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Safety assessment method for macro-stability of dikes with high foreshores

Master Thesis

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HKV LIJN IN WATER

Safety assessment method for macro-stability of dikes with high foreshores

Master Thesis

By

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Master of Science
In Hydraulic Engineering

at the Delft University of Technology,
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Picture cover page: Dike with high foreshore at Zuider IJsseldijk Gouda.
Picture from www.zuider-ijsseldijk.nl

Preface

This Master Thesis forms the final product in completing the Hydraulic Engineering Master Track at the faculty Civil Engineering of the TU Delft. The research performed in this Thesis, on the subject of macro-stability of dikes with high foreshores, was carried out in the period of May 2017 until January 2018. HKV-consultants, located in Delft and Lelystad, supported the development of the research.

I would like to thank my graduation committee for guiding me through the Thesis project with their feedback, expertise and enthusiasm. First of all, I want to thank my daily supervisor from HKV-consultants, Ir. B. Maaskant, for his advice and feedback throughout the research, which kept me motivated and helped me reach my goals. Also Dr.ir. T. Schweckendiek and Dr. P.J. Vardon, working for the TU Delft, were of great importance for this Thesis, due to their expertise on the subject. Finally, I would like to thank Prof.dr.ir. S.N. Jonkman for being the chairman of my graduation committee and for his advice and guidance, which helped me to perform the research.

I also want to thank my colleagues of HKV-consultants, for always being eager to help me with my questions and for creating a pleasant working environment. Finally, I want to thank my study friends for giving me a great time at the University.

Delft, January 2018

Mick van Montfoort

Executive Summary

Introduction

The Netherlands has always been a country threatened by floods. To protect the land from flooding, it contains approximately 3800km of primary flood defences, like dikes. The safety of these defences is regulated by law in the Dutch Water Act, which states for all dike trajectories the maximum allowed failure probability. By dividing this probability over the different failure mechanisms and over the length of the trajectory the target probability is obtained: the maximum allowed failure probability for a certain failure mechanism per cross-section. The safety of a dike is assessed by calculating its failure probability and comparing it to the target probability.

For the failure mechanism of *macro-stability* (failure of a dike by sliding of the inner slope) the failure probability is determined using a semi-probabilistic method, which results in an approximated failure probability based on a calibrated formula. This formula is calibrated in such a way, that in most cases it should lead to a conservative failure probability, i.e. overestimates the failure probability.

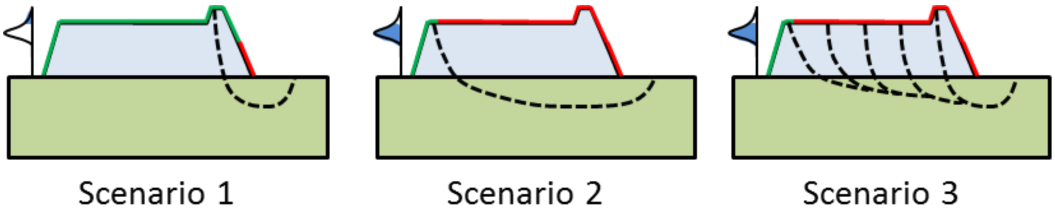
However, for *dikes with high and wide foreshores* (land in front of the dike that is most of the time not flooded) the semi-probabilistic method is probably too conservative, i.e. leading to large overestimations of the failure probability. After all, for water levels below foreshore level, for which the foreshore is not flooded, the dike is probably significantly safer than for water levels above foreshore level. In a semi-probabilistic calculation only one (design) water level is used, which is above foreshore level. The lower water levels, for which the dike is probably significantly safer, are not taken into account, leading to overestimations of the failure probability. This could lead to unnecessary expenses for dike reinforcements. Therefore, this Thesis studies whether it is possible to improve the current safety assessment for dikes with high foreshores, i.e. determine the failure probability in a better (less conservative) way.

Failure definitions

Before calculating any failure probabilities, it is studied what can be defined as failure. Failure is defined as loss of function of the dike, which means the dike is no longer able to retain the water. For a normal dike, without foreshore, the WBI (assessment guidelines) proposes to regard all slip surfaces that have an entrance point halfway the inner slope or further to the water side as failure: the *WBI failure definition*. For dikes with a foreshore, this failure definition neglects the *residual width*. In case the water level is below foreshore level, after sliding of the inner slope there is often still a large foreshore left to retain the water; the dike is nowhere near failure.

To be able to properly define failure for dikes with high and wide foreshores, one has to take into account the residual width. Doing so, there are three scenarios that can lead to macro-stability failure of a dike with a high and wide foreshore. These scenarios are mentioned below and are shown in the figure.

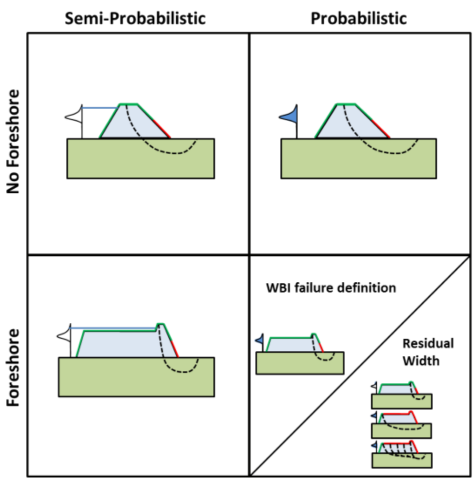
1. Failure of the dike due to a (shallow) slip surface in case the water level is higher than the foreshore level.
2. Failure of the dike due to a deep and long slip surface in case the water level is below the foreshore level.
3. Failure of the dike due to successive shallow slip surfaces in case the water level is below the foreshore level (i.e. progressive failure).



Generally, long sliding planes (scenario 2) or progressive failure (scenario 3) go along with lower probabilities of occurrence than a shallow sliding plane (scenario 1). This supports the hypothesis that a dike is significantly safer for water levels below the level of the foreshore than for higher water levels, as mentioned before. The foreshore thus adds safety to the dike.

Calculations

In this Thesis, different calculations are performed using seven different theoretical test cases. The failure probabilities of these cases, both with and without foreshore, are calculated using both the semi-probabilistic method and fully probabilistic calculations using fragility curves. The latter take into account all water levels, instead of just using one design water level. Furthermore, for the fully probabilistic calculation of the cases with foreshores, two different failure definitions are used: the WBI failure definition and the three scenarios taking into account the residual width, as specified above. All calculation types are shown in the matrix below. By comparing the results of all the calculations the influence of the presence of the foreshore on the failure probability is studied, for different calculation methods and failure definitions. Furthermore, it is studied whether it is possible to improve the current semi-probabilistic safety assessment.



Results and Conclusions

From the results of the calculations, it follows that the presence of a foreshore has a significant influence on the failure probability of a dike. In this Thesis, a maximum reduction of the failure probability, compared to a dike without foreshore, of a factor 25,336 ($\Delta\beta$ = 1.88) is found. Differentiating between clay dikes and sand dikes shows that the influence of the presence of the foreshore is larger for clay dikes. Furthermore, the research shows that the smaller the probability that the foreshore is flooded, the more influence the foreshore has on the failure probability.

The influence as described above is only found if one uses fully probabilistic calculations and takes into account residual width. This combination of calculation type and failure definition best resembles the reality. The semi-probabilistic method and the probabilistic calculation using the WBI failure definition lead to overestimations of the failure probability for dikes with foreshores, and using these does therefore not result in a significant influence of the presence of a foreshore on the failure probability.

Besides, the research shows that the semi-probabilistic method indeed leads to significant overestimations of the failure probability for dikes with foreshores, compared to fully probabilistic calculations. A maximum overestimation of a factor 19,192 ($\Delta\beta$ = 1.82) is found. Using the semi-probabilistic method for the safety assessments can therefore lead to unnecessary dike reinforcements; based on the semi-probabilistic assessment a reinforcement is needed, whereas, if one would perform a fully probabilistic calculation, no reinforcement is needed at all. The maximum overestimation described above corresponds to a stability berm of 2.5m high and 8.5m wide which is needed in the semi-probabilistic method to arrive at the same amount of safety as found in the probabilistic calculation using no berm at all. This proves that there is room for improvement of the current semi-probabilistic safety assessment.

Improvements safety assessment

After proving the need for improvement of the semi-probabilistic safety assessment, this Thesis proposes possible improvements for the assessment. The best way to prevent large overestimations of the failure probability, is to switch from a semi-probabilistic assessment method to a fully probabilistic method, as used in this Thesis. This fully probabilistic method is based on fragility curves and takes into account the residual width of the foreshore. As concluded before, this resembles the reality in the best way.

However, fully probabilistic calculations are relatively difficult to perform. Therefore, as an alternative safety assessment method, this Thesis proposes the concept of a semi-probabilistic assessment using two separate assessment rules: one for water levels below foreshore level and one for water levels above foreshore level. Because the probability of flooding of the foreshore is included in this assessment, it better resembles reality than the current semi-probabilistic method.

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Roman Symbols

c'	Cohesion	[kN/m ^{2}]
D	Thickness aquifer	[m]
d	Thickness aquitard	[m]
$F(x)$	Cumulative distribution function (CDF)	[-]
$f(x)$	Probability density function (PDF)	[-]
h	Water level	[m+NAP]
k	Hydraulic conductivity aquifer	[m/s]
k'	Hydraulic conductivity aquitard	[m/s]
m	Stress increase exponent	[-]
m_d	Model uncertainty factor	[-]
N	Length effect factor	[-]
P_f	Annual failure probability	[-]
$P_{f\|h=..}$	Conditional failure probability given water level h	[-]
$P_{f,target}$	Target probability for a dike cross section	[-]
R_k	Characteristic value strength parameter	[-]
S	Undrained shear strength ratio	[-]
S_k	Characteristic value load parameter	[-]
s_u	Undrained shear strength	[-]
u	Location parameter Gumbel distribution	[-]
Z	Limit State Function	[-]

Greek Symbols

α	Scale parameter Gumbel distribution	[-]
α^2	Influence factor	[-]
β	Reliability index	[-]
γ	Volumetric weight soil	[kN/m^3]
γ_d	Model factor	[-]
γ_n	Damage factor	[-]
λ	Leakage length	[m]
μ	Mean value (log)normal distribution	[-]
σ	Standard deviation (log)normal distribution	[-]
σ_n	Normal stress on slip surface	[kN/m^2]
σ'_{vi}	Vertical effective stress	[kN/m^2]
σ'_{vy}	Vertical yield stress	[kN/m^2]
τ	Ultimate shear stress (shear strength)	[kN/m^2]
Φ	Cumulative standard normal distribution	[-]
φ'	Effective friction angle	[degrees]
φ_{cs}	Critical state friction angle	[degrees]
ω_i	Contribution of failure mechanism i to the total failure probability (according to the failure probability budget)	[-]

Abbreviations

CBA	Cost Benefit Analysis	[-]
COV	Coefficient of Variance	[-]
FoS	Factor of Safety	[-]
GHW	Mean high water	[m+NAP]
LEM	Limit equilibrium model	[-]
MHW	Design water level, used in the calculations	[m+NAP]
OCR	Over-consolidation ratio	[-]
POP	Pre-overburden pressure	[kN/m^2]

1. Research Introduction

1.1. Introduction

The Netherlands has always been a country threatened by floods. Approximately 60% of the Netherlands is prone to flooding. This is not just because part of the Netherlands lies below sea level; large river discharges also pose flood threats. In case of a flood, water levels of more than five meters can be reached. Needless to say, flood protection is very important for the people living and working in the Netherlands. Therefore, the protection against flooding is regulated by law in the Dutch Water Act.

To protect the land from flooding, the country of the Netherlands contains approximately 3800 kilometres of primary flood defences like dikes, dams and dunes. To guarantee the safety against flooding, the Water Act contains standards for the safety of the primary flood defences. However, recently these standards changed. Since the beginning of 2017, they are based on a new approach. The old safety standards stated exceedance frequencies for the design water level for all flood defences in the Netherlands. The safety of a defence was assessed by checking the ability of that defence to withstand the design water level. The new safety standards are not based on a design water level, but on a probability of flooding: for all flood defences the maximum allowed failure probability is stated in the Water Act. To assess the safety of defences according to the new safety standards, the failure probability of a defence has to be determined and compared to the maximum allowed probability. To determine the failure probability of defences, probabilistic calculations have to be performed, for which guidelines have been developed by the Dutch government (Rijkswaterstaat, 2016a; Slootjes & van der Most, 2016).

The failure probability of a flood defence is not just the probability of the water level exceeding the height of the defence, but different failure mechanisms are taken into account. For every failure mechanism there is a safety assessment method. In this Master Thesis the focus will be on the failure mechanism of *macro-stability* (failure of a dike by sliding of the inner slope). For this failure mechanism, the guidelines propose a semi-probabilistic assessment, based on a design water level, like in the old safety standards. This approach is probably *conservative* (overestimating the failure probability), especially in the case of dikes with *high and wide foreshores* (land in front of the dike that is only flooded in case of very high water levels). Therefore, the question arises whether the current macro-stability assessment for dikes with high and wide foreshores can be improved.

1.2. Problem Statement

As mentioned, the new safety standards prescribe the maximum allowed failure probability for all flood defences in the Netherlands. As a result of this, to assess the safety of a flood defence, for example a dike section, one has to determine its failure probability and compare it to the maximum allowed failure probability. In order to determine the failure probability of the dike section, one computes the failure probability for every failure mechanism and combines these to a total failure probability.

In order to determine the failure probabilities for the different mechanisms, one has to perform probabilistic calculations. In a probabilistic calculation the different parameters (water levels, strength properties) are put in as stochastic variables (with a probability distribution). This way, the uncertainties of the different parameters are taken into account. However, probabilistic calculations are time-consuming and difficult to perform for persons without specific knowledge in probabilistic methods. At this time, most dike managers, who perform the assessments, indeed do not have specific knowledge in this area. After all, the old safety standards were not based on failure probabilities, so the need for probabilistic calculations in safety assessments of flood defences is new.

The procedure to be followed for the safety assessment of flood defences is described in the WBI 2017 (NL: “Wettelijk Beoordelings Instrumentarium”). For the failure mechanism of *macro-stability*, on which this Master Thesis focuses, the WBI proposes a semi-probabilistic assessment, instead of the more complicated fully probabilistic calculations. The semi-probabilistic assessment is based on a design water level. For the strength parameters design values have to be determined too, based on their stochastic distributions. Using the design values of all parameters, a factor of safety[1] is computed. To compare the results to the maximum allowed failure probability, the factor of safety is transformed into a failure probability using the following calibrated formula (Kanning, Teixeira, van der Krogt, & Rippi, 2016):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right) \tag{1.1}$$

Note that this means that this failure probability is an approximated probability. The formula described above has been calibrated in such a way, that the formula results in a conservative failure probability in most cases, meaning that it overestimates the failure probability.

For dikes with *high and wide foreshores,* e.g. at Hollandsche IJssel and in city centers like Delfland, Dordrecht and Zwolle, the semi-probabilistic method as mentioned above may be too conservative, leading to large overestimations of the failure probabilities. High foreshores are not flooded most of the time. When they are, the water levels are very high, and the dike is small.

[1] In the most basic macro-stability models, using a circular slip plane, the factor of safety is equal to the ratio between the driving and the resisting moment along the slip plane. In other, more sophisticated models, the factor of safety is more complicated. This will be described in more detail later on.

When the water level is below the level of the foreshore, the dike will be much broader, and the water is far away from the hinterland, which means that the water-retaining body is larger (See Figure 1.1). This means that a dike is probably significantly safer with respect to macro-stability for water levels below the level of the foreshore than for higher water levels.

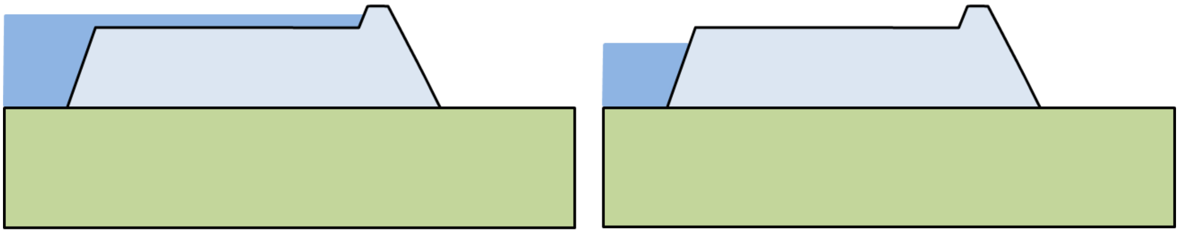


Figure 1.1: Dike with foreshore for a water level above foreshore level and below foreshore.

In reality, all water levels contribute to the failure probability. However, the semi-probabilistic method as proposed by the WBI approximates the failure probability based on a single design water level. This design water level is (in most cases) higher than the level of the foreshore, leading to a small dike (left side of Figure 1.1). The water levels below the level of the foreshore, for which the dike is much broader and probably safer, are not taken into account in the semi-probabilistic method. The assessment is purely based on the design water level and the corresponding small dike. One can imagine that taking into account the water levels below foreshore level, for which the dike is probably safer, may very well decrease the total failure probability. This means that the current semi-probabilistic approach would overestimate the failure probability.

One could take the water levels below foreshore level into account by performing a fully probabilistic calculation. However, dike managers probably rather use a semi-probabilistic method than a fully probabilistic calculation, because it is easier to perform and they do often have no experience in performing fully probabilistic calculations. If the semi-probabilistic method indeed leads to overestimations of the failure probability, using this method may lead to unnecessary expenses for dike reinforcements.

1.3. Research Questions

This Master Thesis will try to answer the following research question:

Is it possible to improve the safety assessment method for macro-stability for dikes with high and wide foreshores?

To answer this research question, the following sub-questions will be used:

1. What method is currently used to assess the safety of dikes in general and specifically for the macro-stability failure mechanism?
2. Which processes can lead to macro-stability failure of dikes with high and wide foreshores?
3. For dikes in general, does the semi-probabilistic assessment method for macro-stability lead to a conservative result?
4. For dikes with high and wide foreshores, is the dike significantly safer for water levels below the level of the foreshore than for water levels above foreshore level?
5. What is the influence of the presence of a foreshore on the failure probability for macro-stability of a dike?
6. Can the current safety assessment method for macro-stability be improved, to account for the presence of a foreshore? If yes, what improvements can be made?

1.4. Approach and report structure

Figure 1.2 shows the approach of this research. The research is divided in a number of steps. The figure shows which steps will be covered in which chapter.

Chapter 2 contains a literature study, which presents the necessary theory and background information for this Thesis. In Chapter 3, this theory is focused on the specific cases of dikes with high foreshores. Chapter 4 sets up the framework for the calculations that will be performed in this research. Chapters 5, 6 and 7 contain the different calculations and the evaluations of the results. Chapter 8 proposes possible improvements for the safety assessment method. In Chapters 9 and 10 the conclusions of this Thesis will be presented and recommendations will be made.

Literature Study • Provide the background information for this Thesis • Analyse the macro-stability failure mechanism • Analyse the current safety assessment method for macro-stability	***Chapter 2***
High Foreshores • Analyse the different processes that can lead to macro-stability failure for dikes with high and wide foreshores	***Chapter 3***
Calculation Set-Up • Set up a framework for the different calculations that will be performed in this Thesis • Specify the schematization and calculation choices used for the calculations	***Chapter 4***
Semi-Probabilistic vs. Probabilistic • Compare the results of semi-probabilistic and fully probabilistic calculations for dikes without foreshores • Analyse whether or not the semi-probabilistic method is conservative	***Chapter 5***
Safety for different water levels • Set up an approach to assess the safety of dikes with foreshores for different water levels in a proper way • Compare the safety of a dike for water levels below foreshore level and above foreshore level	***Chapter 6***
Influence of the foreshore • Analyse the influence of the foreshore on the failure probability, using different types of calculations • Determine whether or not the current semi-probabilistic method is conservative for dikes with high foreshores, i.e. whether there is room for improvement	***Chapter 7***
Improved assessment method • Propose possible improvements for the assessment of macro-stability for dikes with high foreshores	***Chapter 8***
Conclusions and discussion • Present the conclusions of this Thesis, leading to the answer to the research question • Propose recommendations for further research	***Chapter 9, 10***

Figure 1.2: Structure of the report

2. Literature Study: Theory and Background Information

2.1. Introduction

This chapter refers to the first sub-question from Section 1.3: *"What method is currently used to assess the safety of dikes in general and specifically for macro-stability?"* It will give the necessary theory and background information for this research, starting with a general description of the flood protection in the Netherlands and an overview of the different probabilistic calculation methods. Then, going more and more into detail, the failure mechanism of *macro-stability* is covered.

2.2. Flood Protection in the Netherlands

This section describes the flood protection policy in the Netherlands, starting with the general background of this policy. As mentioned before, in the beginning of 2017 new safety standards have come into force. Both the old and the new safety standards will be covered separately, ending up with a general description of the current safety assessment process.

2.2.1. General Background

To get to the background of the current policy for flood protection in the Netherlands, one has to go back to 1953, when a major flood took place in the Netherlands (NL: "Watersnoodramp"). During this major disaster 2000km^2 of land was flooded, which caused 1836 deaths (Kok, Jongejan, Nieuwjaar, & Tánczos, 2016). Directly after the flood, the Delta Committee was founded, which had the responsibility to prevent such a flood from happening again. As a result of that, safety standards for flood defences, the flood risk standards, were legislated in the Dutch Water Act. The Delta Committee proposed design water levels, with a certain probability of exceedance, which the defences should be able to withstand. This resulted in a completely different approach than in the situation before 1953, when the dikes were heightened based on the highest observed local water level. The decisions of the Delta Committee meant a change from a reactive approach (based on the highest observed water level in history) to a proactive approach (based on a certain water level that may be expected in the future).

Since the advice of the first Delta Committee, the safety standards for primary flood defences in the Netherlands have been regulated by law. Primary flood defences protect the land against flooding from the big waters: the sea, large rivers and large lakes. Different types of primary flood defences are dikes, dams, dunes and constructions which can be parts of those, like sluices. In the Netherlands there are approximately 3800 kilometres of primary flood defences. Besides, there are regional flood defences, which protect the land against floods from other, smaller waters, like channels. The consequences of failure of regional flood defences are much smaller than the consequences of failure of primary flood defences. Therefore, safety standards of regional flood defences are not regulated by the Dutch Water Act, but determined by local authorities.

