, die nogal eens aan de onderrand bij wijze van overgangsconstructie is aangebracht. Nabij de onderrand van de bekleding zal de eventuele aantasting van het materiaal (vocht) over het algemeen het grootst zijn en de belasting het meest intensief. Door de berekening te baseren op de eigenschappen van de bekleding in die zone, is de fout die gemaakt wordt door geen rekening te houden met de variatie van eigenschappen in de strook tegen het talud op, gering.

Het advies ten aanzien van de te hanteren lengtes die in deze memo is gegeven, is gebaseerd op min of meer theoretische redeneringen in combinatie met wat pragmatische benaderingen. Uiteindelijk zal moeten worden nagegaan of deze aanbeveling in combinatie met andere relevante elementen in het bepalen van de veiligheid niet leiden tot onmogelijk strenge of juist te geringe eisen aan de veiligheid van asfaltbekledingen. Te strenge eisen kunnen leiden tot onnodig afkeuren; te geringe eisen leidt tot onterecht goedkeuren. Als referentie hierbij dient te worden gekeken naar de praktijk tot op heden. Opgepast moet worden dat bij die vergelijking van de uitkomsten van een nieuwe met de oude beoordelingsmethodiek wel hetzelfde veiligheidsniveau wordt geëist, omdat anders de vergelijking mank gaat.

Literatuur
[2009] R 't Hart: *Verkenning effect van inhomogeniteit in asfalteigenschappen op kleine schaal* Deltares, 17 juli 2009.
(toegevoegd als bijlage)

Kopie aan
Ana Teixeira;DELTARES

Memo

Aan
Projectgroep asfalt

Datum
17 juli 2009

Aantal pagina's
5

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Onderwerp
Verkenning effect van inhomogeniteit in asfalteigenschappen op kleine schaal

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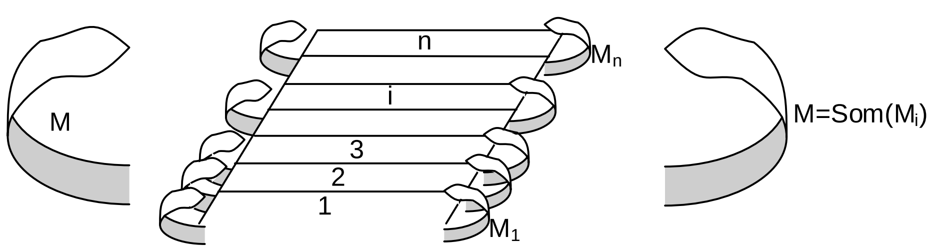
Bekend is dat asfalt niet homogeen is, maar variatie vertoont in eigenschappen. Daarom is het van belang hier rekening mee te houden bij de beoordeling van de veiligheid van asfaltbekledingen op waterkeringen. Momenteel gebeurt dit door te rekenen met karakteristieke waarden die op basis van materiaalproeven zijn bepaald. Dit gaat echter voorbij aan de wijze waarop het materiaal is toegepast in de constructie. Daarom leidt deze aanpak waarschijnlijk tot een zeer conservatief resultaat. Het verdient daarom aanbeveling om middels een verkenning een orde van grootte duidelijk te krijgen in hoeverre het rekenen met karakteristieke waarden conservatief is.

Het gaat met name om de asfaltstijfheid en de breuksterkte. Beide grootheden worden verondersteld normaal verdeeld te zijn met een verwachtingswaarde en een spreiding.

Stijfheid
In geval van buiging van een plaat kan de stijfheid van de plaat worden afgeleid uit de stijfheid van de afzonderlijke onderdelen waaruit de plaat is opgebouwd. Immers de variatie in asfaltstijfheid kan worden geschematiseerd door aan te nemen dat de plaat bestaat uit strookjes met ieder een eigen stijfheid die ongecorreleerd is met die van de buurman. Hoe breed die strookjes zijn om rekening te houden met de feitelijk aanwezige correlatielengte dient nader te worden vastgesteld aan de hand van proeven.
In eerste instantie wordt aangenomen dat de balkjes met de gebruikelijke breedte (5 cm) ongecorreleerde waarnemingen opleveren. Daarnaast wordt nagegaan in hoeverre een breedte van 25 cm tot wezenlijk andere uitkomsten leidt.