In recent years, due to different studies, interest shifted from the safety of flood defences (based on a design water level) towards flood risks. One of the studies responsible for this was VNK2 (NL: "Veiligheid Nederland in Kaart 2") (Vergouwe, 2015), in which the flood risks in the Netherlands were quantified, taking into account not only the safety of the flood defences, but also the consequences in case a flood would occur. Risk is determined as follows (Jonkman et al., 2016):

$$Risk = Probability * Consequences \tag{2.1}$$

This means that flood risk is the product of the probability of a failure of a certain flood defence and the consequences of flooding due to failure of that defence. Accordingly, flood risk can be quantified by the following procedure (Kok et al., 2016):

1. Determine the loads on the flood defence;
2. Determine the failure probability of a flood defence, considering the loads and its strength;
3. Develop flood scenarios: determine the different ways in which a flood can spread, in case failure of a flood defence occurs;
4. Determine the consequences for each flood scenario (death toll, economic damage);
5. Combine the above probabilities and consequences to arrive at a flood risk.

The approach mentioned above was used in the VNK2 project. Due to studies like this, more and more knowledge on flood risk was gained. This has led to new safety standards for flood defences, which have come into force in the beginning of 2017. As said before, the previous safety standards were expressed as a design water level, which the flood defences should be able to withstand. The new safety standards are expressed as a maximum probability of failure. The main reason for this change is that the new standards express the amount of safety in a better way and the fact that there is a clear link between the safety standards and flood risk (Slootjes & van der Most, 2016). In the next sections both the old and the new safety standards will be covered in more detail and differences will be highlighted.

2.2.2. Old Safety Standards

As mentioned before, the first safety standards for flood defences in the Netherlands were developed after the big flood in 1953. The reason for developing clear standards was the need to assess in a unified way whether the country was safe against flooding. The standards were expressed as certain exceedance probabilities for design water levels. The safety assessment of flood defences was merely a check whether a flood defence was able to withstand the corresponding design water level. For the assessment different failure mechanisms have been determined, which are different ways in which failure of a flood defence can take place. The most important failure mechanisms are (Kok et al., 2016):

- Overflow: Water flowing over the crest of the dike (possibly in combination with wave overtopping) causing erosion of the inner slope;
- Macro-instability inner slope: Sliding of the inner slope due to an increase in pore pressures;
- Piping: Development of a “pipe” of flowing water underneath a dike body, due to head difference over the dike, leading to undermining of the dike;
- Micro-instability: Instability of the inner slope by leakage of water through the dike body;
- Erosion outer slope: Erosion of the outer slope due to wave attack or currents;
- Macro-instability outer slope: Sliding of the outer slope due to a rapid decrease of water level.

Note that the failure mechanisms described above are all failure mechanisms for dikes. Other types of flood defences fail due to other mechanisms. These are not further described here. To assess the safety of a dike (or any other flood defence) according to the old safety standards, one had to check for every failure mechanism whether failure would occur if the dike would be loaded by the design water level.

The exceedance probabilities for the design water level were determined by weighing the costs of reinforcing a dike against the reduction of the flood risk. This was done for every *dike ring* (a circular chain of flood defences) separately. In other words: for every dike ring a certain exceedance probability for the design water level was chosen. In the Western part of the country the consequences of a flood would be the biggest, so the safety standards were the most strict in that part: the exceedance probability of the design water level was 1/10,000 per year. This means that the flood defences had to be able to withstand a water level that would occur once every 10,000 years. The different exceedance probabilities for the different dike rings in the Netherlands, as set by the old safety standards, are shown in Figure 2.1.

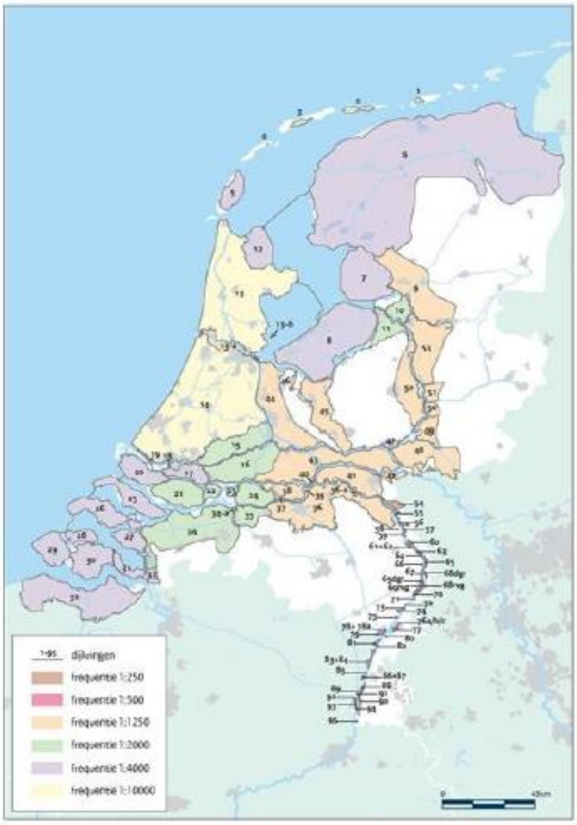


Figure 2.1: Old safety standards for all dike rings (Jonkman & Schweckendiek, 2016)

2.2.3. New Safety Standards

In the beginning of 2017, the new safety standards have come into force. As said before, these safety standards are no longer expressed as an exceedance probability for the design water level, but as a maximum allowed failure probability. Besides, the standards are no longer determined for a dike ring, but for a *dike trajectory.* A dike trajectory is a part of a dike ring that has, for a failure anywhere along the trajectory, approximately the same consequences. For different dike trajectories (of the same dike ring) , the consequences may be different and therefore they get assigned different safety standards. All dike trajectories are legislated in the Dutch Water Act.

As said before, the reason for expressing the new standards as maximum allowed failure probabilities, instead of some design water level, is that the failure probabilities provide a better understanding of the amount of safety of the dike. Moreover, they are closely linked to flood risk. The maximum allowed failure probabilities for the dike trajectories, as stated in the Dutch Water Act, are based on what amount of flood risk is supposed to be acceptable. The standards are derived following four principles (Kok et al., 2016):

1. Every person, living in the Netherlands, should be able to rely on the same basic level of safety. This is accounted for by the *LIR*, the Local Individual Risk, which is the probability per year of an individual person dying as a result of a flood. The LIR may not be higher than 10^{-5} anywhere in the Netherlands, which gives the minimum level of protection. The LIR at a certain location can be calculated as follows:

$$LIR = P_f * M * (1 - EF) \tag{2.2}$$

In which:

P_f Annual failure probability of a dike trajectory (probability of a flood)

M Mortality: The fraction of people that would die in case of a flood [-]

EF Evacuation fraction: The fraction of people that can be successfully evacuated in case of a flood [-]

By studying flood scenarios for all dike trajectories, the mortality and evacuation fraction can be determined. Given the LIR of 10^{-5} the maximum allowed failure probabilities can be determined, that fulfil the LIR criterion.

For dike trajectories where the consequences of a flood are very large, a lower maximum allowed failure probability is more appropriate, based on the *group risk* criterion and a *Cost Benefit Analysis (CBA)*. This is the case for dike trajectories that protect areas with a lot of residents or a lot of economic value.

2. A CBA searches for the most cost optimal failure probability for a dike trajectory, by weighing the consequences of a flood against the costs to decrease the probability of a flood. By investing in the strength of dikes, the probability of a flood is decreased and thus the risk is decreased. The most optimal failure probability is the failure probability for which the total costs (investment costs + flood risks) are minimal. This approach is illustrated in Figure 2.2.

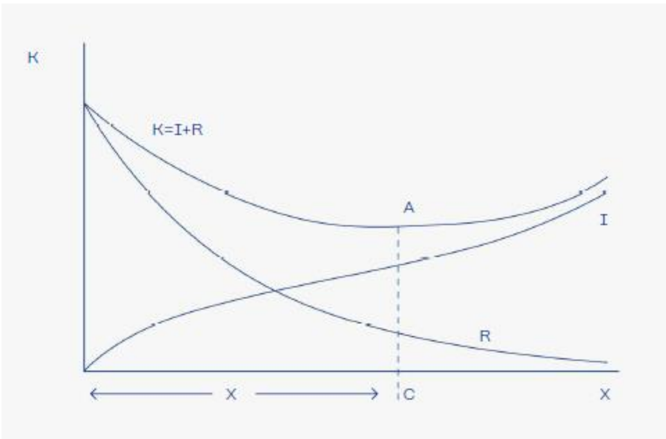


Figure 2.2: CBA (Kok et al., 2016) ***K are total costs, I are investment costs and R are risks***

In case the optimal failure probability as determined by the CBA is lower than the one determined by LIR for a certain dike trajectory, the failure probability from the CBA is used as safety standard.

3. The group risk describes the probability of a flood with a high death toll. The group risk is mostly expressed by a FN-curve, which gives the relation between a flood with a certain number of deaths and the probability of occurrence of that flood. In first instance, the group risk is determined for the complete country, because it is the total amount of deaths that matters, not the ones per dike trajectory. For the group risk of the complete country, a maximum boundary is determined. Based on this maximum boundary, it can be concluded that the safety standards following from LIR and CBA make sure the group risk on the scale of the country is sufficiently small. However, for six trajectories the maximum allowed failure probability is decreased based on the high number of potential deaths.

4. Finally, in case critical infrastructure, e.g. a nuclear power plant, is protected by a certain dike trajectory, even stricter safety standards than derived using the criteria above are used.

Based on the criteria above, for every dike trajectory the maximum allowed failure probability has been determined. These are shown in Figure 2.3. As the safety standards are now expressed as maximum allowed failure probabilities, the assessment of the safety of a dike trajectory means that the total failure probability of that dike trajectory has to be calculated and compared to the standard. For the design of a dike reinforcement (or a new dike), the same approach is used. The only difference is that a design is made for certain period of time, and the safety assessment is merely checking whether a dike is safe at the moment. This means that for a design one has to use values for the water level as they will be at the end of the design period, taking into account climate change (e.g. higher sea water levels and higher river discharges). This also holds for the strength of the dike, where subsidence and degradation should be taken into account. Apart from that, the approaches for design and assessment are the same (Rijkswaterstaat, 2017). Therefore, from now on, only assessment will be covered.

Figure 2.3: New safety standards: maximum allowed failure probabilities for all dike trajectories

To calculate the total failure probability of a dike trajectory, the trajectory is split up into multiple sections. These sections are approximately homogeneous in subsoil conditions and hydraulic loads. For every failure mechanism, in every dike section the most critical cross section is chosen. For these cross sections, for every dike section, for every failure mechanism the failure probability is calculated. In theory, one could, for every failure mechanism, combine the failure probabilities of the different sections, taking into account the *length effect*. This results for every failure mechanism in a failure probability for the complete trajectory. These failure probabilities (one for every failure mechanism) could be combined to one total failure probability for the trajectory, which can be compared to the maximum allowed failure probability.

The *length effect,* that was mentioned above, can be described as: “the increase in failure probability with the length of the dike due to partial correlations or independence between different cross-sections and/or elements” (Kanning, 2012). The length effect thus largely depends on the spatial correlation of parameters. The correlation lengths of hydraulic loads are much higher than those of resistances. Loads are mostly fully correlated over a dike trajectory. The correlation lengths of resistances are much smaller, which means that the dike sections show little or no correlation in resistance parameters. Because the dike trajectory can be seen as a *series system*, meaning that the trajectory fails if one element fails, little correlation between the sections means that the failure probability increases with the length of the trajectory. After all, the sections are (almost) independent, meaning that the probability of finding a weak spot increases with the length. To account for the length effect in the approach described above, one has to include correlations in the calculations. There are various methods to do this, but these fall outside the scope of this research, so they are not described.

Different guidelines describe the process that has to be followed in the safety assessment of dikes. Together, these guidelines form the WBI (NL: "Wettelijk Beoordelingsinstrumentarium") Besides the theoretical approach for calculating the total failure probability of a dike trajectory as explained above, the WBI (Rijkswaterstaat, 2016a) provides a simplified approach, which is most commonly used. This approach works just the other way around. The maximum allowed failure probability for the complete dike trajectory (safety standard) is divided between the different failure mechanisms, following a failure mechanism budget. This budget determines which contribution every failure mechanism may have to the total maximum allowed failure probability. (Rijkswaterstaat, 2016a) gives a basic failure mechanism budget. However, one may change the budget at one's own discretion. The next step is to determine, for every failure mechanism, the allowed failure probability per dike section, taking into account the length effect. However, in this simplified approach the length effect is approximated by a length effect factor N. Dividing the failure probability for a certain failure mechanism (on trajectory scale) by the length effect factor, results in the target failure probability (maximum allowed failure probability for a dike section / cross section). The approach can be schematized by the following formula:

$$P_{f,target} = \frac{P_f * \omega_i}{N} \tag{2.3}$$

In which:

$P_{f,target}$	The target failure probability [-]
P_f	The maximum total failure probability for the dike trajectory [-]
ω_i	Contribution of failure mechanism i to the total failure probability (according to the failure probability budget) [-]
N	Length effect factor [-]

The approach is also shown in Figure 2.4. In this simplified approach, the length effect parameter is determined using standard formulas, and only depends on the length of the dike trajectory and the type of failure mechanism. One does not have to define correlations. However, this means that the length effect is an approximation. The simplified approach leads to target probabilities for the different sections and failure mechanisms that are stricter than necessary, to be sure that, combined, they do not exceed the total maximum allowed failure probability of the dike section (Jonkman & Schweckendiek, 2015). In other words, the simplified approach is easier to use than the theoretical approach, but it is also conservative.

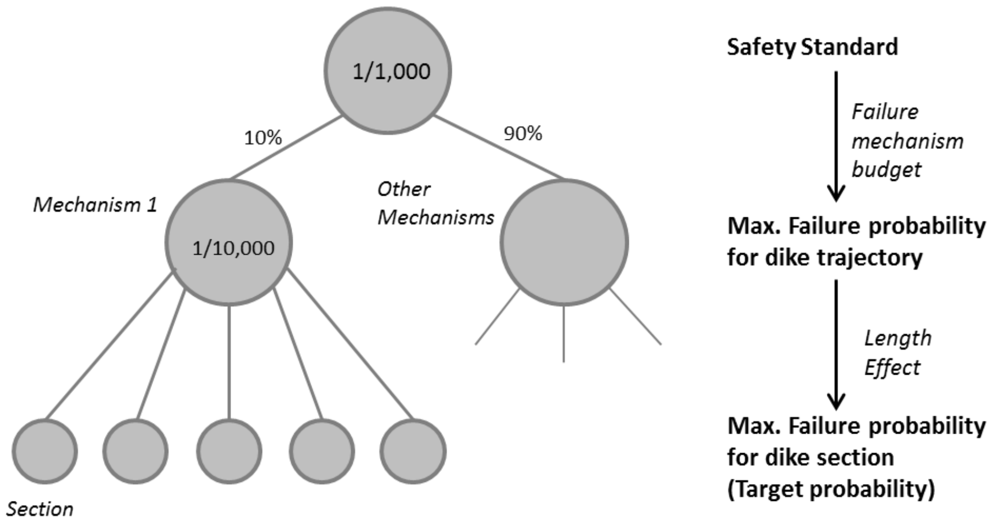


Figure 2.4: Approach determining the target reliability (Rijkswaterstaat, 2016a) (*translated to English*)

The simplified approach as described above results in a target failure probability per cross section per failure mechanism. To assess the safety of the dike, for a certain failure mechanism, the failure probability of the cross section has to be compared to the target failure probability. The failure probability can be calculated using a probabilistic calculation. However, again the WBI (Rijkswaterstaat, 2016a) proposes a simplified semi-probabilistic approach, based on partial safety factors, which are the outcomes of calibration studies. The resulting failure probabilities are merely approximated probabilities, which are in most of the cases conservative estimates.

In the next section, more attention is paid to the probabilistic framework, which will give more clarity in the difference between probabilistic and semi-probabilistic calculations.

2.3. Probabilistic Framework

The previous section gave a description of the new safety standards and the safety assessment of dikes, going more and more into detail and finishing with the calculation of the failure probability for a certain failure mechanism, on cross section scale. As mentioned, there are two possible approaches to calculate this failure probability: a fully-probabilistic approach and a semi-probabilistic approach, which is an approximation. To be able to better understand the differences between the two approaches, this section covers failure probabilities in general and some probabilistic calculation methods and their differences.

2.3.1. Failure Probability

In its simplest form the verification of safety is performed by expressing that the structural resistance R is larger than the load S that can act on the structure (a dike in this study) over its lifetime (Jonkman et al., 2016):

$$R > S \tag{2.4}$$

However, resistance and load parameters are not deterministic quantities, but random variables. The strength of a dike for instance varies from location to location. In other words, there is uncertainty in the values of these parameters. This means they cannot be described by one single value, but have to be described by a probability density function.

Based on equation (2.4) a limit state function (Z) can be determined:

$$Z = R - S \tag{2.5}$$

Failure occurs if the load S exceeds the resistance R, i.e. for Z < 0. The probability of failure is thus described by $P_f = P[Z < 0]$. Another way to describe the reliability is using the reliability index β. The failure probability and the reliability index are related as follows:

$P_f = \phi(-\beta)$, in which ϕ is the cumulative standard normal distribution.

In Figure 2.5 the probability of failure is illustrated for certain probability density functions of S and R. In here, the area where it holds that R < S is equal to the failure probability.

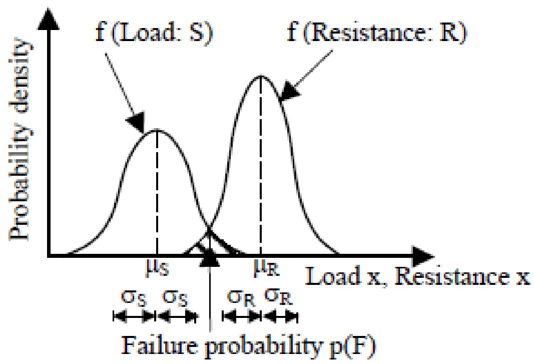


Figure 2.5: Illustration of the failure probability: area where S exceeds R in probability density plane (Möllmann & Vermeer, 2010)

In (R,S)-plane, the line Z=0 can be drawn. This is the borderline between the safe domain and the unsafe domain. In Figure 2.6 this is shown. Besides, the joint probability density function for the strength- and resistance parameters is shown using lines for equal probabilities. From this illustration, it can be seen that the failure probability is equal to the volume of the joint probability density function in the unsafe domain. This means that the failure probability can be calculated by integrating the joint probability density function in the unsafe domain. In formula, this becomes (assuming R and S are independent and consist of m and n parameters respectively) (Jonkman et al., 2016):

$$P_f = \iiint_{z<0} \dots \int f_R(x_1, x_2 \dots . x_m) f_S(x_{m+1}, x_{m+2} \dots . x_n) dx_1 dx_2 \dots . dx_n \tag{2.6}$$

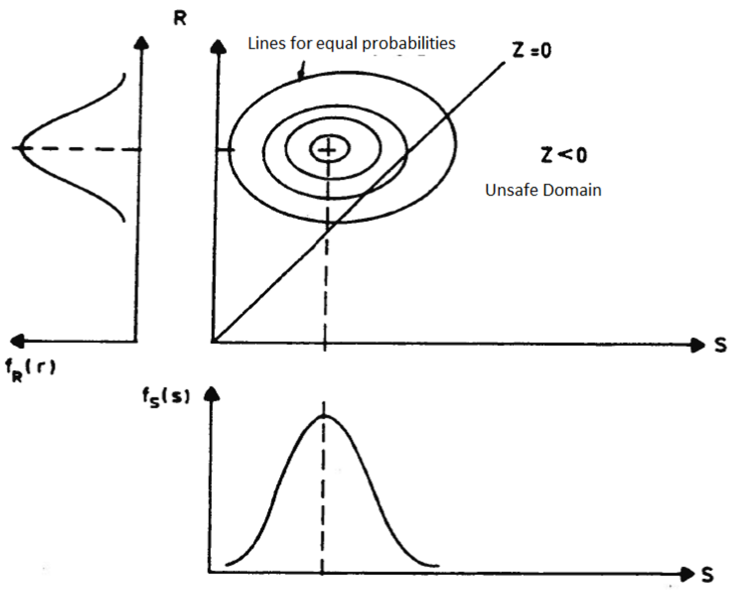


Figure 2.6: Illustration of failure probability: area of the joint probability density function in the unsafe domain in (R,S)-plane (Jonkman et al., 2016)

2.3.2. Reliability Methods

There are different methods to calculate the reliability (or failure probability) of a dike. Generally, the methods can be divided into five groups (Jonkman et al., 2016):

- **Level IV methods**: In these methods the consequences of failure are also taken into account and the risk is used as measure of reliability, instead of the failure probability. Actually, this is what was done in the determination of the safety standards. This method is not used to actually calculate the failure probabilities and will therefore not be further discussed.

- **Level III methods:** In these methods the uncertain parameters are modelled by their joint distribution functions, as explained before. The probability of failure is calculated exactly, e.g. by numerical integration as explained before (equation (2.6)). Another frequently used level III method is Monte-Carlo simulation, in which random samples of the parameters are generated and for each generation of samples it is determined whether failure would occur (Z<0). By taking the ratio of the amount of generations leading to failure and the total amount of generations, the failure probability is calculated.

- **Level II methods:** These methods are a simplification of the level III methods. The mostly used one is FORM (First Order Reliability Method). In FORM the limit state function is linearized in the design point. The design point is the most probable combination of input parameters at failure. If the input parameters R and S are transformed into a standard-normalized space, the design point is the point where the distance between the limit

state function Z and the origin is shortest, as can be seen in Figure 2.7. Furthermore, this illustration shows that the distance between Z and the origin is equal to the reliability index β. After linearization, the reliability index can be calculated as follows:
$\beta = \frac{\mu_z}{\sigma_z}$, in which μ_z is the mean of Z and σ_z the standard deviation. Using β, the failure probability can be computed. The advantage of this method compared to for example Monte Carlo Simulation, is that it is much less time-consuming. Another advantage is that influence coefficients (α) are obtained, which for each parameter describe its relative contribution to the uncertainty. However, note that the linearization of the limit state function leads to a small error, which is a disadvantage of this method.

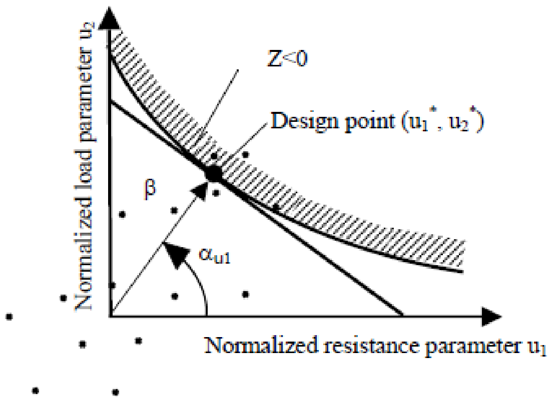


Figure 2.7: Design point in standard normal space (Möllmann & Vermeer, 2010)

- **Level I methods:** Level I methods are semi-probabilistic calculations. The essence of these methods is that design values are used for strength and load parameters, which take into account the uncertainty in the parameters. The design values for strength parameters are found by multiplying some characteristic value with a safety factor and those for load parameters are found by dividing a characteristic value by a safety factor, after which the following must hold:

$$\frac{R_k}{\gamma_R} > S_k * \gamma_S \tag{2.7}$$

The characteristic values R_k and S_k are calculated as follows:

$$R_k = \mu_R + k_R * \sigma_R \tag{2.8}$$
$$S_k = \mu_S + k_S * \sigma_S \tag{2.9}$$

In which k_R has a negative value and k_S a positive value. Both parameters depend on the distribution quantile which is used for the characteristic value. In flood defence design generally the 5% and 95% quantile are used for strength and load parameters respectively, corresponding to k_R = -1.645 and k_S = 1.645 (Jonkman et al., 2016). The procedure explained above is shown in Figure 2.8. The figure shows the probability density functions of both strength and load parameters, together with their characteristic values and design values. Note that R_d (design value of R) is larger than S_d, which is the essence of the design.

- **Level 0 methods:** These methods are deterministic calculations. The approach of level 0 methods is actually the same as for level I methods. The only difference is that the assessment is not carried out using design values or characteristic values, but using mean values. Uncertainty is not taken into account

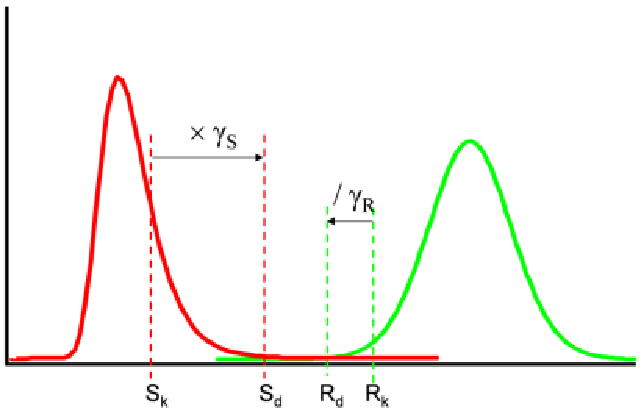


Figure 2.8: Illustration of characteristic values and design values, used in the Level I method (Jonkman & Schweckendiek, 2016)

As mentioned before, the WBI guidelines propose a semi-probabilistic method to calculate the failure probability of a dike cross section for a certain failure mechanism. As explained above, a semi-probabilistic method means that for every parameter one (design) value is used, and using these values the safety is assessed. This means that only one water level is used for the assessment, the design water level.

In this Thesis, both the semi-probabilistic method as proposed by the WBI and fully probabilistic calculations will be used. For the fully probabilistic calculations, the method of fragility curves is used. This method will be described in the next section.

2.3.3. Fragility Curves

Fragility curves are often used as method for performing probabilistic calculations for dike assessments. Especially for dikes with foreshores, they prove to be a useful method. This will be explained later on. As fragility curves form an important part of this Thesis, they will be covered in detail in this paragraph.

A *fragility curve* expresses the failure probability as a function of the load. For different values of the load, in most cases the water level, the fragility curve shows the conditional failure probability, i.e. the failure probability given a certain water level. An important advantage of fragility curves is that they are relatively easy to produce. Besides, they show in a graphical way the influence of the water level (the load) on the failure probability. The steeper the fragility curve, the higher the influence of the water level. If fragility curves for different failure mechanisms (for the same cross-section) are depicted in the same figure, one can easily see which failure mechanism is critical for a certain water level. This is illustrated in Figure 2.9, in which fragility curves for different failure mechanisms are shown. As can be seen, in this case the curve for overtopping is the steepest, meaning that the water level influences the failure probability of the overtopping mechanism the most. Besides, it can be seen that for relatively low water levels the piping and macro-stability mechanisms are critical, as their conditional failure probabilities are relative high, and for high water levels the overtopping mechanism becomes critical (Wojciechowska et al., 2015). Note that a fragility curve is not necessarily constructed using the water level as variable. Other parameters can be used too. In some cases this may even be better, because for macro-stability, the water level is not always the dominant parameter.

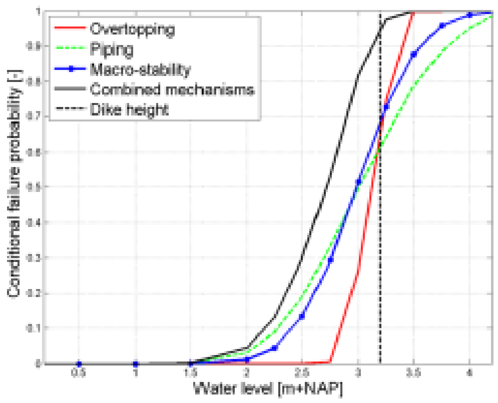


Figure 2.9: Illustration of fragility curves for different failure mechanisms (Wojciechowska et al., 2015)

For the construction of a fragility curve for a certain failure mechanism, the following steps are followed, based on (Bischiniotis, 2013):

1. Identify a Limit State Function (Z) for the failure mechanism;
2. Choose a value for the water level (h_w);
3. Take all other parameters into account, including their uncertainties and probability distributions. For some failure mechanisms, like macro-stability, some parameters depend on the water level (e.g. pore pressure distribution). It is important to determine these parameter for every chosen water level (step 2) separately;
4. Perform a probabilistic calculation using the water level from step 2 as a deterministic value and the distributions as determined in step 3 for the other parameters. The calculation can be performed either by using a Monte Carlo simulation or using a FORM calculation;
5. Repeat steps 2-4 for different water levels.

The result of these steps is a set of conditional failure probabilities. By plotting these against the corresponding water levels, a fragility curve is obtained. In Figure 2.10 a fragility curve is shown. Note that a fragility curve, expressing the probability of failing given a water level, at the same time represents the cumulative distribution function of the critical water level (h_c), which is the water level at which the dike fails. As the critical water level can be seen as the resistance of the dike, the fragility curves also represents the cumulative distribution function of the resistance ($F_r(h_w)$) (Kanning & Schweckendiek, 2016; Schweckendiek & Kanning, 2016).

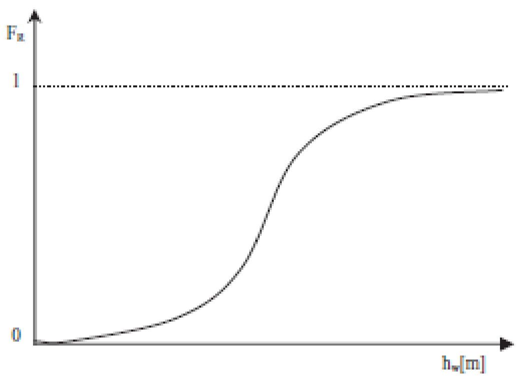


Figure 2.10: Illustration Fragility Curve (van der Meer, ter Horst, & van Velzen, 2009)

A fragility curve shows the conditional failure probabilities, i.e. the failure probability given a certain water level. In dike safety assessments, one is interested in the total annual failure probability of a cross section. Assuming R and S to be independent, the failure probability can be described by the following integral, where s is a certain value of the load (van der Meer et al., 2009):

$$P_f = \int_{s=-\infty}^{s=\infty} \int_{r=-\infty}^{s} f_s(s) f_R(r) dr ds \tag{2.10}$$

Which can be written as:

$$P_f = \int_{s=-\infty}^{s=\infty} f_s(s) * \int_{r=-\infty}^{s} f_R(r) dr ds = \int_{s=-\infty}^{s=\infty} f_s(s) F_r(s) ds \tag{2.11}$$

In which:
$f_s(s)$ Probability density function of the load
$F_r(s)$ Cumulative distribution function of the resistance

In case the water level h_w is the only load parameter, the expression for the failure probability becomes:

$$P_f = \int_{s=-\infty}^{s=\infty} f_{h_w}(h_w) F_R(h_w) dh_w \tag{2.12}$$

In which:
$f_{h_w}(h_w)$ Probability density function of the water level

$F_R(h_w)$ Cumulative distribution function of the resistance given a certain water level (i.e. the fragility curve)

Concluding, the annual failure probability can be found by integrating the product of the fragility curve and the probability density function of the water level over the water levels. In Figure 2.11, an example of a fragility curve and the corresponding water level distribution are shown in the same figure.

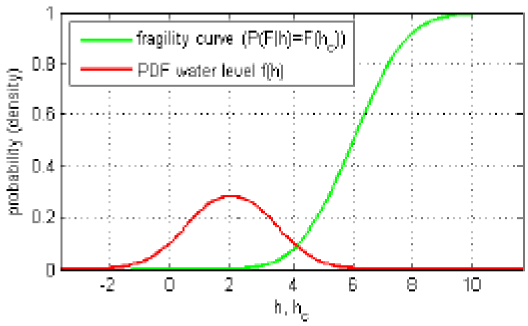


Figure 2.11: Illustration fragility curve and PDF of the water level distribution (Schweckendiek & Kanning, 2016)

Important to realize in the method of fragility curves, is that it takes all water levels into account in calculating the failure probability, instead of just using a design water level, as is the case in the semi-probabilistic method. Moreover, because for different water levels separate calculations are performed, a clear distinction can be made between water levels below the level of the foreshore and water levels above this level. This aspect is fundamental in this research, as will be explained later on.

As a last remark for this paragraph, (Schweckendiek & Kanning, 2016) states that instead of constructing fragility curves in probability plane, it could be wise to construct them in (β,h)-plane, i.e. use the reliability index instead of the failure probability. The reason for this is that, in Dutch experience, in (β,h)-plane, the fragility curve can be reasonably well approximated by linear interpolation between the calculated conditional failure probabilities. This linear interpolation is shown in Figure 2.12. After a complete (β,h)-curve is obtained using interpolation, this can easily be transformed into a fragility curve by using:

$$P_f = \varphi(-\beta) \tag{2.13}$$

In which φ is the cumulative standard normal distribution.

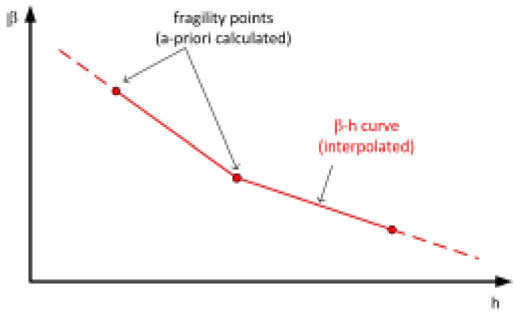


Figure 2.12: Fragility curve in (β,h)- plane, using linear interpolation between the calculated fragility points (Schweckendiek & Kanning, 2016)

2.4. Macro-Stability

2.4.1. Introduction

Until now, both flood protection in the Netherlands and probabilistic theory have been discussed in a general sense. However, in the research description (Chapter 1), it was already stated that this research will focus on the failure mechanism of macro-stability of the inner slope. This section will provide the necessary background information and theory on this mechanism, starting with a general description of the mechanism and going more and more into detail, covering the calculation models, the semi-probabilistic calculation method and the shear strength models.

The failure mechanism of macro-stability of the inner slope (inner slope stability) leads to a failure of a dike due to instability of the inner slope. Instability of the inner slope means that a large volume of soil slides off the dike following a straight or curved slip surface. Shallow slip surfaces (less than 1m) are not considered as macro-instability but are considered as micro-instability or failure of the dike cover. Macro-instability can also occur at the outer slope, but this falls outside the scope of this research. From now on, when macro-(in)stability is mentioned, this refers to a volume of soil slipping off the inner slope of the dike along a slip surface deeper than 1m ('t Hart, De Bruijn, & de Vries, 2016).

2.4.2. Mechanism

As the name of the mechanism, macro-(in)stability, already suggests, the failure is caused by a loss of stability (balance) of a soil mass along a slip plane. In a basic analysis of macro-stability, this balance consists of a driving moment, which is caused by the mass of soil at the left side of the centre point of a circular slip plane (side of the water), and a resisting moment, which is caused by the mass of soil at the right of the centre point (side of the land) and the shear stress along the slip plane. A rising water level causes an increase of pore water pressures in the soil, due to which the effective stresses in the soil decrease and the shear capacity also decreases, which causes loss of balance and instability of the soil. Note that for the macro-stability failure mechanism the weight of the soil is the actual load (driving moment), and not the high water level, as is the case for most other failure mechanisms ('t Hart et al., 2016). For macro-stability the water level is an initiator, as it increases the pore water pressures in the soil. The actual load, the weight of the soil, does not change, but the resistance does, which can lead to instability. Due to the decrease in shear strength of the soil, a zone may grow, in which the shear strength is exceeded, which leads to deformations and eventually sliding of the soil.

Note that an increase of pore water pressures is not only caused by high water levels, but can also be caused by heavy rainfall. However, such as situation does generally not lead to failure of the dike, in terms of flooding. After all, heavy rainfall is most of the time not accompanied by a high water level, which can flow over the dike.

In Figure 2.13 the basic principle of macro-instability is shown. (Jonkman & Schweckendiek, 2015) states that especially geotechnical failure mechanisms (piping and macro-stability) cause a high failure probability for river dikes. This indicates the importance of the failure mechanism of macro-stability.

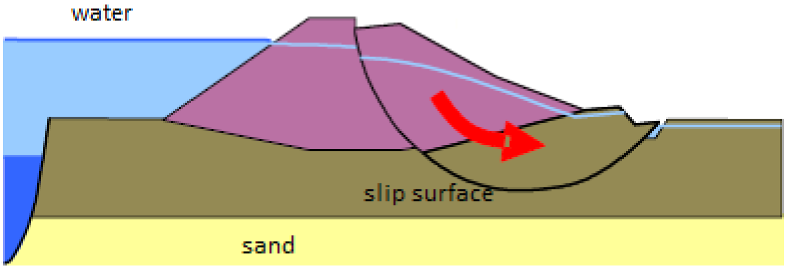


Figure 2.13: Basic principle of macro-instability ('t Hart et al., 2016) (converted to English)

Most of the time, the start of instability of the inner slope goes along with crack formation in the soil. Especially in soils which are sensitive to creep (in the so-called “Benedenrivierengebied” in the Netherlands), large cracks can form before actual sliding occurs. This means that if one notices a crack in time, there is still time to take emergency measures. If the soil starts to slide, the ground level at the land side of the crack decreases. Then, a volume of soil will slide, along the slip surface, until a new equilibrium situation is reached. At the equilibrium situation, the soil is stable again. However, the cover of the slope is gone and the core material will be on top. After the equilibrium situation is reached, other mechanisms can cause further damage to the dike.

Sometimes, macro-instability can lead to immediate failure (= flooding) of the dike. This is the case when the crest of the dike is lowered over its complete width after sliding of the soil. In that case, water can flow over the dike. However, in most cases, a part of the crest will remain intact. In those cases, macro-instability will not directly lead to failure of the dike. However, failure can still occur due to other mechanisms that will occur after the macro-instability. This is called progressive failure. The mechanisms that can cause progressive failure, after instability of the slope, are ('t Hart et al., 2016):

- A second volume of soil slides, which causes the remaining crest to be lowered;
- Micro-instability (in case of a sandy core), leakage of water through the core;
- Overtopping, which causes erosion to the core that is not anymore protected by a cover.

The complete pathway, from increasing pore pressures to complete failure, is illustrated in Figure 2.14. In the figure, a definition of failure is shown. Failure is defined as sliding of a soil volume along a slip surface that enters halfway the inner slope or further towards the water side. However, as explained above, this does often not immediately lead to actual failure (flooding) of the dike, as most of the time there is still a part of the crest that remains intact. Progressive failure (the mechanisms explained above) then needs to occur for complete failure to take place. Only when after progressive failure the height of the remaining dike profile is equal to or lower than the water level, and a breach is formed due to water flowing over the dike, one can speak of real failure of the dike. The situation between the definition of failure (assessed safety) and the real failure of the dike, is called residual strength, as the dike has actually more strength than is used for the assessment. The amount of residual strength depends on the width of the crest that is left after the first sliding of the soil, thus it depends on the critical slip surface.

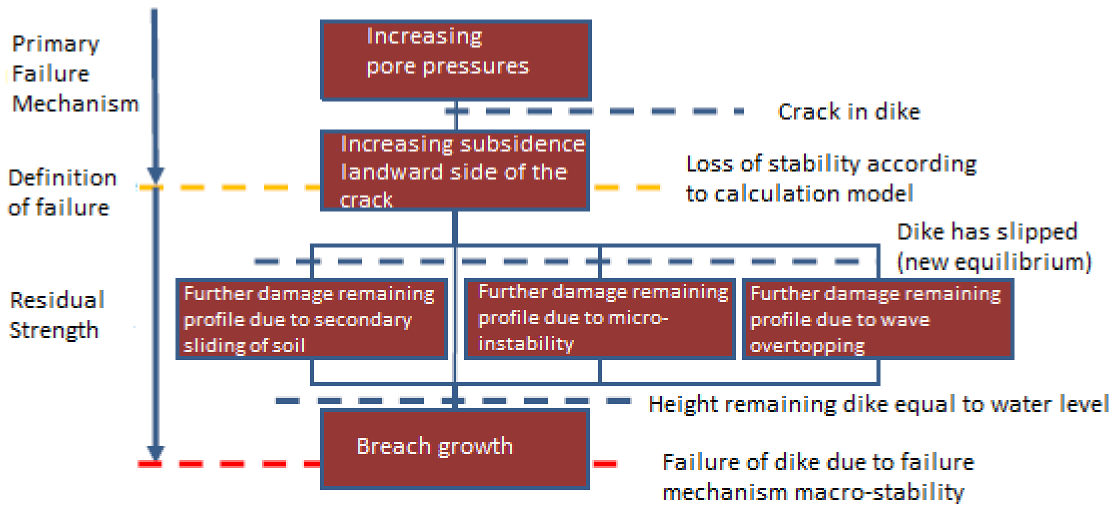


Figure 2.14: Pathway of failure inner slope stability ('t Hart et al., 2016) (translated to English)

2.4.3. Calculation Models

This section covers the different calculation models used for the calculation of the stability of dikes. The macro-stability of dikes can be assessed using LEM (Limit equilibrium method) calculations, which compare the loads to the maximum mobilizable resistance for moment and force equilibria. These calculations use a slip surface as boundary condition. By trying a lot of different slip surfaces, the critical slip surface is found, which is the slip surface that leads to the least stability.

In Figure 2.15 a basic soil balance along a circular slip plane is schematized. In this balance, the driving moment (load) is compared to the maximum resisting moment. The driving moment is calculated as follows (TAW, 1985):

$$M_a = a * Q \tag{2.14}$$

The maximum resisting moment is calculated as follows :

$$M_r = \sum \tau_m \Delta s r = \int_{-\theta 2}^{+\theta 1} \tau_m r^2 d\theta \tag{2.15}$$

In which τ_m is the maximum shear stress the soil can withstand. This maximum shear stress (or shear capacity) can be calculated using both drained analysis and undrained analysis. This will be covered later on. A factor of safety can be calculated as follows:

$$FoS = \frac{M_r}{M_a} \tag{2.16}$$

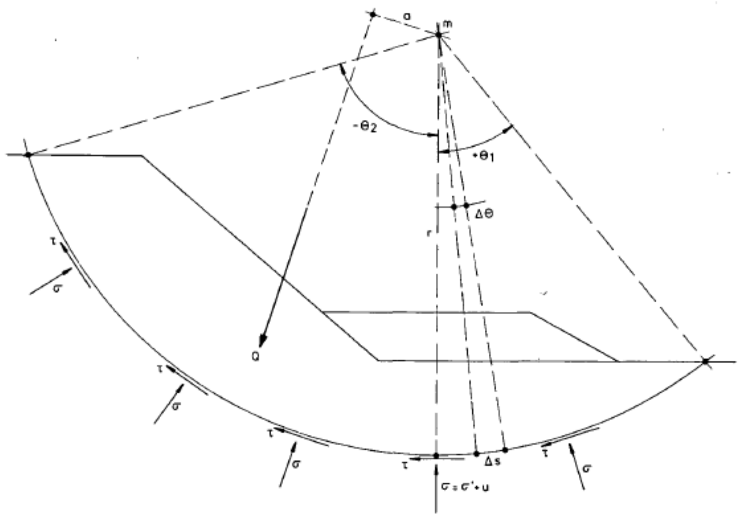


Figure 2.15: Basic schematization for the soil balance of the inner slope (TAW, 1985)

There are a couple of different LEM models, each with a corresponding slip surface shape and based on different assumptions. The mostly used LEM models are (Jonkman & Schweckendiek, 2016):

- **Bishop / Fellenius:** The Bishop method and the method by Fellenius check the moment equilibrium of a circular slip plane, as in the example above. The maximum resisting moment is calculated by dividing the soil into slices, calculating for each slice the maximum shear stress τ_m and calculating for every slice the contribution to the total maximum resisting moment. Besides the moment equilibrium, the methods of Bishop and Fellenius also ensure that the vertical force equilibrium is fulfilled, meaning that the vertical forces between the slices are in balance. The Bishop/Fellenius models are the most simple ones; they are easy to use and take only little calculation time.

- **Uplift Van:** In some conditions, the critical slip surface is not circular, which means it is not well represented by the Bishop/Fellenius model. This is the case in uplift conditions, in which due to increase of pore pressures, the effective stresses under the aquitard (impermeable top layer of the subsoil, consisting of clay/peat) are reduced to zero. The aquitard then “floats” on top of the aquifer (permeable sand layer underneath the aquitard, through which groundwater will flow) , with zero shear capacity at its bottom. The part of the aquitard that is lifted can then only transfer a horizontal force. In such conditions, the critical sliding surface is typically an elongated one.

  The Uplift Van model was created to deal with such conditions. It consists of a circular segment at the active side, a horizontal plane and a circular segment at the passive side. Like in the Bishop model, the soil is divided into slices. The slices in the horizontal plane are modelled such that the horizontal forces are transferred properly from the active to the passive side. Note that this is an advantage compared to the Bishop/Fellenius method, which does not fulfil the horizontal force equilibrium. In Figure 2.16 the slip surface shape for the Uplift Van model is shown.