Voor de uitwerking moet ook een schatting worden gemaakt met betrekking tot de breedte waarover er sprake is van uitmiddeling. In eerste instantie wordt verondersteld dat er sprake is van uitmiddeling over een breedte die overeenkomt met de hoogte van de significante golfbelasting op de bekleding. Aannemende een reële belasting van H_s = 2,0 m dient dus te

worden gemiddeld over 2 m, hetgeen bij een stookbreedte van 0,5 m leidt tot 40 onafhankelijke stijfheden (n = 40). Voor een strookbreedte van 0,25m: n = 8.
(Voor een iets meer gedetailleerde uitwerking is het beter een schatting te baseren op de karakteristieke lengte van de bekleding.)
Als de stijfheid van de onderdelen wordt gegeven door de verdeling $\{\mu_E, \sigma_E\}$, dan resulteert dit voor de aannamen in een plaatstijfheid met een verdeling: $\{\mu_E, \sigma_E/\sqrt{n}\}$.
Een veilige, karakteristieke waarde kan dus worden berekend uitgaande van een (veel) lagere spreiding dan de spreiding in de proefresultaten.
De voorafgaande redenering is een theoretische redenering gebaseerd op simpele statistische en mechanica-principes. In de praktijk van het bepalen van materiaalparameters met buigproeven op balkjes en VGD-metingen op de constructie spelen er foutenbronnen een rol die in deze theoretische redenering niet voorkomen. Zo is de spreiding in VGD-stijfheden groot ten opzichte van de spreiding in buigproefresultaten, hetgeen in tegenspraak is met het feit dat een met de VGD bepaalde stijfheid al een gemiddelde zou moeten.

Breuksterkte
Voor de breuksterkte wordt verondersteld dat de middeling over eenzelfde breedte plaatsvindt als voor de stijfheid. Echter de werkwijze zelf is wel een geheel andere. Het gedrag van asfalt bij breuk is namelijk niet per se een taaie breuk en niet alle onderdelen zullen op exact hetzelfde moment (bij dezelfde vervorming) breken. De vraag is nu, bezwijkt het geheel als het eerste element bezwijkt, of is de spreiding in de breuksterkte zodanig groot dat er één of enkele elementen kunnen bezwijken alvorens de samenstellende plaat bezwijkt?
Dit wordt uitgewerkt door bros gedrag te veronderstellen en een homogene stijfheid. (Die homogene stijfheid is in praktijk niet aanwezig, maar voor dit gedachten-experiment levert het wel een case die uit te werken is.)
Voor de breukspanning wordt ook weer een normale verdeling aangenomen: $\{\mu_{Br}, \sigma_{Br}\}$.
Het breukmoment voor de plaat dient te worden bepaald uit de breukmomenten van de samenstellen balkjes. Daartoe worden de n breukmomenten M_i op oplopende volgorde van grootte gesorteerd en het breukmoment voor de plaat wordt dan gegeven door:
$M_{Breuk} = \max\{(n-(i-1))M_i\}$ voor i = 1 t/m n.
Bij de vervorming waarbij het zwakste element breekt is het moment namelijk: $n.M_1$;
Als het tweede element bezwijkt is het totale moment: $(n-1).M_2$, etc.
Het maximum van die reeks M_i-waarden kan worden aangemerkt als het bezwijkmoment voor de plaat.

Om na te gaan in hoeverre het reëel is te veronderstellen dat een element bezwijkt, terwijl het geheel dan nog niet bezwijkt, is er een case numeriek geëvalueerd.
Allereerst dient te worden bepaald wat de (verwachtings)waarden zijn voor de gesorteerde trekkingen. Daartoe wordt aangenomen dat de trekkingen keurig gelijkmatig over de totale kansruimte zijn verdeeld.
Als er slechts één trekking wordt gedaan voor de breuksterkte, dan is de verwachtingswaarde voor deze trekking gelijk aan de verwachtingswaarde voor de verdeling van breuksterktes:
1 trekking: overschrijdingskans = 0,5: $\mu_{Br} + 0.\sigma_{Br}$
Indien er twee trekkingen worden gedaan, dan vormen de volgende waarden de beste schatting:
2 trekkingen: grootste waarde heeft een overschrijdingskans = 0,33: $\mu_{Br} + 0{,}43.\sigma_{Br}$
de kleinste waarde heeft een overschrijdingskans = 0,67: $\mu_{Br} - 0{,}43.\sigma_{Br}$
Indien er drie trekkingen worden gedaan, dan vormen de volgende waarden de beste schatting:
3 trekkingen: grootste overschrijdingskans = 0,25: $\mu_{Br} + 0{,}68.\sigma_{Br}$