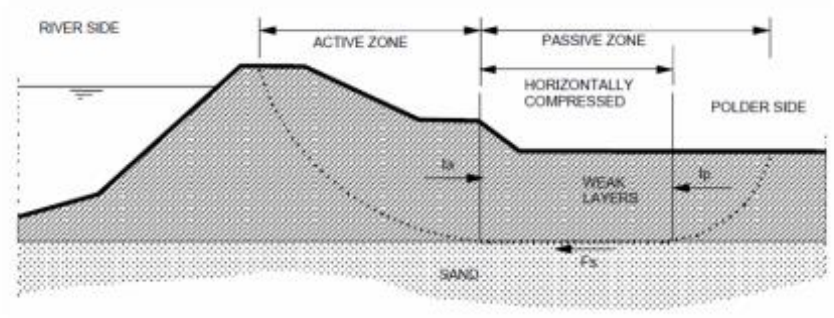


Figure 2.16: Uplift Van slip surface model (Jonkman & Schweckendiek, 2016)

- **Spencer:** The Spencer model is the most sophisticated LEM model, because its slip surface shape can be defined freely using interconnected linear elements. This means that all kinds of shapes can be defined: circular shapes as in the Bishop/Fellenius model, elongated shapes like in Uplift Van, or completely different shapes. Besides, like the Uplift Van model, Spencer fulfils all three equilibria (i.e. moments, horizontal forces and vertical forces). The Spencer model ensures that the critical slip surface that is found is actually the slip surface which leads to the lowest safety, because the slip surfaces are not limited to (elongated) circular shapes, but can take any shape. However, this model takes a larger calculation time than the other two models. Moreover, there is only little experience in using the Spencer model.

Besides the LEM models described until now, macro-stability can also be assessed using numerical models like FEM (Finite element models). These fall outside the scope of this study and will not be covered here.

2.4.4. Safety Assessment Macro-Stability

As mentioned before, the process for assessing the safety of a dike with respect to macro-stability is described in the WBI (Rijkswaterstaat, 2016a). The process starts with a simple safety check, in which the relevance of the failure mechanism is assessed based on simple geometric properties. If, based on this check, the failure mechanism is considered relevant, one should continue the assessment with the semi-probabilistic method, as mentioned earlier.

As mentioned in Section 2.2, the maximum allowed failure probability for a dike trajectory has to be divided between the different failure mechanisms, based on a mechanism budget, and between the different dike sections, based on the length-effect factor, leading to a *target failure probability* per cross-section (formula (2.3)). The proposed failure mechanism budget for macro-stability is 0.04, which means that 4% of the maximum failure probability on trajectory scale can be used for macro-stability. The length-effect factor for macro-stability is determined as follows (Rijkswaterstaat, 2016a):

$$N = 1 + \frac{\alpha_l * L_{trajectory}}{b_l} \tag{2.17}$$

In which:

α_l	Describes mainly the correlations between the dike sections for this mechanism, and has a value of 0.033 (Rijkswaterstaat, 2016b)
b_l	Representative length for the analysis in a cross-section, has a value of 50m (Rijkswaterstaat, 2016b)
$L_{trajectory}$	Length of the dike trajectory, as stated in the Dutch Water Act

As mentioned before, the semi-probabilistic calculation uses design values for all the parameters. The result of this calculation is a Factor of Safety, which can be transformed into an approximated failure probability using a calibrated formula. This formula is calibrated in such a way, that the approximated failure probability will in most of the cases be conservative, as will be explained in the next section. The approximated failure probability is compared to the target probability. If the approximated failure probability is smaller than the target probability, the cross-section is safe. Otherwise, one has three possibilities (Rijkswaterstaat, 2016a, 2016b):

a) Perform a customized assessment on cross-section scale, e.g. by performing a fully probabilistic calculation (without using the conservative approximation of the semi-probabilistic method) or by redefining the failure definition by taking into account residual strength.
b) Do not anymore take into account the failure mechanism budget and length-effect factor. Using probabilistic calculations, compute the failure probabilities for all dike sections and all mechanisms (possibly also taking into account residual strength) and combine these to a failure probability on dike trajectory scale, which can be compared to the maximum

allowed failure probability (safety standard). This is theoretically the right approach and uses no simplifications, but it is more complicated, as correlations between dike sections should be taken into account.

c) Reinforce the dike, e.g. by applying a stability berm

In this research, the focus will be on the option a), meaning that the research is performed on cross-section level, i.e. using a target probability, based on the mechanism budget and length-effect factor.

Note that the process proposed by the WBI looks like a coarse to fine method. At first, the semi-probabilistic method leads to an approximated failure probability which is conservative in most cases. If this probability does not fulfil the target probability, one can choose to put more effort in the assessment by performing fully probabilistic calculations, which are not based on a conservative approximation. By putting in more effort, one can reduce the resulting failure probability, and thereby possibly fulfil the target probability.

2.4.5. Semi-probabilistic assessment method

In this section, the semi-probabilistic assessment method that is currently used for the macro-stability mechanism will be described in more detail.

As mentioned before, the semi-probabilistic calculation is based on design values for all parameters. The design value of the water level is the water level with an exceedance frequency equal to the safety standard (maximum allowed failure probability) of the dike trajectory. For the other parameters, one wants to use values which are close to the design point (FORM, see Section 2.3.2) values. To obtain these values one first takes representative/characteristic values of the parameters, which are equal to the 5%-quantile (for resistance parameters) of the probability distributions. Partial safety factors are then used to bridge the gap between the characteristic values and the design values.

For the macro-stability mechanism, these safety factors have been determined in a calibration study (Kanning et al., 2016). In theory, every variable should have a partial safety factor which depends on the target reliability. However, the calibration study uses a simplified approach: for every variable a partial safety (material) factor is established, which is independent of the target reliability (β_T-invariant), and then one β_T-dependent safety factor is calibrated: the damage factor.

The calibration study concluded, based on FORM influence coefficients, that only for parameter S (undrained shear strength ratio) it was possible to assign a value other than 1.0 for the partial safety factor (material factor). All material factors are 1.0, and thus do not have any influence, except for parameter S, for which material factors 1.0 and 1.3 both are possible. However, it is chosen to use the material factor 1.0. This means that material factors do not have any influence and the β_T-dependent safety factor covers all uncertainties. The design values used in the semi-probabilistic calculation are therefore equal to the characteristic values.

The safety requirement for the semi-probabilistic method looks as follows:

$$\frac{FoS_{design}}{\gamma_d * \gamma_n} > 1 \tag{2.18}$$

In which:

FoS_{design}	Factor of safety for the stability of the dike (determined using the LEM models from Section 2.4.3, e.g. the ratio between driving moment and resisting moment for the Bishop model), based on design (characteristic) values for all the parameters [-]
γ_n	β_T-dependent damage factor [-]
γ_d	Model factor [-]

The model factor is used to take into account model uncertainties, and the value of the factor depends on which LEM model is used for the slip circle analysis. As mentioned before, there are three different slip surface models. Each model has its own corresponding safety factor, as shown in Table 2.1(Rijkswaterstaat, 2016b).

Slip circle model	**Model factor (γ_d)**
Bishop	1.11
LiftVan	1.06
Spencer- Van der Meij	1.07

Table 2.1: Model factors for the different LEM models

A calibration study was performed to determine γ_n (Kanning et al., 2016).The approach used for the calibration, is as follows. Different cases have been assessed both probabilistically and semi-probabilistically. The first results in a reliability index β. The second results in a FoS_{design} . It can be calculated what the value of the damage factor should be to fulfil the requirement of (2.18): $\gamma_n = \frac{FoS_{design}}{\gamma_d}$.

By doing this for all cases a (γ_n-β)- scatterplot is obtained. By fitting a line at the 20%-quantile (of β) of this plot, the following relation for γ_n has been established:

$$\gamma_n = 0.15 * \beta_{T,cross} + 0.41 \tag{2.19}$$

This plot and the fitted relation are shown in Figure 2.17. The calibrated damage factor leads to the following relation between the calculated factor of safety (the actual result of the semi-probabilistic calculation) and the approximated failure probability (Rijkswaterstaat, 2016a):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right) \tag{2.20}$$

From Figure 2.17, it immediately becomes clear that the result of the semi-probabilistic method is an approximated reliability (or failure probability). There is a 20% probability that the reliability is lower and a 80% probability that the reliability is higher than the calculated result. After all, the line has been fitted to the 20%-quantile of the reliabilities β in the scatter plot. Note that this means that in most of the cases the reliability of the dike is underestimated, and the failure probability is thus overestimated, which is conservative. Due to the big scatter in the plot, it can be a good choice to perform a fully probabilistic calculation, once a semi-probabilistic calculation has a negative result (Kanning et al., 2016).

Note that the calibration study as explained above was performed for normal dike cases, without (high and wide) foreshores.

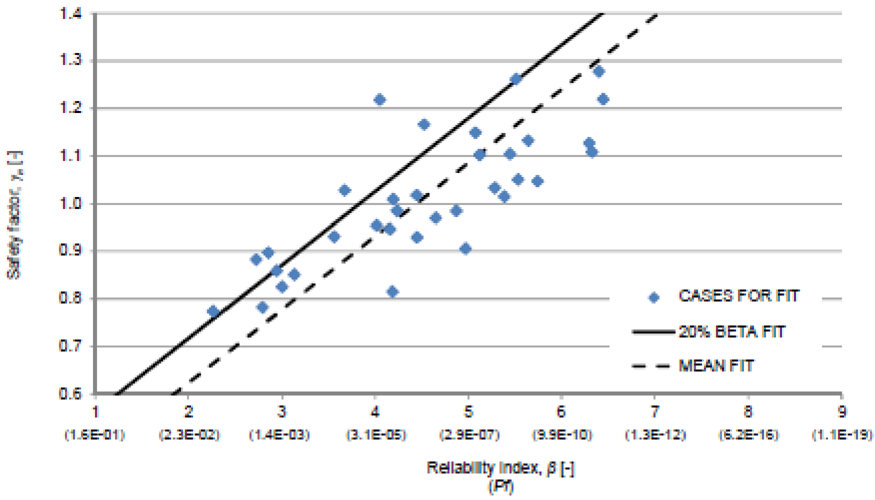


Figure 2.17: Scatter plot of the results of the calibration study (Kanning et al., 2016)

2.4.6. Shear strength models

Up to this point, the macro-stability mechanism, the calculation models and the assessment process have been discussed. In Section 2.4.3, it was already mentioned that calculation models use the shear strength of soils as a resistance against instability. This section covers the different models that are used to determine the shear strength.

In the old safety standards, the shear strength of the soil in the macro-stability analysis was calculated using drained analysis only. For this, the Mohr-Coulomb model was used, calculating the shear capacity as follows (Verruijt, 2001):

$$\tau = c' + \sigma_n * \tan(\varphi') \tag{2.21}$$

In which:

τ Ultimate shear stress [kN/m^2]
c' Cohesion [kN/m^2]
σ_n Normal stress on the slip surface [kN/m^2]
φ' Effective friction angle [°]

In the new standards, a new soil model is introduced: Critical State Soil Mechanics (CSSM) (Schofield & Wroth, 1968).This model distinguishes between peak strength and critical state strength, between normally consolidated behaviour and over-consolidated behaviour of the soil, and between drained and undrained behaviour. The critical state of the soil is a proper measure for the resistance against sliding of the soil. This critical state is the state in which there is no further change in volume or effective stress, so at the end of the stress path (Wroth, 1984). Figure 2.18 shows the basics of the CSSM model. The Critical State Line (CSL) gives the relation between the mean effective principal stress (s') and the shear stress at critical state (τ). The slope of the CSL is equal to sin(φ_{cs}), in which φ_{cs} is the critical state friction angle. The cohesion does not play a role in the CSL. In the CSSM model, cohesion is the result of over-consolidation. Cohesion only plays a role in the determination of the peak strength of over-consolidated soils. In an over-consolidated soil the yield stress (σ'_{vy}) is larger than the vertical effective stress (σ'_{vi}); the effective stress in the soil has been higher in the past then it is at that moment. If the soil is normally consolidated, the vertical effective stress is equal to the yield stress.

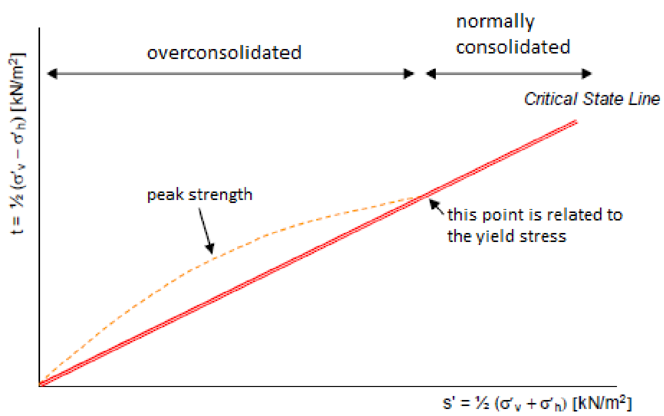


Figure 2.18: Illustration critical state line in the CSSM model (Rijkswaterstaat, 2016b)

An important consideration in stability calculations is the choice between drained and undrained analysis of the soil. Undrained analysis takes excess pore pressures into account, which are generated when the soil deforms and water cannot flow out of the soil. In practice, undrained analysis is performed for layers with low permeability, like clay and peat. For permeable layers, mostly consisting out of sand, drained analysis is used. Figure 2.19 shows typical stress paths for both drained and undrained behaviour. The difference u between the paths is the excess pore pressure, generated due to undrained behaviour of the soil. The figure shows that the excess pore pressures result in a decrease of the critical state shear strength; the shear stress (t) at the end of the stress path is lower for undrained behaviour than for drained behaviour. Excess pore pressures therefore lead to a strength reduction of the soil.

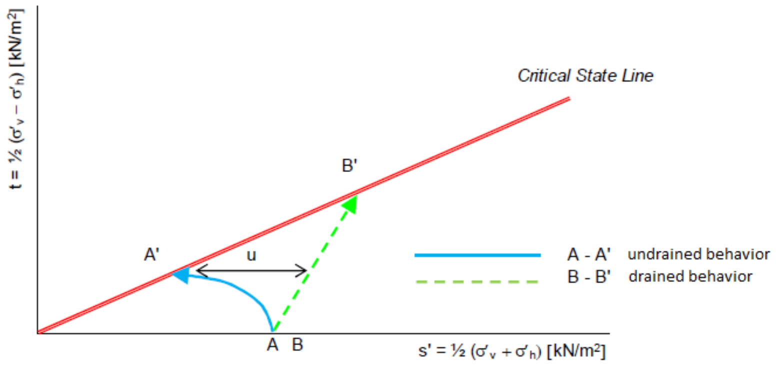


Figure 2.19: Illustration of stress paths for both drained and undrained behaviour in the CSSM model (Rijkswaterstaat, 2016b)

The shear strength of drained layers is calculated using the normal stress on the failure plain, like in the Mohr-Coulomb model (equation (2.21)). Note that in the CSSM model the cohesion is zero and the critical state friction angle φ_{cs} is used.

The shear stress of undrained layers is calculated using the SHANSEP model, and is determined as follows (Rijkswaterstaat, 2016b):

$$s_u = \sigma'_{v,i} * S * OCR^m \tag{2.22}$$

with $OCR = \frac{\sigma'_{vy}}{\sigma'_{v,i}}$ and $\sigma'_{vy} = \sigma'_{v,i} + POP$

In which:

s_u The undrained shear strength [kN/m²]
$\sigma'_{v,i}$ In situ effective vertical stress [kN/m²]
S The undrained shear strength ratio [-]
OCR Over consolidation ratio [-]
m Stress increase exponent [-]
σ'_{vy} Vertical yield stress [kN/m²]
POP Pre-overburden pressure [kN/m²]

2.5. Conclusion

The purpose of this chapter was to provide the reader with the necessary theory and background information for the research performed in this Thesis. Moreover, it was meant to give insight into the current safety assessment method for dikes.

In short, this assessment method is as follows. For every dike trajectory a maximum allowed failure probability has been defined. This maximum allowed probability is divided over the different failure mechanisms and dike sections in the trajectory, resulting into a *target probability* for every mechanism (see Figure 2.4). The safety assessment of the dike, for a certain failure mechanism, is a check whether or not the failure probability for that mechanism is below the target probability.

This Thesis focuses on the failure mechanism of *macro-stability*: a volume of soil sliding off the dike due to an increase in pore pressures. For this failure mechanism the WBI proposes a semi-probabilistic method to calculate the failure probability, which can be compared to the target probability. However, this semi-probabilistic method results in an approximated failure probability, which is based on a calibration study and is conservative in most cases. If this approximated probability is higher than the target probability, it may be wise to put in more effort and perform a more complicated fully probabilistic calculation, which is not based on a conservative approximation. Possibly, the result of the fully probabilistic calculation does fulfil the target probability. This approach can be seen as a coarse-to-fine method.

This answers the first research sub-question of this study: *"What method is currently used to assess the safety of dikes in general and specifically for macro-stability?"* Furthermore, different probabilistic calculation methods have been described and the physics and calculation models of the macro-stability mechanism have been covered in this chapter. In the next chapter, the theory will be applied to the specific case of *dikes with high and wide foreshores*, on which the focus of this Thesis will be.

3. High Foreshores

The theory covered in the previous chapter, will in this chapter be applied to the specific case of dikes with *high and wide foreshores*, on which this Thesis will focus. This chapter will answer the second sub-question: *"Which processes can lead to macro-stability failure of dikes with high and wide foreshores?"* This way, more insight is gained into the hypothesis that a dike with a high and wide foreshore is significantly safer with respect to macro-stability for water levels below the level of the foreshore than for higher water levels, as stated in Section 1.2.

As explained before, to assess the safety of a dike with respect to macro-stability, the failure probability for this mechanism is compared to the target probability. To determine the failure probability, the WBI proposes a semi-probabilistic method, which results in an approximated failure probability. An important simplification of this semi-probabilistic method is that it considers only one (design) water level, and does not take into account the contribution of lower water levels to the failure probability. If a dike would be significantly safer for lower water levels, not taking the contributions of these water levels into account would probably result in an overestimation of the failure probability. However, based on the calibration study (Kanning et al., 2016) and (POVM, 2015), it can be concluded that for most cases, the water level has only little influence on the safety of a dike (with respect to macro-stability), and that extreme rainfall is more important. This means that not taking into account the lower water levels in a semi-probabilistic approach would not lead to a significant overestimation of the failure probability.

However, in case a dike has a *high and wide foreshore*, this may be different. After all, the foreshore will most of the time not be flooded. Only for very high water levels, like the design water level, the foreshore will be flooded. This means that for water levels below the level of the foreshore the dike is significantly wider than for high water levels, which means that the dike is probably also safer. This hypothesis is also proposed by (POVM, 2015). To explain this, reference is made to Figure 3.1, in which a dike with a high and wide foreshore is shown, both for a (design) water level above the foreshore level, and a water level below the foreshore level.

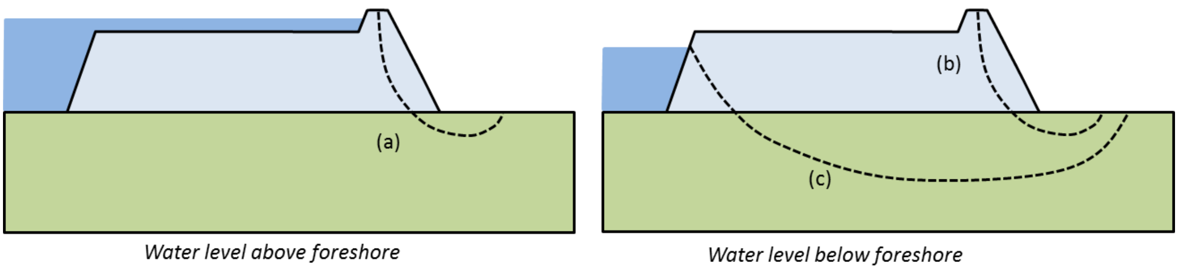


Figure 3.1: Possible slip surfaces in a dike with foreshore both for a water level above foreshore level and below

For the water level above the level of the foreshore, a possible slip surface (along which the soil will slide) is drawn (a). Probably, the critical slip surface will have a shape similar to (a), entering in the crest of the dike. One can expect that for water levels below the level of the foreshore, a similar slip surface will be critical: slip surface (b). After all, the only difference between both situations is that for the (design) water level above foreshore level, the phreatic line will be somewhat higher (depending on the permeability of the foreshore and the duration of the high water) and the aquifer head will be higher, leading to higher pore pressures in the dike body and the subsoil, and thus to some less stability, but probably not to a radical change in critical slip surface.

This is where the *failure definition* comes into play. As stated in Section 2.4.2, in most cases sliding of the soil does not lead to immediate failure of the dike, because often part of the crest remains intact, which is still able to retain the water. Therefore, the WBI states that for the macro-stability assessment, only the slip surfaces which are close to loss of function (failure) of the dike, should be taken into account. Loss of function of the dike means that the dike is no longer able to retain the water and a flooding takes place. In practice, the WBI proposes to take into account only the slip surfaces that enter halfway the inner slope or further to the water side, as illustrated in Figure 3.2. From now on, this is referred to as the *WBI failure definition*.

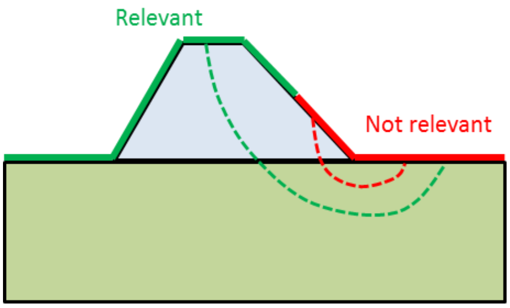


Figure 3.2: WBI failure definition: a slip surface entering in the green area is considered relevant (failure), one in the red area is not.

According to this WBI failure definition, both slip surfaces (a) and (b) in Figure 3.1 are relevant, and can thus be regarded as failure of the dike. For slip surface (a), sliding along this slip surface would indeed lead to a situation which is close to loss of function of the dike, because after the slide, only little part of the crest is intact to retain the water. However, for slip surface (b) this is different. In this case, the water level is below the level of the foreshore, so after sliding of the soil along slip surface (b), the complete foreshore and part of the dike crest are still intact and form a large body to retain the water. In other words: there still a wide dike left. From now on, this will be referred to as *residual width*. Slip surface (b) is therefore nowhere near loss of function of the dike. This means that for water levels below foreshore level, the WBI failure definition should not be used. Instead, the residual width has to be taken into account; one has to look for slip surfaces that are close to loss of function of the dike.

To reach loss of function of the dike for water levels below the level of the foreshore, a slip surface similar to (c) is needed, which has an entrance point far more to the water side and causes the complete foreshore to slide away (see Figure 3.1). A long and deep slip surface, like (c), generally goes along with a smaller probability of occurrence, than a more shallow slip surface like (a) and (b). This means that, taking into account residual width, the dike will be safer for water levels below the level of the foreshore than for higher water levels. After all, for water levels below foreshore level a long and deep slip surface (c) is needed to reach failure, whereas for higher water levels a shallower slip surface (a) suffices. As said before, this means that not taking into account these lower water levels (which lead to more safety) in a semi-probabilistic assessment will lead to an overestimation of the failure probability. This supports the hypothesis stated in Section 1.2.

However, slip surface (c) is not the only way loss of function of the dike can be reached for water levels below the level of the foreshore. Another possibility is *progressive failure*. This means that a shallow slip surface (b) is followed by another shallow slip surface, and possibly by a third and a fourth and so on, until such a large part of the foreshore has disappeared, that one can speak of loss of function and thus failure of the dike. This phenomenon is shown in Figure 3.3. As a shallow slip surface generally has a higher probability of occurrence than a long and deep slip surface (c), possibly the combination of all those successive shallow slip surfaces, leading to progressive failure, goes along with a higher probability than the probability of occurrence of just one deep and long sliding plane leading to failure.

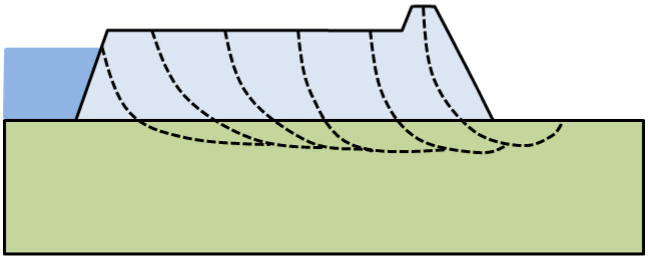


Figure 3.3: Illustration of progressive failure

Concluding the above, there are three scenarios of dike failure for dikes with high foreshores:

1. Failure of the dike due to a (shallow) slip surface in case the water level is higher than the foreshore level.
2. Failure of the dike due to a deep and long slip surface in case the water level is below the foreshore level.
3. Failure of the dike due to successive shallow slip surfaces in case the water level is below the foreshore level (i.e. progressive failure).

These scenarios are used to build a fault tree, which is shown in Figure 3.4. Note that in the safety assessment, for the water levels higher than the level of the foreshore, the *WBI failure definition* can be used (scenario 1). For water levels below the level of the foreshore, the *residual width* has to be taken into account, by looking for a long and deep slip surface (scenario 2) or by using progressive failure (scenario 3). Taking into account residual width will probably mean that the dike is significantly safer for water levels below foreshore than for higher water levels. Later on in this report, this will be validated by performing calculations.

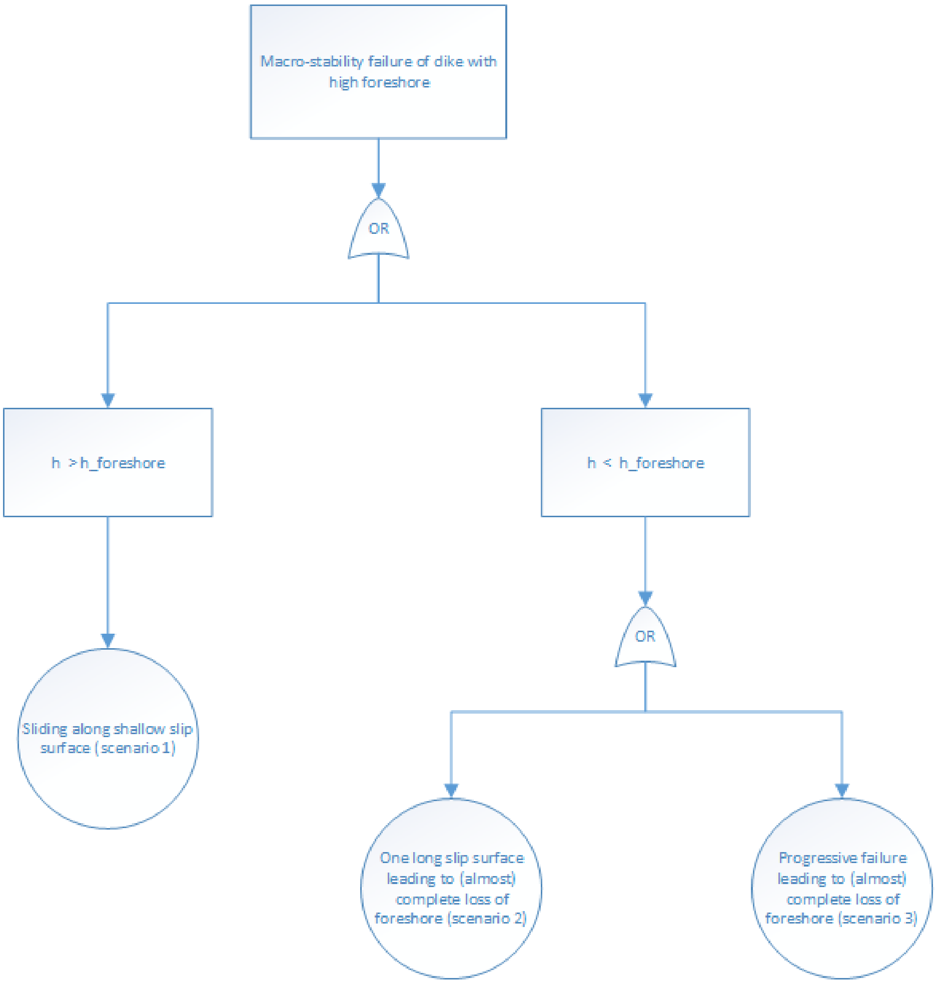


Figure 3.4: Fault tree for macro-stability of a dike with a high foreshore

Note:

In this section two definitions are used, which can be easily confused: stability and safety. Stability refers to the resistance against sliding of the inner slope of the dike; it considers the probability of any soil slide in land inwards direction. Safety refers to the resistance against failure of the dike in terms of flooding; it considers the probability of reaching a failure definition and therefore the probabilities of only the soil slides leading to a situation which is defined as failure. In other words: instability does not always lead to failure.
Reference is made to Figure 3.1. For the stability of the dike, the same slip surfaces are considered for water levels above foreshore level (a) and below foreshore level (b). The only difference for the stability is caused by the pore pressures, which are lower for water levels below foreshore level, leading to a little more stability. For the safety of the dike, a completely different slip surface (or progressive failure) has to be considered for water levels below foreshore (c). The difference in safety between the two water levels is thus not only caused by a difference in pore pressures, but also by the different slip surface shape. The difference in safety is therefore probably larger than the difference in stability.

4. Calculation Set-Up

4.1. Introduction

To be able to answer the remaining sub-questions of this Thesis, calculations have to be performed on different theoretical test cases. This chapter describes the different calculations that will be performed, the test cases that will be used, and the schematization choices used for the calculations. Besides, the assumptions and limitations of the different calculation methods will be discussed.

4.2. Calculation Matrix

In this Thesis, different calculations will be performed. The failure probabilities of different test cases will be calculated using both the semi-probabilistic method (see Section 2.4.5) and fully probabilistic calculations. And both cases with foreshore and cases without foreshore will be used. The results of these different cases and different calculation methods will be compared to each other.

To give a clear overview of the different calculations that will be performed in the remainder of this report, a calculation matrix is shown in Figure 4.1. Each cell shows an illustration of the calculation it represents: a sketch of the dike geometry, including the water level distribution and the water levels that are taken into account for the calculation, and a possible slip surface. Besides, it is shown which slip surfaces are taken into account for the calculation (failure definition): slip surfaces with an entrance point in the green area are considered relevant for that cell (calculation), those with an entrance point in the red area are not.

The cells of the matrix are described below:

1) **No foreshore, Semi-Probabilistic:** The failure probability of a test case without foreshore is calculated using the semi-probabilistic method. Only one water level (design water level) is taken into account for the calculation. The WBI failure definition will be used, taking into account all slip surfaces with an entrance point halfway the inner slope or further to the water side (shown in green).
2) **No foreshore, Probabilistic:** The failure probability of a test case without foreshore is calculated using fully probabilistic calculations. The complete water level distribution is taken into account for the calculation. The WBI failure definition will be used.
3) **Foreshore, Semi-Probabilistic:** The failure probability of a test case with foreshore is calculated using the semi-probabilistic method. Only one water level (design water level) is taken into account for the calculation. The WBI failure definition will be used (because the design water level is above the level of the foreshore).
4) **Foreshore, Probabilistic:** The failure probability of a test case with foreshore is calculated using fully probabilistic calculations. The calculations are based on the three scenarios of

Chapter 3. For water levels above foreshore level, the WBI failure definition will be used (1). For water levels below foreshore level, the residual width is taken into account by looking at a long and deep sliding plane (2) or by analysing progressive failure (3) (See Figure 4.1).
One could also choose to perform the calculations of this cell without taking into account residual width, and just using the WBI failure definition for all water levels (this is not shown in the matrix). However, this will probably lead to overestimation of the failure probability. This will be further studied in Chapter 6.

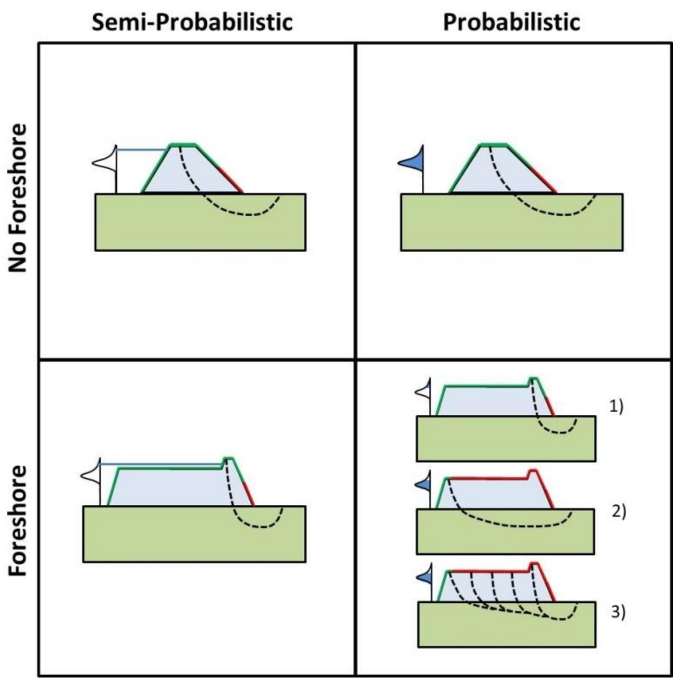


Figure 4.1: Calculation matrix showing the different calculations performed in this Thesis

In the next section, the test cases, on which the calculations above are performed, will be introduced.

4.3. Test Cases

For this study different theoretical test cases have been created. The basis for these test cases is shown in Figure 4.2. Each case consists of a dike and a foreshore, both out of the same material. The subsoil consists of an aquitard out of clay on top of an aquifer out of sand with a thickness of 10m. The foreshore has a width of 100m and the dike crest a width of 5m. The outer slope of the foreshore and of the dike both are at 1:3 and the inner slope is at 1:4. In all cases, the surface level is at 0m+NAP.

Note: For the cases without foreshore (see Figure 4.1), the foreshore in Figure 4.2 is absent and the geometry follows the dashed line.

The material of the dike and foreshore, the thickness and strength parameter S (undrained shear strength ratio, see Section 2.4.6) of the aquitard, and the height of the foreshore and the dike crest are varied in the different Test Cases. The height of the foreshore and the dike crest are determined based on a water level distribution. It is chosen to base the test cases on a Gumbel distributed water level with parameters u = 3 and α = 4.5.

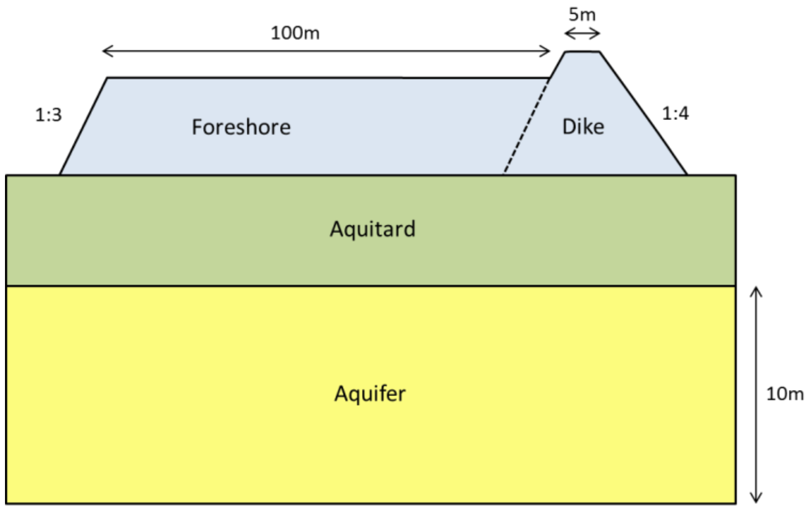


Figure 4.2: Standard case used as basis for all test cases

The different Test Cases are shown in Figure 4.3. In short, they can be described as follows:

- **Test Case 1:** The dike and the foreshore consist out of sand. The aquitard is a weak clay layer with a mean value of the undrained shear strength ratio S = 0.25 and a thickness of 5m. The level of the foreshore is at 4.02m+NAP, based on the mentioned water level distribution and a probability of flooding of the foreshore of 1/100. The level of the dike crest is at 5.54m+NAP, based on a design water level with an exceedance frequency of 1/10,000 (5.04m+NAP) and a freeboard of 0.5m. The daily water level is at 2m+NAP and the polder level is at 0m+NAP (inner surface level).
- **Test Case 2:** The geometry of this case is equal to Test Case 1. The differences are the material of the foreshore and dike, which are out of clay, and the strength of the clay aquitard, for which S = 0.32.
- **Test Case 3:** The basis of this case is equal to Test Case 1. The thickness of the aquitard has been reduced to 2.5m, to create conditions in which uplift of the aquitard becomes important. Furthermore, the strength of the aquitard has been increased to S = 0.32.
- **Test Case 4:** The basis of this case is equal to Test Case 1. The strength of the aquitard has been increased to S = 0.32.
- **Test Case 5:** The basis of this case is equal to Test Case 2. The strength of the aquitard has been increased to S = 0.35.
- **Test Case 6:** The basis of this case is equal to Test Case 4. The height difference between the foreshore and the crest has been reduced. The probability of flooding of the foreshore is equal to 1/1,000 and the exceedance frequency of the design water level is 1/3,000.

- **Test Case 7:** The basis of this case is equal to Test Case 5. Again, the height difference between foreshore and crest have been reduced, and the corresponding probabilities are the same as in Test Case 6.

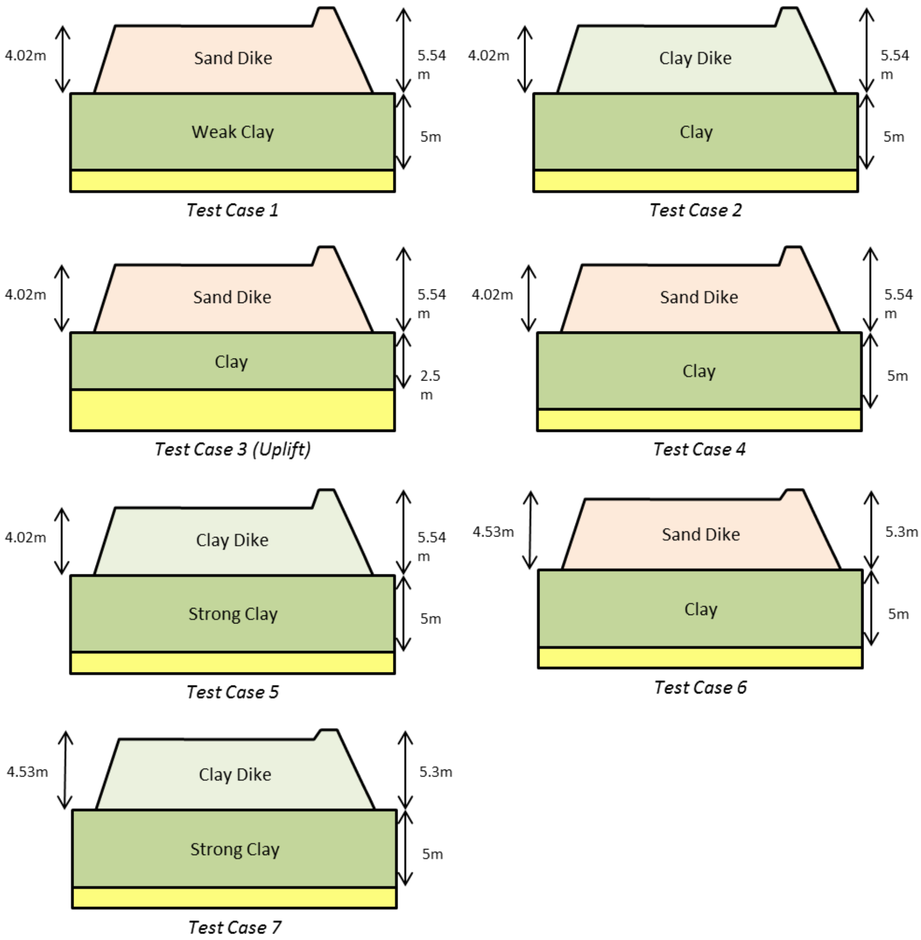


Figure 4.3: Overview of the test cases considered in this Thesis

Note: In all test cases as shown in Figure 4.3 a foreshore is present. The same test cases will also be used without foreshore, as mentioned before. Besides the absence of the foreshore, these cases are the same as shown in the figure.

4.4. Water level distributions

In the previous section, the different test cases have been described. The height of the foreshore and the height of the dike crest in these test cases were based on an assumed water level distribution (Gumbel distributed with u = 3 and α = 4.5, see Section 4.3), which is also used in determining the total failure probability from the fragility curve in the fully probabilistic calculation.

Note that in the different test cases, a lot of variables have been varied. However, one important variable has not been varied: the water level distribution. To study the influence of this distribution, all calculations are performed twice: once using the basic water level distribution (α = 4.5) and once using a water level distribution with reduced spreading (α = 7.5). Both are Gumbel distributed and have location parameter u =3. The heights of the foreshore and of the dike crest, which are based on the basic water level distribution, are kept the same (as shown in Figure 4.3).

Reducing the spreading, means that higher water levels get a lower probability of occurrence. This is illustrated in Figure 4.4, which shows a fragility curve for macro-stability, together with both water level distributions (α = 4.5 in black, α = 7.5 in red). Reducing the spreading changes the results of both semi-probabilistic calculations and fully probabilistic calculations:

- The semi-probabilistic method is based on a design water level. This design water level has a fixed exceedance frequency (which is equal to the safety standard of the dike). For a water level distribution with less spreading (but the same mean value), this fixed exceedance frequency corresponds to a lower design water level, probably leading to a higher FoS in the semi-probabilistic method and thus to a lower failure probability.
- In the fully probabilistic calculation the fragility curve does not change, as the water levels are put in as deterministic values. However, the total failure probability is obtained by integration of the product of the fragility curve and the water level distribution over the water levels. For a water level distribution with less spreading (but the same mean value), this total probability decreases.

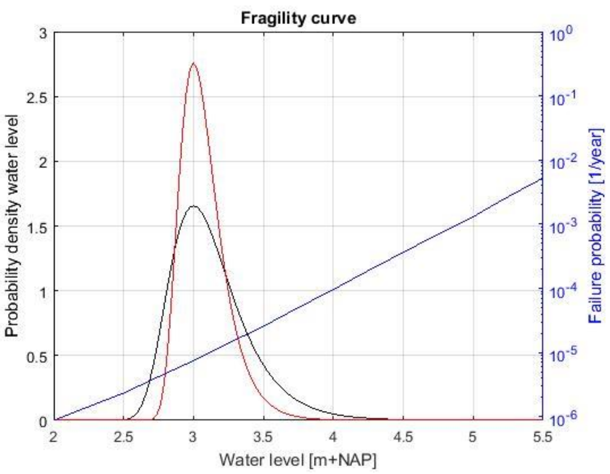


Figure 4.4: Illustration of a fragility curve combined with different water level distributions

4.5. Schematization and Calculation choices

The previous sections described the different calculations that will be performed in this Thesis and the test cases and water level distributions that will be used for these calculations. This section covers the calculation choices that are made for the calculations, like the software and the models that will be used, the schematization choices, and specific choices for the semi-probabilistic, fully probabilistic, and progressive failure calculations.

Only a brief overview of the choices will be given here. The complete description of all choices can be found in Appendix B.

4.5.1. Software & Model

The calculations in this study are performed using the Beta version of the D-GeoStability software in combination with the Probabilistic Toolkit of Deltares. Within this software the WBI calculation kernel is used.

The slip surface calculations are performed using the UpliftVan model, because long sliding planes are needed to reach failure in case the water level is below foreshore level (as described in Chapter 3), and the elongated slip surface shape of the Uplift Van model is best suited for this. Besides, Uplift Van model is the standard used model in WBI 2017, and this model was also used in the calibration study mentioned in Section 2.4.5.

4.5.2. Schematization

Figure 4.5 shows an example of the schematization of a dike for the macro-stability mechanism, with the different parameters that play a role. For the macro-stability mechanism, the parameters that play a role can be divided into two groups.

On the one hand there are the soil parameters, which determine the strength of the soil layers. As mentioned before, all test cases consist of three soil layers: the dike (and foreshore) body, a clay aquitard, and an aquifer consisting of Pleistocene sand. Figure 4.5 shows for the different layers which parameters are used to determine the strength of the layer. The unit weight of the material (γ), plays a role in both the strength of the dike and the load on the dike. Note that for the sand aquifer drained analysis is used and for the for the clay aquitard undrained analysis (see Section 2.4.6). For the dike and foreshore body, the choice between drained and undrained analysis depends on the material.

On the other hand, there are the parameters that determine the pore pressures inside the dike (the waternet). This waternet (including the phreatic line) is automatically created in the D-GeoStability software by using the Waternet Creator, based on the parameters shown in Figure 4.5. The pore pressures in the soil act as an initiator for macro-stability: the higher the pore pressures, the lower the shear strength of the soil and the lower the stability of the dike.