middelste, overschrijdingskans = 0,50: $\mu_{Br} + 0.\sigma_{Br}$
kleinste, overschrijdingskans = 0,75: $\mu_{Br} - 0{,}68.\sigma_{Br}$

Indien er n trekkingen worden gedaan, dan vormen de volgende waarden de beste schatting:
n trekkingen: overschrijdingskans: 1/(n+1) levert grootste waarde;
……..
overschrijdingskans: 1-1/(n+1) levert kleinste waarde.

Voor n = 40 leidt dit tot een overschrijdingskans = 1-1/(40+1) = 0,9756: $\mu_{Br} – 1{,}97.\sigma_{Br}$
Aldus wordt $M_1 = 40 (\mu_{Br} – 1{,}97.\sigma_{Br})$.
De eerstvolgende waarde voor de breuksterkte hoort bij een kans: 1-2/(40+1) = 0,9512, hetgeen leidt tot $\mu_{Br} – 1{,}66.\sigma_B$. Het plaatbreukmoment vlak voor bezwijken van het tweede element is dus: $M_2 = 39 (\mu_{Br} – 1{,}66.\sigma_{Br})$.
De eerstvolgende waarde voor de breuksterkte hoort bij een kans: 1-3/(40+1) = 0,9268, hetgeen leidt tot $\mu_{Br} – 1{,}45.\sigma_B$. Het plaatbreukmoment vlak voor bezwijken van het tweede element is dus: $M_3 = 38 (\mu_{Br} – 1{,}45.\sigma_{Br})$.
Evenzo: $M_4 = 37 (\mu_{Br} – 1{,}30.\sigma_{Br})$; $M_5 = 36 (\mu_{Br} – 1{,}16.\sigma_{Br})$.

Uit gelijkstellen van M_1 aan M_2 kan worden bepaald dat als $\sigma_{Br} < 1/14\ \mu_{Br}$ dat dan de laagste getrokken waarde van de breuksterkte bepalend is voor plaat als geheel. De representatieve breuksterkte is dan dus grofweg $(\mu_{Br} – 2.\sigma_{Br})$
Als de spreiding in de breuksterkte groter is, dan wordt de situatie iets gunstiger, maar per saldo niet erg veel beter zoals uit volgend voorbeeld blijkt. Stel n = 40; $\sigma_{Br} = 1/5\ \mu_{Br}$.

i	M_i/μ_{Br}
1	24/40
2	26/40
3	27/40
4	27,4/40
5	27,6/40

Het blijkt dat er dan al enkele elementen kunnen bezwijken, maar dat dit per saldo niet tot een erg veel hoger plaatbreukmoment leidt: $\mu_{Br} – 1{,}55.\sigma_{Br}$ i.p.v. $(\mu_{Br} – 2.\sigma_{Br})$.
Als er sprake is van middeling over veel minder onafhankelijke deelgebieden, dan veranderen de getallen iets, maar de tendensen blijven dezelfde, zie bijlage voor n = 8.

In deze redenering is keurig uitgegaan van “verwachtingswaarden” voor de verschillende trekkingen uit de sterkteverdeling. Wat is echter het effect van een toevallig lokaal veel slechtere breuksterkte dan volgens de “verwachting”? Een dijk is namelijk veel langer dan de breedte waarover mag worden gemiddeld en er mag dus worden verwacht dat in die gehele breedte zich een nog veel slechter segmentje zal bevinden dan wat op basis van een 40-voudige trekking mag worden verwacht. Als de spreiding in verhouding tot de verwachtingswaarde maar groot genoeg is, dan zal dat element als eerste mogen bezwijken, zonder dat het tot bezwijken van de plaat aanleiding geeft. Als de spreiding erg klein is ten opzichte van de verwachtingswaarde, dan zal bij het bezwijken van het slechtste deel de verwachte plaatsterkte niet worden bereikt. Verwacht mag worden dat de plaat dan nog niet zijn totale bezwijksterkte heeft bereikt. Want zoals bij het bezwijken van het eerste element in geval van een grotere spreiding de sterkte niet echt veel toeneemt, zal bij het bezwijken van het eerste element in geval van een kleine spreiding de sterkte niet echt veel afnemen.