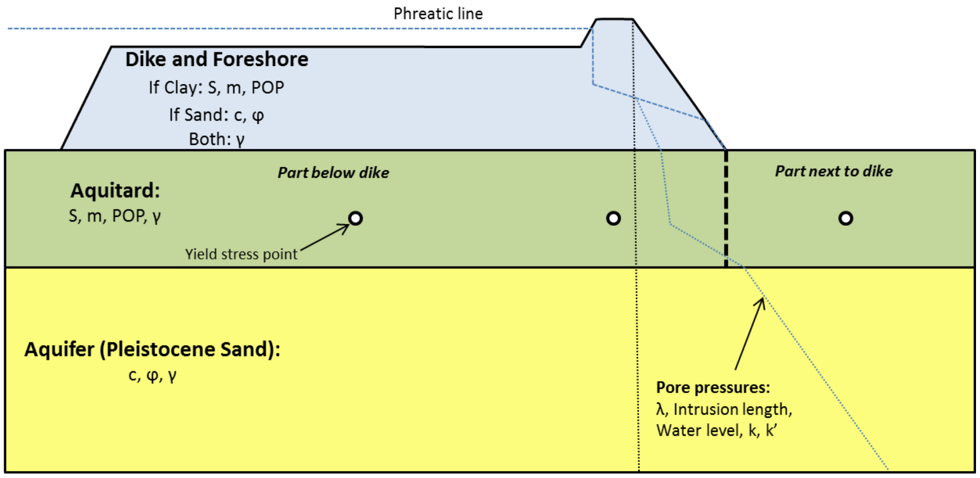


Figure 4.5: Overview schematization for the macro-stability assessment, showing the parameters that play a role

The shear strength of the aquitard is determined based on the SHANSEP model using yield stresses (Section 2.4.6), which are a measure of the stress history of the soil. Figure 4.5 shows the positions of the yield stress points in the schematization: one below the foreshore, one below the dike crest and one below the hinterland. Besides, Figure 4.5 shows that the aquitard layer is split up into a part next to the dike and a part below the dike. Generally, these parts have a different stress history. To schematize this in a right way, they are modelled as separate layers in the D-GeoStability software.

The values of the soil parameters that are used in the calculations are, for all test cases, specified in Appendix A. The schematization and the meaning of the different parameters are described in more detail in Appendix B.

4.5.3. Fully probabilistic vs. Semi-probabilistic

As mentioned in Section 4.2, in this Thesis both fully probabilistic and semi-probabilistic calculations will be performed. Both calculations use the schematization as described above in a different way.

The fully probabilistic calculations will be performed using fragility curves (see Section 2.3.3): for a range of water levels the conditional failure probabilities will be computed using a FORM calculation (see Section 2.3.2). This fragility curve then leads, combined with the water level distribution, to the total failure probability. The fully probabilistic calculation uses the complete probability distributions for the water level, the strength parameters of the soil, and for λ and the intrusion length (Figure 4.5). Besides, correlations (measures of mutual dependence) between different variables have to be specified (see Appendix B).

While the fully probabilistic calculation takes a lot of calculation steps, as the conditional probabilities are computed for a range of water levels, and for each water level iteration is needed in the FORM calculation, the semi-probabilistic calculation takes only one calculation step. It is a simplified approach. Instead of using the complete probability distributions of the parameters, characteristic values are used, which are the values corresponding to a certain quantile of the distribution. The quantiles to be used can be found in Appendix B. Besides, instead of using the complete water level distribution, only one design water level is used. The single calculation step leads to a factor of safety (FoS), computed in the D-GeoStability software, which is transformed into an approximated failure probability using formula (2.20):

$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right)$, in which the model factor is γ_d = 1.06 for the Uplift Van slip surface model (see Section 2.4.5).

4.5.4. Progressive failure analysis

In the progressive failure analysis, the probability of failure of a dike due to successive soil slides is studied. This analysis refers to scenario 3 of Chapter 3. For this analysis, more calculation and schematization choices are needed than for the other calculations. Therefore, they are described separately in this section. Note that this section only gives a brief description of the choices that are made. The choices are described in detail in Appendix B.

In the progressive failure analysis, the individual probabilities of the successive soil slides are calculated. After every single slide, a new remaining profile has to be schematized. For every slide, this remaining profile gets a more gentle inner slope, and will therefore become more stable. This means that the probability of occurrence a new individual slide decreases with every successive soil slide.

For failure of the dike to occur, the successive soil slides must lead to a situation in which there is almost no foreshore left to retain the water. In this Thesis, a situation in which the remaining width is less than 3m is defined as failure. The remaining width is defined in Appendix B. Based on the individual probabilities of the successive soil slides, the total failure probability can be determined. Progressive failure can be seen as a parallel system: all soil slides have to occur before failure occurs. The total failure probability is assumed to be equal to the minimum individual probability of all soil slides, which is a conservative choice. This is described in Appendix B. As the probability of occurrence of a soil slide decreases with every slide, the total probability of a combination of slides is generally equal to the individual probability of the last slide.

For simplicity reasons, the progressive failure analysis is performed using only one water level, the highest water level for which the foreshore is not flooded. It is assumed that the conditional failure probability that is found for this water level, is also the conditional failure probability for the other water levels below foreshore level. This is a conservative assumption, because in reality the failure probability decreases for decreasing water levels.

For the analysis, a stop criterion is determined, equal to β = 6.44. Once the reliability of a new individual soil slide exceeds this value (i.e. the failure probability becomes lower than $6.0*10^{-11}$), the probability of progressive failure can be considered negligible and the analysis can be stopped.

For the remaining profile and the slip surfaces that are taken into account in the analysis, some assumptions have been made. These are described in Appendix B. The next section describes the effect these assumptions have on the results.

4.6. Limitations

This section discusses the assumptions that are used in the different calculation methods and their limitations. In reality a dike has a certain failure probability. For now, this is defined as the *real failure probability*. Important to realize is that in all the calculations assumptions are made. This means that none of the calculation methods calculates the exact real failure probability; they all lead to estimations. However, some of the methods resemble the reality better than others, and therefore lead to results which are closer to the real failure probability.

This section discusses for the different calculations methods to which degree they resemble the reality, based on the assumptions and simplifications that are used.

4.6.1. Limitations Semi-Probabilistic calculations

The semi-probabilistic method is based on a lot of simplifications. As mentioned before, the calculation is performed using design values for all parameters, instead of using the complete probability distributions. Besides, the result of this calculation is a factor of safety (FoS). Based on a calibrated relation (β-γ_n), this FoS is used to approximate the failure probability. Note that this is an important limitation of the semi-probabilistic method.

Besides, as described in Appendix B, the phreatic line in the dikes is modelled using standard schematizations, which are conservative estimates. The time-dependence of the phreatic line development is not taken into account. For dikes without foreshores, this has probably only little effect on the result. For dikes with foreshores however, the effect is probably bigger. After all, it takes time for the foreshore to get completely saturated, depending on the permeability of the foreshore. It is therefore possible that a high water level situation does not last long enough for the complete foreshore body to get saturated. By using the standard phreatic line schematizations this fact is neglected and the foreshore is assumed to be completely saturated as soon as the water level is above foreshore level. For the design water level, which is (almost always) above foreshore level, the foreshore is thus assumed to be completely saturated. This simplification may lead to overestimations of the failure probability.

4.6.2. Limitations Probabilistic calculations

The probabilistic calculations are performed using the complete probability distributions for all parameters. This way, the complete uncertainties in the parameter values are incorporated in the calculations. Besides, the result of a probabilistic calculation is a failure probability. There is no calibrated relation needed for this. Therefore, a fully probabilistic calculation better resembles the reality than the semi-probabilistic method. However, simplifications and assumptions are made for these calculations too.

First of all, the standard phreatic line schematizations as described above are used. For water levels above foreshore level, this may therefore lead to overestimations of the conditional failure probabilities, because the foreshore is assumed to be completely saturated.

Besides, in reality, a lot of different slip surfaces are possible, each with a certain probability. The total failure probability is the combination of all those slip surfaces, with the corresponding probabilities, taking into account the correlations between different slip surfaces. However, in the probabilistic calculation, the failure probability is calculated using only the critical slip surface (the slip surface that leads to the lowest FoS using design values for the parameters). The failure probability is assumed to be equal to the probability of occurrence of this critical slip surface (see Appendix B). (Schweckendiek, Krogt, Rijneveld, & Teixeira, 2017) states that this is a reasonable approximation, because different slip surfaces close to each other are (almost) fully correlated and completely different slip surfaces differ largely in failure probability, and summing up smaller probabilities has only little influence on the total failure probability. The combined probability of all possible slip surfaces will therefore probably not differ a lot from the probability of the critical slip surface. However, it is still a simplification of the reality.

4.6.3. Limitations Progressive Failure analysis

To take into account the residual width for water levels below foreshore level, a progressive failure analysis is performed, as described before. This analysis is based on a lot of assumptions. The most important assumptions made for the analysis are:

- There is an infinite amount of pathways leading to failure; for every individual soil slide, a lot of slip surfaces are possible. In the analysis this is accounted for by combining two types of slip surface shapes (see Appendix B) in a conservative way. However, note that a lot of possible slip surfaces are neglected, so it is uncertain whether the combination of the two types of slip surfaces is really conservative, and to what extent;
- The shape of the remaining profile is after every individual soil slide assumed based on reductions of the dike height of 1m to 2m, which are mostly found in practice ('t Hart et al., 2016; ENW, 2009). However, the shape of the remaining profile is highly uncertain, and depends, besides on the slip surface, on dike material and strength.
- Brittle behaviour in the subsoil is assumed after a soil slide takes place, causing a reduction in the shear strength: the undrained shear strength ratio S in the subsoil is locally reduced to a mean value of 0.15, and the standard deviation is doubled. However, it is uncertain whether in reality the strength is reduced, and to what extent;
- The total conditional failure probability for progressive failure for a certain water level is assumed to be equal to the minimum probability of the individual soil slides. In reality, this is the upper limit of the total failure probability.

- For the analysis, only one water level was used, which is equal to the foreshore level. It is assumed that the conditional failure probabilities for the other water levels below foreshore are equal to the conditional failure probability of the used water level.

More information about the assumptions described above can be found in Appendix B. The last two assumptions are known to be conservative. The other three are conservative to the best of the available knowledge. However, there is a lot of uncertainty in these three assumptions, and they influence each other and the result to a large extent. For example, the slip surface shape determines for a large part the remaining profile, which determines for a large part the next slip surface shape, and so on. Neglecting a lot of possible slip surfaces therefore also means that a lot of possible remaining profiles are neglected. As the progressive failure analysis is not the primary goal of this Thesis, the assumptions are considered to be acceptable. However, it is highly recommended to perform further research into the behaviour of the soil after a soil slide, to determine whether a strength reduction occurs and to what extent. Furthermore, it is recommended to establish a better method to take all possible pathways leading to failure into account, without neglecting a lot of possible slip surfaces.

4.6.4. Limitations failure definition

In all the calculations, the probability of reaching some failure definition is calculated. However, in reality, reaching this failure definition will not always lead to failure in terms of flooding of the dike. The failure definitions describe a situation in which it can be expected that a flooding may occur. In other words: the failure definition resembles the real failure to some extent. In some cases this definition is closer to real failure than in others.

For example, as mentioned before, a dike with foreshore has a lot of residual width for water levels below the level of the foreshore. The WBI failure definition then considers relatively small slip surfaces as failure, as it neglects the residual width. By taking this residual width into account, and thus altering the failure definition, one obtains a failure probability which better resembles real failure.

4.7. Schematization Example

This section introduces one of the test cases, Test Case 4, which will be used as an example throughout this report. In this section, the schematization of the geometry and the parameters of this case are described.

4.7.1. Geometry

Figure 4.6 shows the geometry of Test Case 4 with foreshore, and Figure 4.7 shows the geometry without foreshore, as schematized in the D-GeoStability software.

It concerns a sand dike (and a sandy foreshore) on top of a clay layer, with an aquifer (sand layer) at -5m+NAP. Both the inner and outer surface levels are at 0m+NAP. The outer slope of the foreshore and the outer slope of the dike are at 1:3. The inner slope of the dike is at 1:4. The thickness of the aquifer (Sand, Pleistocene) is 10m. The foreshore is 100m wide and the crest is 5m wide.

The level of the foreshore is at 4.02m+NAP and the dike crest is at 5.54m+NAP. The design water level is 5.04m+NAP. The river water level during daily conditions is 2m+NAP and the polder level is 0m+NAP.

Note: Clay,B and Clay,N, are “Clay below the dike” and “Clay next to the dike” respectively. These are two parts of the same layer, as described in Section 4.5.2.

The only difference between the two schematizations of Figure 4.6 and Figure 4.7 is, apart from the presence/absence of the foreshore, that the most left yield stress point has disappeared for the case without foreshore. This yield stress point was used to model the stress history below the foreshore, and is therefore not needed in the case without foreshore.

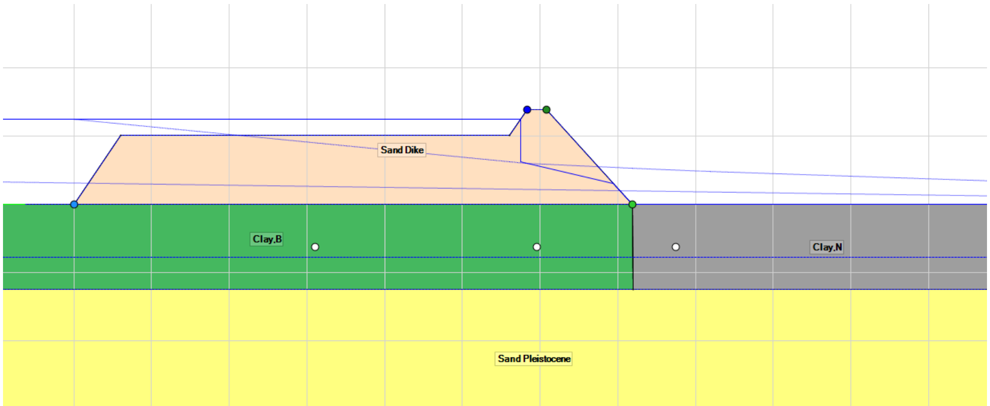


Figure 4.6 Dike geometry of Test Case 4 with foreshore as schematized in D-GeoStability

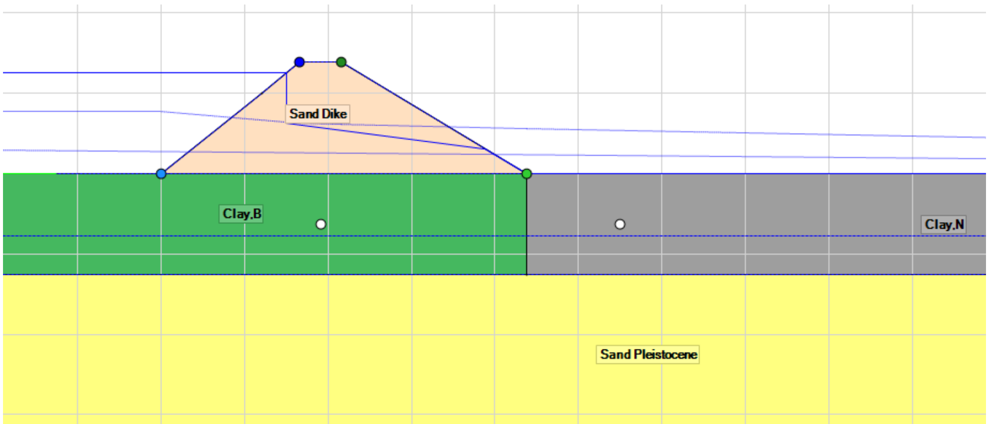


Figure 4.7: Dike geometry of Test Case 4 without foreshore as schematized in D-GeoStability

4.7.2. Parameters

For Test Case 4, the soil parameters are shown in Table 4.1. For every parameter, the distribution type, mean, variation and the design value (used in the semi-probabilistic calculation) are shown. The design values are based on certain quantiles of the probabilistic distributions of the parameters. These quantiles are specified in Appendix B.

Parameter	Distribution type	Mean value	Variation	Design value
Sand Dike				
$\gamma_{dry}/\gamma_{wet}$ [kN/m^3]	Deterministic	18/20	-	18/20
c' [kN/m^2]	Deterministic	0	-	0
φ [°]	Lognormal	34	CoV = 0.05	31.278
Sand Pleistocene				
$\gamma_{dry}/\gamma_{wet}$ [kN/m^3]	Deterministic	18/20	-	18/20
c' [kN/m^2]	Deterministic	0	-	0
φ [°]	Lognormal	34	CoV = 0.05	31.278
Clay, B				
$\gamma_{dry}/\gamma_{wet}$ [kN/m^3]	Deterministic	15/15	-	15/15
S [-]	Lognormal	0.32	σ = 0.03	0.273
m [-]	Lognormal	0.9	CoV = 0.03	0.856
POP	Lognormal	25	σ = 7.5	14.775
Clay, N				
$\gamma_{dry}/\gamma_{wet}$ [kN/m^3]	Deterministic	15/15	-	15/15
S [-]	Lognormal	0.32	σ = 0.03	0.273
m [-]	Lognormal	0.9	CoV = 0.03	0.856
POP	Lognormal	25	σ = 7.5	14.775

Table 4.1: Soil parameters Test Case 4

As explained in Appendix B, the POP values are not used in the calculation, but are used to compute the yield stresses. The yield stress points are shown as the white dots in Figure 4.6. The yield stress values are calculated by adding the POP values to the vertical effective stresses (during daily conditions) in the yield stress points. The yield stress distributions are shown in Table 4.2. The distributions have the same variation as the POP values, and a shift equal to the value of the effective stress (see Appendix B). Also, the design values as used in the semi-probabilistic calculation are shown.

X-coordinate	Distribution type	Mean value	Variation	Shift	Design value
82.06	Lognormal	101.86	σ = 7.5	76.86	91.635
139.12	Lognormal	133.22	σ = 7.5	108.22	122.99
175	Lognormal	37.5	σ = 7.5	12.5	27.275

Table 4.2: Yield stresses Test Case 4

The values used for the Waternet Creator parameters are shown in Table 4.3. The hydraulic conductivity of the Pleistocene sand layer (aquifer) is assumed to be $1.0*10^{-4}$m/s and the hydraulic conductivity of the clay layer (aquitard) is assumed to be $1.0*10^{-7}$m/s. These values are used to calculate the mean values of the leakage lengths. For an aquitard consisting of clay, (Rijkswaterstaat, 2016b) proposes a value for the intrusion length of 2m, for a long lasting high water (20 days). For a short lasting high water (5 days), a value of 1m is proposed. A larger intrusion length leads (most of the time) to less stability, therefore the higher value of 2m is chosen as mean, which is a little conservative choice.

Parameter	Distribution type	Mean value	Variation	Design value
λ_{in} [m]	Lognormal	223.6	CoV = 0.2	219.26
λ_{out} [m]	Lognormal	223.6	CoV = 0.2	219.26
Intrusion length [m]	Lognormal	2	CoV = 0.3	1.916

Table 4.3: Parameters used for input Waternet Creator Test Case 4

The schematization of Test Case 4 has now been covered. In the remaining chapters, Test Case 4 will be used as an example, too. The schematization specified above will be used for the calculations performed in these examples.

5. Semi-probabilistic vs. Fully probabilistic

5.1. Introduction

This chapter refers to the third research sub-question of this study: "*For dikes in general, does the semi-probabilistic assessment method for macro-stability lead to a conservative result?"* Note that this chapter refers to dikes in general, and not specifically to dikes with high foreshores.

As mentioned in Section 2.4, the calibrated semi-probabilistic assessment rule (formula (2.20)) is fitted at the 20%-quantile of the reliabilities, which means that for 20% of the cases considered in the calibration study , the semi-probabilistic rule underestimates the failure probability (Kanning et al., 2016). For the other cases, the rule is *conservative*, meaning that it overestimates the probability. Based on this, it can be expected that for dikes in general the semi-probabilistic method will most of the time lead to a conservative failure probability.

Looking at the safety assessment process as proposed by the WBI (described in Section 2.4.4), this makes sense. One first performs a relatively easy semi-probabilistic calculation, which results in an approximated failure probability, which overestimates the probability in most cases. If this approximated probability is higher than the target probability, one can choose to perform a more complicated fully probabilistic calculation, which may result in a probability lower than the target probability. This process can be seen as a coarse-to-fine method: by putting in more effort in the assessment one obtains a more positive result.

Based on the above, there is the hypothesis that the semi-probabilistic method will most of the time lead to a conservative failure probability. In this section it will be examined whether or not this is true for the test cases defined in Section 4.3. This will give more insight into the interpretation of the results later on in this Thesis. Comparisons will be made between the semi-probabilistic method and fully probabilistic calculations. Note that the calibration study of the semi-probabilistic rule was performed for dikes in general, not for dikes with foreshores. Therefore, dikes without foreshores will be used in this chapter. The above is made clear by the arrow in the calculation matrix, shown in Figure 5.1.

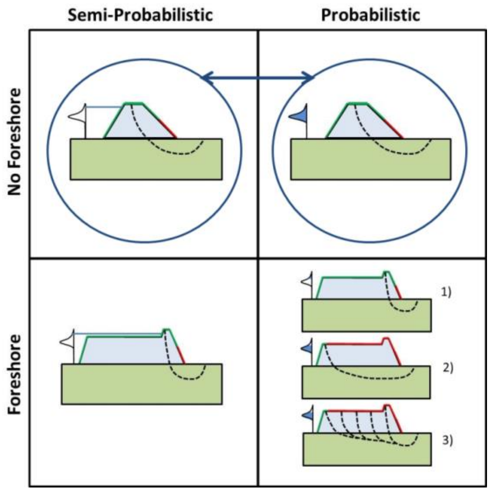


Figure 5.1: Calculation matrix showing the calculations performed in Chapter 5

5.2. Approach

To study whether or not the semi-probabilistic rule leads to a conservative failure probability, the test cases (as specified in Section 4.3) are used without foreshores. For these test cases the failure probabilities for macro-stability are calculated, both using the semi-probabilistic method and using a fully probabilistic calculation. For these calculations, the calculation choices as specified in Chapter 4 are used. The results of both calculations are compared, to see whether or not the semi-probabilistic method leads to a conservative failure probability (i.e. overestimates the probability). The approach is illustrated in the flowchart of Figure 5.2.

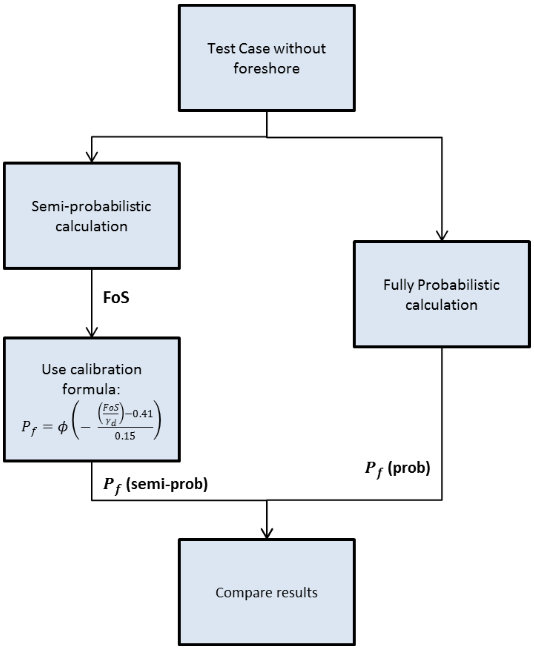


Figure 5.2: Flowchart showing the approach of the research performed in Chapter 5

5.3. Example

In this section, the calculations performed in this chapter will for Test Case 4 be covered in detail, as an example. This case was already introduced in Section 4.7, where the schematization of the geometry and the parameters were discussed.

First, the failure probability is calculated using the semi-probabilistic method. Then, the fully probabilistic calculations are performed. Finally, the results of both calculations will be compared.

5.3.1. Semi-Probabilistic calculation

The semi-probabilistic calculation is performed using a design water level (5.04m+NAP, see Section 4.7.1). For the other parameters, design values are used too, as described in Section 4.7.2.

Figure 5.3 shows the result of the semi-probabilistic calculation in the D-GeoStability software. The critical slip surface is shown. The corresponding factor of safety (FoS) is 1.058. This FoS can be transformed into an approximated failure probability using the calibrated formula (2.20):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right)$$

Using the FoS = 1.058 and γ_d=1.06 (model factor Uplift Van), this leads to an annual failure probability of P_f= 4.41*10^{-5} (β = 3.921).

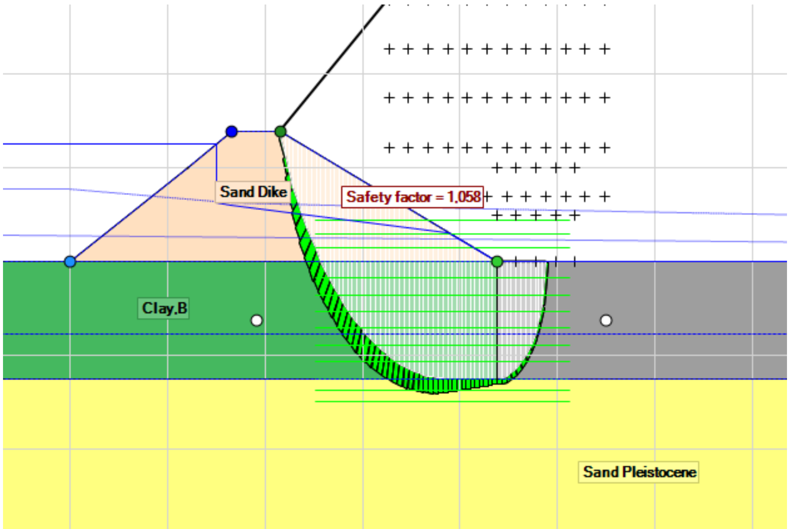


Figure 5.3: Result semi-probabilistic calculation Test Case 4 without foreshore (taken from D-GeoStability)

5.3.2. Fully probabilistic calculation

This section covers the fully probabilistic calculation of Test Case 4, without a foreshore. As mentioned before, for a dike without foreshore, the *WBI failure definition* can be used for all water levels, meaning that all slip surfaces with an entrance point halfway the inner slope or further to the water side are considered relevant. The results of the FORM calculations are shown in Table 5.1, which shows for all water levels separately the conditional reliability index, the conditional failure probability and the influence factors of all parameters.

	Water level [m+NAP]							
	2	2.5	3	3.5	4	4.5	5	5.5
Failure analysis								
Reliability β [-]	4.78	4.58	4.33	4.05	3.73	3.38	3.01	2.56
Failure probability	8.76	2.32	7.46	2.56	9.57	3.62	1.31	5.23
	E-07	E-06	E-06	E-05	E-05	E-04	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.045	0.189	0.462	0.893	1.482	2.891	14.479
λ_{out}	0	0.031	0.13	0.319	0.616	1.012	1.915	4.249
Intrusion length	0.117	0.404	0.044	0.722	3.04	7.333	7.468	13.38
Yield (139.12)	14.162	14.617	15.435	16.272	17.089	18.097	18.449	15.822
Yield(175)	3.784	3.887	4.019	4.103	4.111	4.509	5.273	4.019
φ (Sand dike)	0.54	0.519	0.498	0.468	0.427	0.361	0.314	0.218
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	68.223	67.452	66.735	64.97	61.676	55.957	52.677	39.333
m (Clay,B; Clay,N)	1.125	1.185	1.271	1.374	1.475	1.666	2.094	1.908
m_d	12.049	11.86	11.678	11.31	10.672	9.584	8.918	6.592

Table 5.1: Results FORM calculations Test Case 4 without foreshore: showing for all water levels the conditional reliability index, conditional failure probability and the influence coefficients of all parameters

The conditional reliabilities for the different water levels are used to construct a (β-h)-curve , which is transformed into a fragility curve (see Figure 5.4). Integration of the product of the fragility curve and the water level distribution over the water levels leads to a total annual failure probability of $P_f = 1.52*10^{-5}$ (β = 4.171).

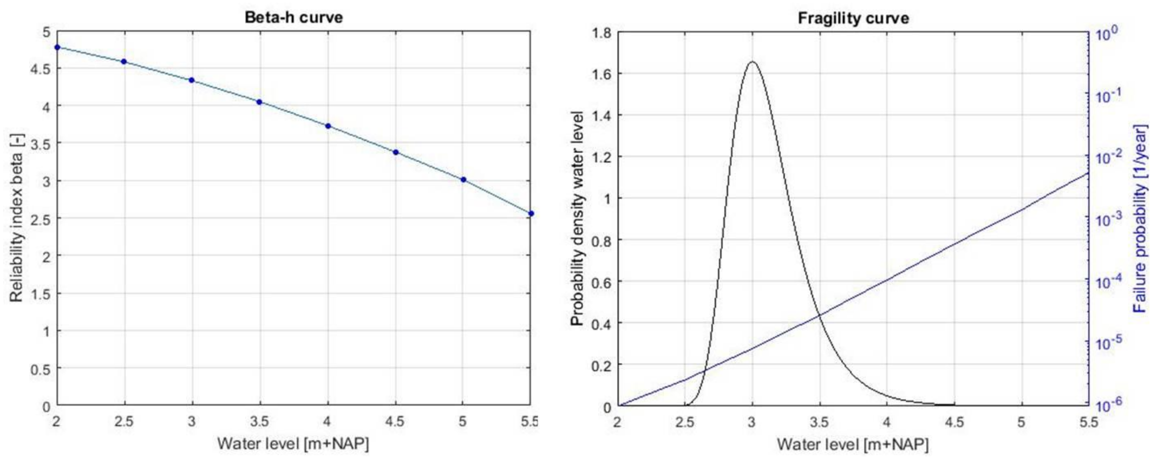


Figure 5.4: Beta-h curve (left) and fragility curve (right) of Test Case 4

5.3.3. Comparison

For Test Case 4, without a foreshore, the semi-probabilistic method resulted in an approximated failure probability of $4.41*10^{-5}$. The probabilistic failure probability, calculated using fragility curves and FORM calculations, is $1.52*10^{-5}$. This means that the semi-probabilistic method overestimated the failure probability with approximately a factor 3. For Test Case 4, the semi-probabilistic method therefore indeed leads to a conservative result.

In the next section the results of the other test cased are covered, to see whether this also holds for these test cases.

5.4. Results

Table 5.2 shows the results for all test cases for both the fully probabilistic calculation and the semi-probabilistic calculation (the complete results of the fully probabilistic calculations can be found in Appendix D). Besides, the ratio between the semi-probabilistic failure probability and the fully probabilistic failure probability is shown. Note that for a ratio higher than one, the semi-probabilistic method leads to a conservative result, and for a ratio lower than one, it leads to underestimation of the failure probability. The table shows that for most cases, the semi-probabilistic method is not conservative, but leads to underestimation of the failure probability.

Test Case	P_f probabilistic [-]	P_f semi-prob [-]	Ratio P_f semi-prob / P_f probabilistic [-]
1	4.07E-02	6.92E-03	0.17
2	8.68E-04	2.22E-04	0.26
3	6.40E-03	1.61E-03	0.25
4	1.52E-05	4.41E-05	2.90
5	1.47E-05	1.43E-05	0.97
6	1.50E-05	2.97E-05	1.98
7	1.14E-05	1.14E-05	1.00

Table 5.2: Results of both the semi-probabilistic and fully probabilistic calculations for all test cases without foreshores (alpha = 4.5)

The results shown in Table 5.2 are based on the basic water level distribution ($\alpha = 4.5$). However, as mentioned in Section 4.4, all calculations in this study are performed twice: once using the basic water level distribution ($\alpha = 4.5$) and once using a distribution with less spreading ($\alpha = 7.5$). The results using the narrower water level distribution ($\alpha = 7.5$) are shown in Table 5.3. For this water level distribution, there are even more cases for which the semi-probabilistic method leads to underestimation of the failure probability.

Test Case	P_f probabilistic [-]	P_f semi-prob [-]	Ratio P_f semi-prob / P_f probabilistic [-]
1	3.84E-02	3.15E-03	0.08
2	8.39E-04	1.63E-04	0.19
3	5.00E-03	7.73E-04	0.15
4	1.02E-05	1.05E-05	1.03
5	1.38E-05	9.92E-06	0.72
6	9.86E-06	7.69E-06	0.78
7	1.07E-05	9.38E-06	0.88

Table 5.3: Results of both the semi-probabilistic and fully probabilistic calculations for all test cases without foreshores (alpha = 7.5)

The results of both tables are shown in the scatter plot of Figure 5.5. The left part shows the complete scatter plot. The right part is zoomed in at the area of the black square. The black line in both plots shows the line of equal probability: if a point is on this line, the result of semi-probabilistic method is equal to the fully probabilistic result. A point above this line means the semi-probabilistic method leads to an underestimation of the failure probability and a point below the line means the semi-probabilistic method is conservative. Based on Figure 5.5, it seems that the semi-probabilistic method is not as conservative as was expected, because for most test cases, it leads to an underestimation of the failure probability.

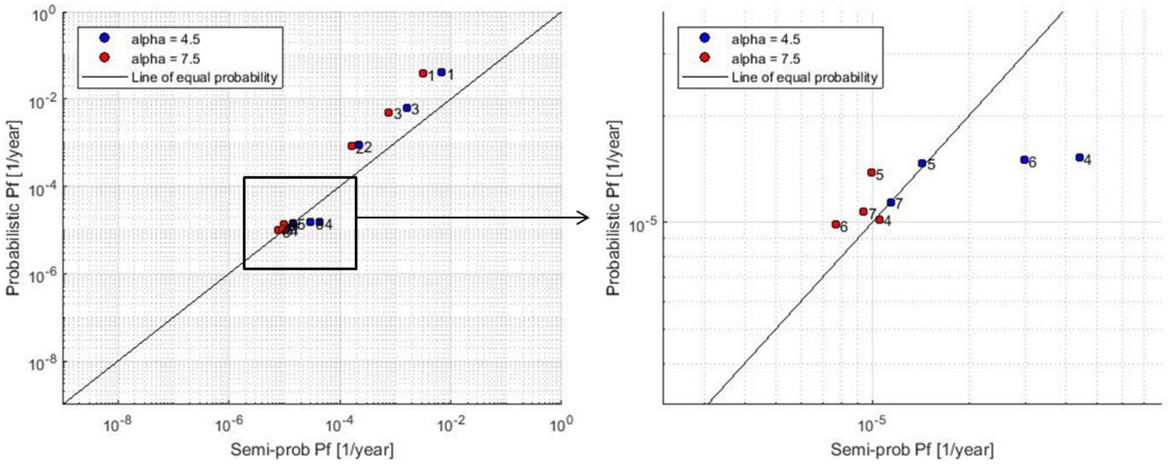


Figure 5.5: Scatter of the failure probabilities of the semi-probabilistic method vs the fully probabilistic calculations

Figure 5.6 shows the results of the calculations performed in this Chapter (in red), together with the scatter plot of the original calibration study for the semi-probabilistic rule (Section 2.4.5) (Kanning et al., 2016). The x-axis shows the reliability indexes β (fully probabilistic calculations) and the y-axis shows the safety factor γ_n. This safety factor is the direct result of the semi-probabilistic method (before transforming the result into an approximated failure probability, see the example in Section 5.3) and is defined as FoS/γ_d (in which γ_d is the model factor, equal to 1.06 for the Uplift Van model). The black line shows the calibrated (β-γ_n)-relation (formula (2.19)), which was fitted at the 20%-quantile of the reliabilities β:

$$\gamma_n = 0.15 * \beta_{T,cross} + 0.41$$

Figure 5.6 shows that for most test cases (in red), the semi-probabilistic rule leads to an underestimation of the failure probability (the reliabilities are lower than predicted according to the semi-probabilistic rule, i.e. the points are on the left of the black line), like mentioned before. However, the figure also shows that the calculated points fit reasonably well into the scatter of the original calibration study. This means that the calculation results are not completely surprising, because in the calibration study similar results were already found.

What is surprising however, is the fact that for most test cases in this study, the semi-probabilistic rule is not conservative, which, according to the calibration study, should be the other way around. Possibly, this could indicate that the semi-probabilistic rule is not as conservative as was expected. However, the seven test cases that were analysed are not sufficient to draw conclusions of this kind. For illustrational purposes, Figure 5.6 shows where the calibrated relation (at the 20%-quantile of the reliabilities) would be if it was based only on the seven test cases of this Thesis. This clearly illustrates that the test cases in this Thesis show a different trend than was found in the calibration study.

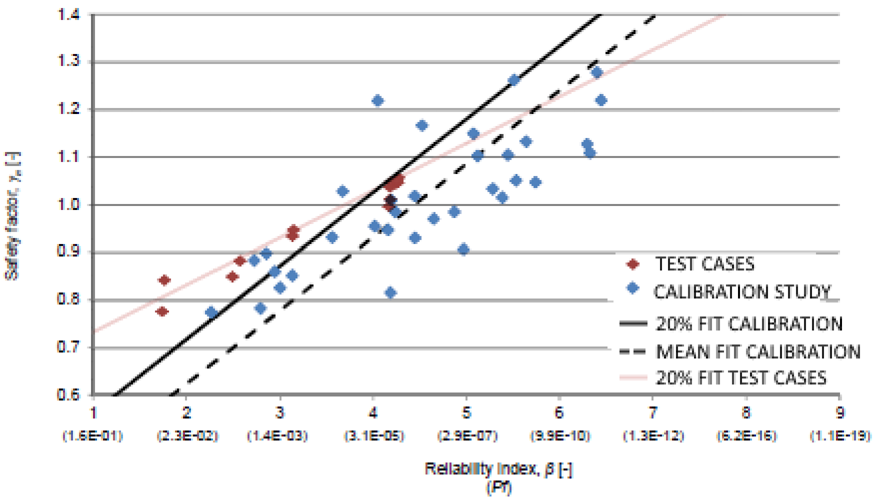


Figure 5.6: Results of the calculations performed in Chapter 5 shown in the scatter plot of the original calibration study

Finally, Figure 5.6 shows that for lower reliabilities, the underestimation of the failure probabilities is the biggest; the higher the reliabilities, the further the calculated points move to the right side of the semi-probabilistic rule (black line). This is important for the safety assessment of dikes. If the semi-probabilistic method is used for the safety-assessment, and this leads to underestimation of the failure probability, this means that the real failure probability is higher than the calculated failure probability. In theory, this could lead to a dangerous situation: if the semi-probabilistic result is lower than the target probability, the dike will be assessed as safe, while the real failure probability may be higher than the target probability. However, the points where underestimation is the biggest (see Figure 5.6) correspond to such low reliabilities that the semi-probabilistic rule will not lead to a safe result (a failure probability lower than the target probability). This will only be the case for higher reliabilities, starting around $\beta = 4$, for which, based on the calculations in this Chapter, the semi-probabilistic rule leads to little or no underestimation of the failure probability.

The above can also be seen by comparing the 20% fit of the calibration and the 20% fit of the test cases in Figure 5.6. Based on this comparison, it seems that for reliabilities lower than $\beta=4$ the current semi-probabilistic rule is not conservative enough (i.e. leads to underestimations) and for higher reliabilities it is very conservative. However, again the seven test cases analysed are not sufficient to draw conclusions of this kind.

Another result that was found during this research, is that the failure probability, as calculated with the semi-probabilistic method, depends partly on the safety standard. In other words, for different safety standards, different semi-probabilistic failure probabilities are found for the same dike. This falls outside the scope of this Thesis, and is therefore not further discussed here. Appendix E covers this phenomenon in more detail.

5.5. Discussion results

The results in the previous section showed that for the test cases considered in this study, the semi-probabilistic method for most cases resulted in an underestimation of the failure probability. This is surprising, because the (β-γ_n)-relation was calibrated in such a way, that the semi-probabilistic method would underestimate the failure probability in only 20% of the cases. In the other cases the result should be conservative. This section tries to find an explanation for the differences between the calibration study and the test cases used in this Thesis.

First of all, it should be noted that the calibration study was performed using real dikes from various locations throughout the Netherlands. The test cases, on the other hand, were created by varying some parameters in a basic case. This means that all test cases are based on the same basic case. Possibly, this way a specific subset of dikes was created, which is not representative for all dikes in the Netherlands.

Furthermore, the test cases seem to be biased. The test cases used in this Thesis are simplified (see Section 4.3). The subsoil in the test cases consists of only one aquitard layer and one aquifer layer. However, in reality, the aquitard almost always consists of more than one soil layer. Figure 5.7 shows an example of both a test case schematization and a real dike. Note that in the test case the aquitard consists of only one soil layer, and in the example of the real dike it consists of three layers. In both cases, the same slip surface is shown. In the test case, the slip surface crosses only one subsoil layer. In the real dike schematization, the slip surface crosses three subsoil layers.

For failure to occur along the drawn slip surface, in the test case schematization the strength of only one soil layer has to be low. In the real schematization three soil layers need to have a low strength for failure to occur. The strength of a soil layer has a certain probability distribution, so there is a certain probability that the strength of a soil layer will be weak. Because soil layers are generally modelled as independent, the probability that three soil layers will have a low strength is lower than the probability that only one will have a low strength. This means that the more (independent) soil layers the slip surface crosses, the lower the failure probability will be. Modelling the aquitard as one layer, which is a simplification of the reality, therefore leads to higher failure probabilities.

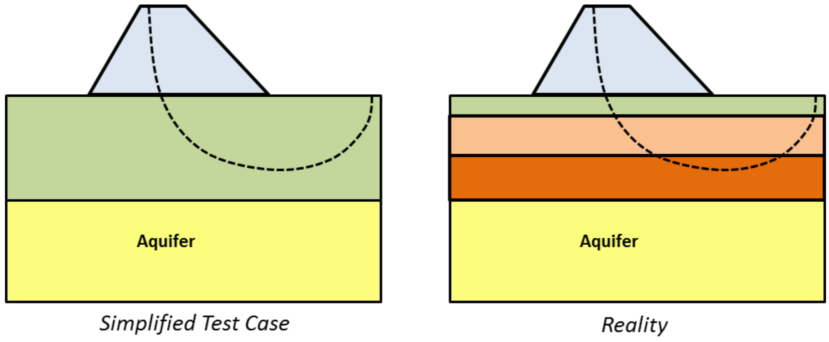


Figure 5.7: Illustration of a simplified test case as used in this Thesis, consisting of only one aquitard layer, and a schematization better resembling reality with three aquitard layers

Note that the above is only the case for the fully probabilistic calculations, which use the complete probability distributions of all parameters. After all, in the semi-probabilistic method, for every parameter (in every soil layer) only one design value is used (the 5% quantile), also for the strength parameters of the soil layers. The probability of occurrence of a certain strength value is not taken into account. Therefore, the probability of failure does not decrease for an increasing amount of layers.

Concluding the above, the test cases lead to unrealistically high failure probabilities resulting from the fully probabilistic calculations, because the aquitard is modelled as one layer. In the calibration study, real dikes were used, in which the aquitard consists of more layers. Therefore, lower failure probabilities were found in the calibration study. As explained, the amount of layers will probably not affect the semi-probabilistic calculations much. This means that the calculated points (in this Thesis) in Figure 5.6 are too far to the left (too low reliabilities), they are biased. If more soil layers would be used in the aquitard, thus better resembling the reality, the points would shift to the right. This is illustrated in Figure 5.8, which shows both the results of the (biases) test cases as considered in this thesis, and possible results for more realistic cases, using more soil layers (note that the latter are not real results, they are purely meant as illustration). The figure shows that using more soil layers will cause a shift to the right of the scatter. Possibly, this shift will cause the scatter points to end up at the right of the 20% beta fit (black line), which means that the semi-probabilistic method would be conservative.

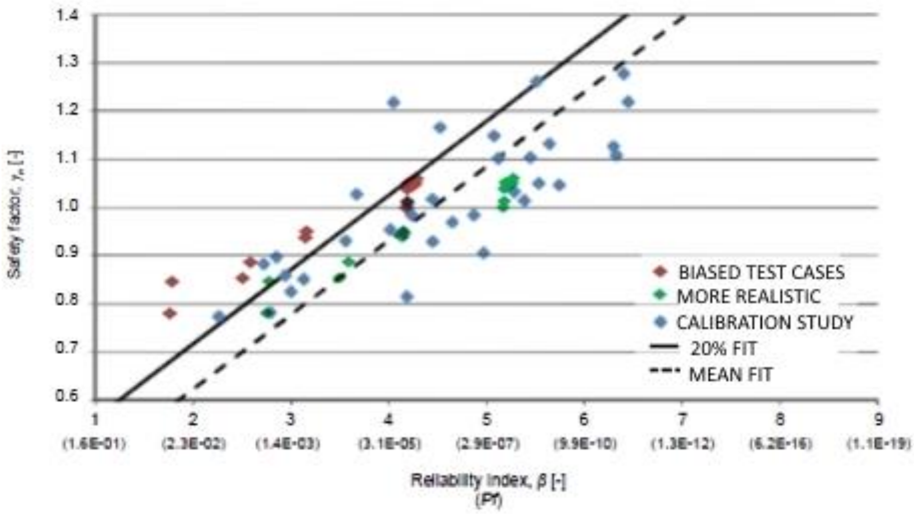


Figure 5.8: Calibration plot showing the calculation results of the test cases and possible results for more realistic cases, using more soil layers

5.6. Conclusion

In the beginning of this chapter, the hypothesis was stated that the semi-probabilistic method for normal dikes would in most cases lead to a conservative failure probability (i.e. higher than the fully probabilistic failure probability).