Bovenstaande redenering is opgezet op basis van de breuksterkte, waarbij er du impliciet is aangenomen dat bij een enkel belastingsgeval de bekleding wordt overbelast. In de praktijk zal daarvan mogelijk geen sprake zijn, maar zal vermoeiing uiteindelijk tot bezwijken leiden. Het hierboven gegeven principe verandert daardoor niet wezenlijk. Als door vermoeiing het eerste, zwakste element bezwijkt, dan zullen de andere elementen navenant zwaarder worden belast, waardoor deze ook eerder aan bezwijken toe zijn.

Conclusies

Deze verkenning leert dat het reëel is om bij de berekening voor de stijfheid rekening te houden met een middeling door de spreiding die op kleine schaal is bepaald te reduceren. Voor de stijfheid lijkt het dus op dat minder extreme waarden in rekening kunnen worden gebracht.

Voor de sterkte is de uitkomst duidelijk anders. Bij het bepalen van een veilige waarde dient er rekening te worden gehouden met het bezwijken van de zwakste delen, waardoor de effectieve sterkte van de plaat inderdaad in de orde van de karakteristieke waarde bepaald op basis van de proefresultaten zal liggen. Toch speelt ook voor de sterkte uitmiddeling een rol: er hoeft niet met de zwakste schakel in de gehele bekleding rekening te worden gehouden. De uitmiddeling mag echter niet worden gebruikt als argument om met het gemiddelde van de sterkte te gaan rekenen.

Als kan worden beschikt over voldoende statistische gegevens met betrekking tot de stijfheid en breuksterkte, dan verdient het aanbeveling om de hier gebruikte aanpak concreter uit te werken en te laten resulteren in een aanpassing van de in de praktijk in rekening te brengen waarden. De correlatielengte van de materiaalparameters is noodzakelijk om te komen tot een betere schatting van de breedte waarover waarnemingen onafhankelijk kunnen worden verondersteld.

Bijlage, numerieke uitwerking plaatbreukmoment voor n = 8

Voor n = 8 leidt dit tot een overschrijdingskans = 1-1/(8+1) = 0,8888: $\mu_{Br} - 1{,}22.\sigma_{Br}$
Aldus wordt $M_1 = 8\ (\mu_{Br} - 1{,}22.\sigma_{Br})$.
De eerstvolgende waarde voor de breuksterkte hoort bij een kans: 1-2/(8+1) = 0,7778, hetgeen leidt tot $\mu_{Br} - 0{,}77.\sigma_B$. Het plaatbreukmoment vlak voor bezwijken van het tweede element is dus: $M_2 = 7\ (\mu_{Br} - 0{,}77.\sigma_{Br})$.
Evenzo: $M_3 = 6\ (\mu_{Br} - 0{,}43.\sigma_{Br})$.
Uit gelijkstellen van M_1 aan M_2 kan worden bepaald dat als $\sigma_{Br} < 1/(4{,}4)\ .\ \mu_{Br}$ dat dan de laagste getrokken waarde van de breuksterkte bepalend is voor de plaat als geheel. De representatieve breuksterkte is dan dus grofweg $(\mu_{Br} - 2.\sigma_{Br})$
Als de spreiding in de breuksterkte groter is: $\sigma_{Br} = 1/3\ \mu_{Br}$.

i	M_i/μ_{Br}
1	0,59
2	0,65
3	0,51

Het blijkt dat er dan een enkel element kan bezwijken, maar dat dit per saldo niet tot een erg veel hoger plaatbreukmoment leidt: $\mu_{Br} - 1{,}05.\sigma_{Br}$ i.p.v. $(\mu_{Br} - 1{,}2.\sigma_{Br})$.