However, for the test cases considered in this Thesis, without foreshores, the semi-probabilistic method did in most cases not lead to a conservative result, but to an underestimation of the failure probability. This is mainly the case for reliabilities (fully probabilistic results) lower than β=4. This could be an indication that the semi-probabilistic method is not as conservative as was expected for those reliabilities. However, this cannot be concluded just based on these seven test cases. Moreover, as explained in the previous section, the test cases are biased, leading to unrealistically high failure probabilities (in the fully probabilistic calculations), which can explain the fact that the semi-probabilistic method underestimates the failure probabilities.

One still has to be aware of the fact that the semi-probabilistic method for dikes without foreshores results in an approximated failure probability which is certainly not always conservative. However, this is not clearly stated in the WBI documents (Rijkswaterstaat, 2016a, 2016b), in which the approach seems more like a coarse-to-fine method. One should always keep in mind that the semi-probabilistic method can lead to underestimation of the failure probability, and performing a fully probabilistic calculation, by putting in more effort, may therefore result in a higher failure probability. In other words: one has to use the semi-probabilistic method with care.

Besides, it would be wise to collect the data of studies which use both the semi-probabilistic method and a fully probabilistic calculation to calculate the failure probabilities of real dikes (or more realistic test cases). Once more (realistic) data are available, one is able to draw better conclusions about how conservative the semi-probabilistic method actually is.

6. Dike safety for different water levels

6.1. Introduction

This Chapter refers to the fourth sub-research question of this Thesis: "*For dikes with high and wide foreshores, is the dike significantly safer for water levels below the level of the foreshore than for water levels above foreshore level ?*" The differences in conditional failure probabilities for water levels below foreshore and above foreshore, and how these are affected by the failure definition, will be discussed. This chapter completely focuses on probabilistic calculations for dikes with foreshores. This is shown in the calculation matrix of Figure 6.1. As mentioned before, two different failure definitions can be used for these calculations.

The first possibility is using the *WBI failure definition* for all water levels, considering all slip surfaces that enter halfway the inner slope or further to the water side relevant. However, for water levels below the level of the foreshore, this means that a (relatively) shallow slip surface is regarded as failure, while there is still a large dike body (including foreshore) left to retain the water; the dike has *residual width,* as explained in Chapter 3. This residual width is neglected by the WBI failure definition.

The second possibility is taking into account the *residual width* of a dike, by using the three scenarios mentioned in Chapter 3. For water levels above foreshore, there is little or no residual width and the WBI failure definition can be used (scenario 1). For water levels below foreshore, the residual width can be taken into account by looking for a long and deep sliding plane (scenario 2) or analysing progressive failure (scenario 3).

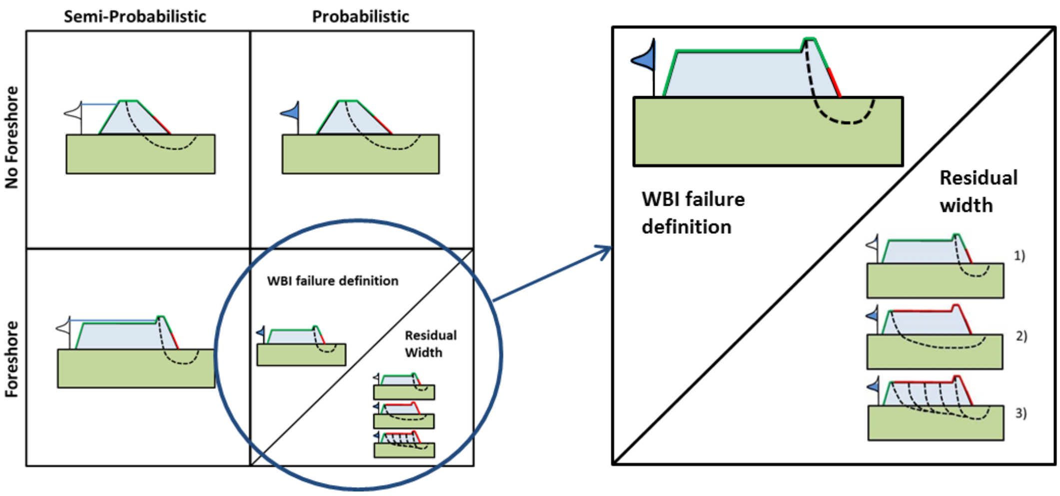


Figure 6.1: Calculation matrix showing the calculations performed in Chapter 6

A deep and long sliding plane and progressive failure probably lead to lower failure probabilities than a shallow slip surface. This means that, if one takes into account residual width, it can be expected that the dike is significantly safer (has a lower conditional failure probability) for water levels below foreshore than for higher water levels. Furthermore, if this residual width, and therefore the extra safety, would be neglected, this would probably lead to overestimation of the failure probability.

Based on the above, there are the following hypotheses:

- Taking into account residual width, the dike is significantly safer for water levels below the level of the foreshore than for higher water levels;
- Using the WBI failure definition, and thus neglecting the residual width, leads to overestimation of the failure probability.

In this chapter it will be validated whether or not these hypotheses are true, by performing fully probabilistic calculations on dikes with foreshores, using both failure definitions specified above, and comparing the results.

6.2. Approach

As mentioned in the previous section, in this chapter probabilistic calculations will be performed on dikes with foreshores. Two different calculations will be performed: one using the *WBI failure definition,* one taking into account *residual width* using the three scenarios as specified before.

The flowchart of Figure 6.2 shows the process of performing these two different calculations. The process is described below.

First, the calculation using the WBI failure definition is performed. For a range of water levels, the corresponding conditional failure probabilities are calculated, which are used to construct a fragility curve. Based on this fragility curve and the water level distribution, the total failure probability is calculated.

Then the probabilistic calculation taking into account the residual width is performed. Again, for a range of water levels the conditional failure probabilities are calculated. For water levels above foreshore, the WBI failure definition will still be used, as mentioned before. This means that the conditional failure probabilities for the water levels above foreshore of the previous calculation (using the WBI failure definition) can also be used for the calculation taking into account residual width; they don’t have to be calculated again. For the water levels below foreshore, the residual width is taken into account using two scenarios: failure due to one long sliding plane and progressive failure. The conditional failure probabilities for water levels below foreshore are calculated in the following order:

- **Step 1:** First, the probabilities of progressive failure (scenario 3) are calculated. In the progressive failure analysis, the individual probabilities of the successive soil slides are calculated. Section 4.5.4 already described that in the progressive failure analysis, the combined probability of successive soil slides is generally equal to the individual probability of the last slide. Each individual slide corresponds to a certain individual probability, and a certain amount of foreshore that is lost due to all slides up to that point (compared to the initial situation). The total probability of losing that certain amount of foreshore due to progressive failure is thus equal to the individual probability of the last slide.

- **Step 2:** As a result of step 1, it is known what the probabilities are of losing certain amounts of foreshore due to progressive failure. In step 2, the probabilities are determined of those same amounts of foreshore loss caused by one (deep and long) sliding plane in the initial dike profile (scenario 2).
- **Step 3:** The combined result of steps 1 and 2 is a list of certain amounts of foreshore loss and the corresponding probabilities of that loss caused by progressive failure and caused by one sliding plane. Now, for every amount of foreshore loss, the dominant probability (the highest) of the two is picked as result. Example 6.1 gives an example of steps 1 till 3.
- **Step 4:** Steps 1 till 3 are repeated until failure is reached (the remaining width is less than 3m, as defined in Appendix B), or until the stop criterion is reached and the failure probability has become negligible (see Section 4.5.4 and Appendix B). This results in the conditional failure probabilities for water levels below foreshore level.

After performing the steps described above, the conditional failure probabilities for the complete range of water levels (both above and below foreshore) are known, and the fragility curve is obtained. Based on the fragility curve the total failure probability is calculated. By comparing the total failure probability resulting from the WBI failure definition on the one hand and taking into account residual width on the other hand, it will be verified whether the WBI failure definition, which neglects residual width, indeed leads to overestimation of the failure probability.

Besides, it will be verified whether the dike is significantly safer for water levels below foreshore level than for higher water levels, by determining the contribution to the total failure probability of the water levels below foreshore on the one hand and the higher water levels on the other. This will be further explained in the next section.

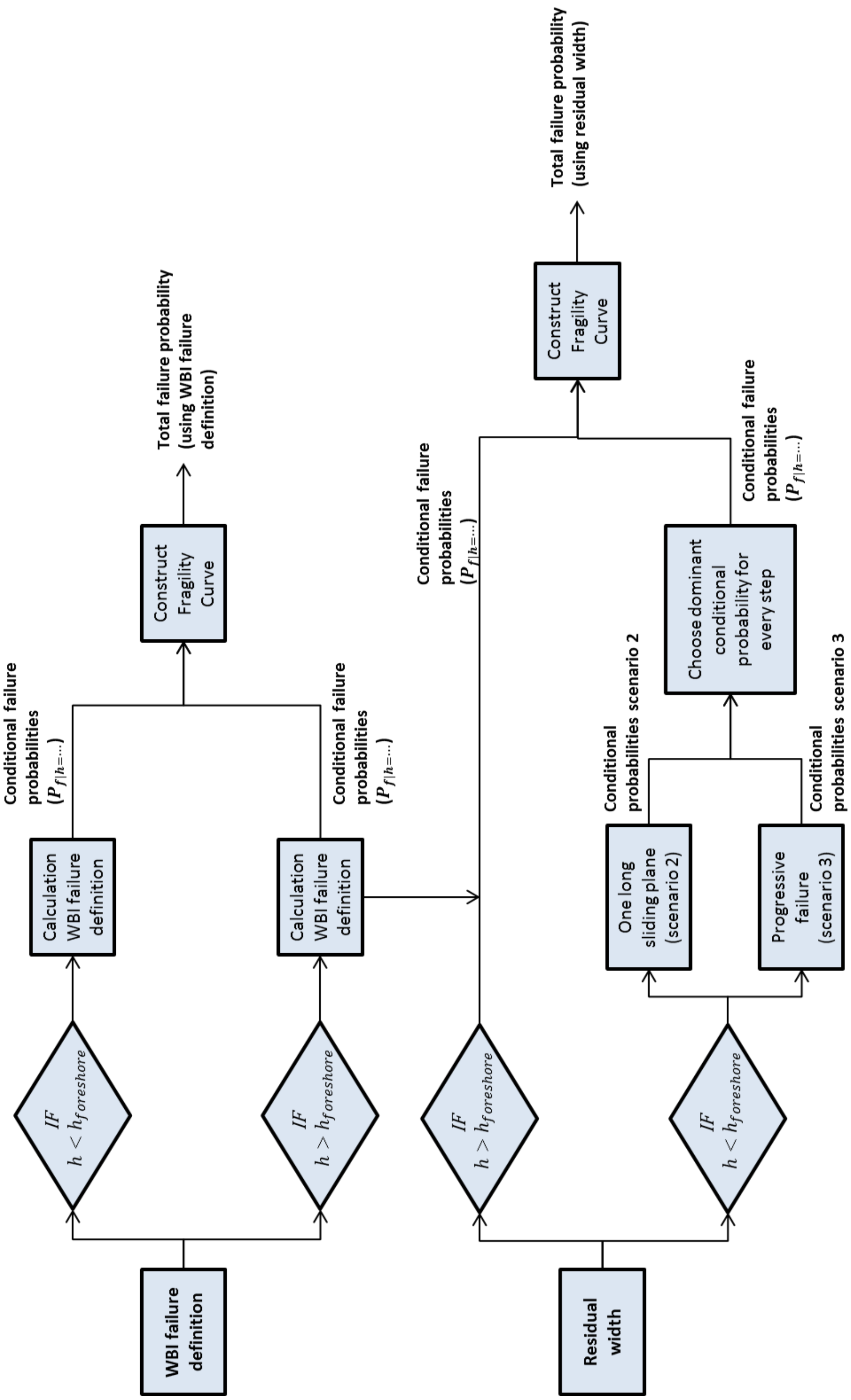


Figure 6.2: Flowchart of the approach for the calculations of Chapter 6

Example 6.1:

Imagine that for a certain profile, (part of) the results of the progressive failure analysis are as shown in the table below. Example 6.2 will give some more information about the meaning of the individual probabilities.

Slide nr	Individual Probability [-]	Total Foreshore loss [m]
1st slide	1/1,000	10
2nd slide	1/3,000	20
3rd slide	1/6,000	35
4th slide	1/10,000	50

As mentioned, the probability of a combination of soil slides is generally equal to the individual probability of the last slide. The table shows that after four slides, 50m of foreshore is lost. This corresponds to a probability of 1/10,000 (equal to the individual probability of the 4th slide). The probability of occurrence of one single sliding plane leading to a loss of 50m of foreshore has a probability of 1/12,000. In this case, progressive failure is dominant (has a higher probability). This means that there is a probability of 1/10,000 that 50m of foreshore will be lost. In Figure 6.3 this is illustrated.

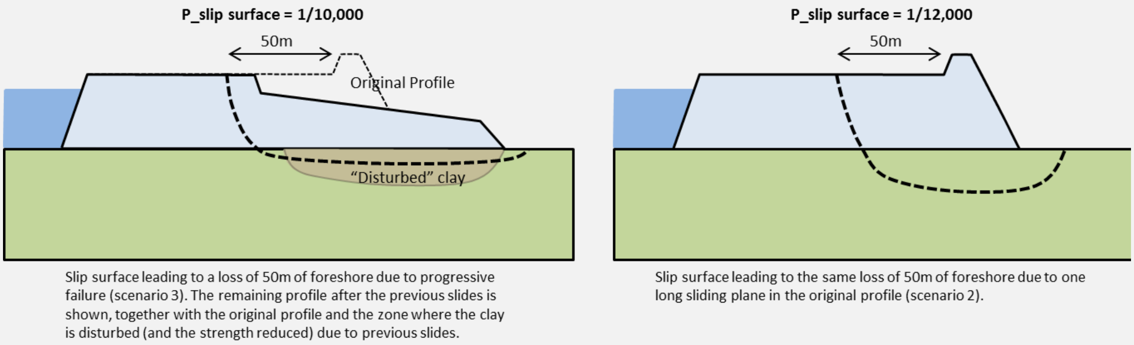


Figure 6.3: Example progressive failure vs. long sliding plane

The approach above is performed for all individual slides (see table above) in the progressive failure analysis. So for all amounts of foreshore loss shown in the table, the probability will be determined both for scenario 2 and 3, to determine which scenario is dominant.

Example 6.2:

This example refers to the table of example 6.1. It is meant to give some more understanding of the meaning of the individual probabilities of the slides. The table below shows the individual probabilities of the slides, the combined probabilities of the slides and the conditional probabilities of the slides given occurrence of the previous slides.

Slide nr	**Individual Probability slide**	**Combined Probability slides**	**Conditional Probability (given previous slides)**
1st slide	P_1 = 1/1,000	P_1 = 1/1,000	P_1 = 1/1,000
2nd slide	P_2 = 1/3,000	$P_{1,2}$ = 1/3,000	$P_{2\|1}$ = 1/3
3rd slide	P_3 = 1/6,000	$P_{1,2,3}$ = 1/6,000	$P_{3\|1,2}$ = 1/2
4th slide	P_4 = 1/10,000	$P_{1,2,3,4}$ = 1/10,000	$P_{4\|1,2,3}$ = 1/1.67

As mentioned before, a combination of successive soil slides can be seen as a parallel system. It is assumed that the combined probability of successive slides is equal to the minimum of the individual probabilities (which is generally equal to the individual probability of the last slide), as shown in the table above. This is the case when the slides are completely correlated.

The conditional probabilities are defined as follows (Jonkman, Steenbergen, Morales-Nápoles, Vrouwenvelder, & Vrijling, 2016):

$$P(A|B) = \frac{P(A \cap B)}{P(B)}$$

In which:

$P(A|B)$ The conditional probability of event A given occurrence of event B
$P(A \cap B)$ The probability of both event A and B occurring
$P(B)$ The probability of event B

For full correlation between A and B, it holds that $P(A \cap B) = P(A)$. After all, the probability of a combination of soil slides is equal to the individual probability of the last slide. The conditional probability than reduces to:

$$P(A|B) = \frac{P(A)}{P(B)}$$

This means that the probability of a new slide given occurrence of the previous slides is equal to ratio of the individual probability of the new slide and the probability of the combination of the previous slides.

Note that this holds for the assumption that the individual slides are fully correlated. In fact, this is a conservative assumption, which is used because the dependence between successive slides is unknown. If one would use the exact correlations, both the combined failure probabilities and the conditional probabilities would decrease.

6.3. Example

In this section the calculations performed in this chapter are covered in detail for Test Case 4 as an example. Test Case 4 was also used in the previous examples in Chapters 4 and 5.

6.3.1. WBI failure definition

The first calculation step is performing fully probabilistic calculations for Test Case 4 (including foreshore), using the WBI failure definition for all water levels (see Figure 6.4).

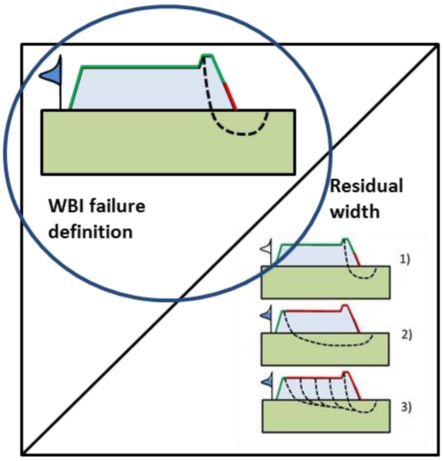


Figure 6.4: Calculation matrix showing the two possible failure definitions to be used for probabilistic calculations of dikes with high foreshores

The results of the FORM calculations are shown in Table 6.1. The table shows, for all water levels separately, the conditional reliability index β (given that water level), the conditional failure probability, and the influence factors of the different parameters.

The conditional reliabilities are used to construct a (β-h)-curve , which is transformed into a fragility curve (see Figure 6.5). Integration of the product of the fragility curve and the water level distribution over the water levels using formula (2.12) leads to a total probability of $9.76*10^{-6}$ (β = 4.27).

	Water level [m+NAP]							
	2	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	4.79	4.4	4.2	3.93	3.72	3.43	3.04	2.7
Failure probability	8.34	5.41	1.33	4.25	9.96	3.02	1.18	3.47
	E-07	E-06	E-05	E-05	E-05	E-04	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.185	0.437	0.849	0.89	1.078	2.621	2.235
λ_{out}	0	0.128	0.302	0.586	0.614	0.737	1.766	1.502
Intrusion length	0.352	0.008	0.106	0.634	4.559	3.038	3.26	7.112
Yield (82.06)	0	0	0	0	0	0	0	0
Yield (139.12)	14.292	15.574	16.252	17.088	17.322	19.525	19.818	21.726
Yield(175)	3.654	3.879	3.975	4.058	3.963	4.77	5.625	4.742
φ (Sand dike)	0.526	0.511	0.5	0.475	0.418	0.36	0.33	0.272
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	68.197	66.872	65.719	63.867	60.522	58.993	55.488	52.005
m (Clay,B; Clay,N)	0.982	1.098	1.172	1.269	1.243	1.481	1.701	1.761
m_d	11.996	11.746	11.537	11.174	10.469	10.018	9.391	8.645

Table 6.1: Results FORM calculations Test Case 4 with foreshore, showing for all water levels the conditional reliability indes, conditional failure probability and the influence coefficients of all parameters

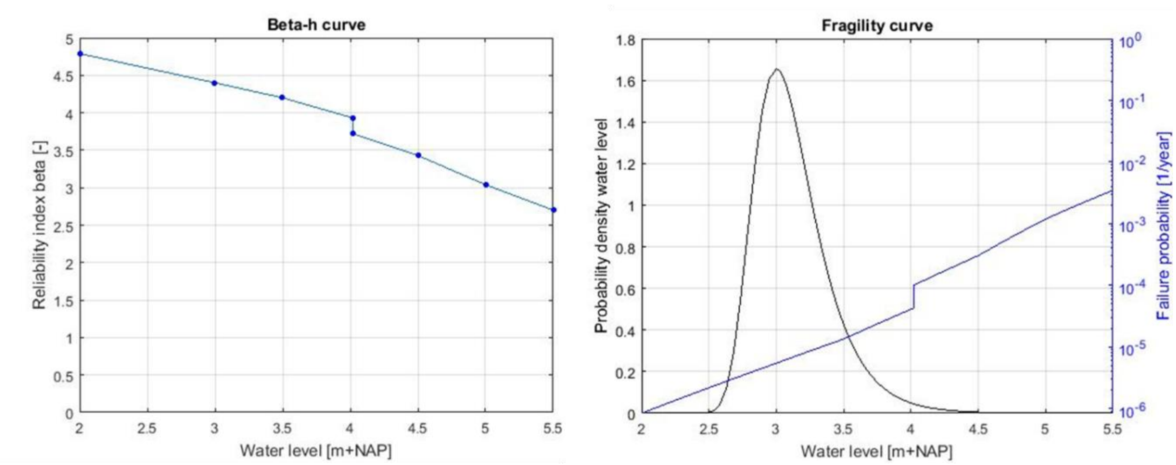


Figure 6.5: Beta-h curve (left) and fragility curve (right) of Test Case 4 with foreshore

Figure 6.6 shows the fragility curve and the water level distribution again. Besides, the figure distinguishes between water levels below foreshore level and water levels above foreshore level. The level of the foreshore is depicted as the red line, the water levels below foreshore level are depicted in yellow and the water levels above foreshore level in green. Note that there is a step in the fragility curve, just at the level of the foreshore. This step can be explained by the fact that the phreatic line at the location of the dike crest will be somewhat lower for water levels below foreshore level, because the phreatic line develops over a larger distance (100m extra due to the foreshore). This lower phreatic line causes a little more stability (and thus leads to a smaller conditional failure probability).

As explained before, the total failure probability is obtained by integrating the product of the fragility curve (the conditional failure probabilities) and the water level distribution over the water levels. By integrating this product over the water levels that are below the level of the foreshore and above the level of the foreshore separately, the contribution to the total failure probability of the water levels below foreshore on the one hand and the water levels above foreshore on the other hand are obtained. For the water levels below the foreshore level this leads to a probability of $7.68*10^{-6}$ and for water levels above foreshore to a probability of $2.08*10^{-6}$. The summation of the two is equal to the total failure probability, $9.76*10^{-6}$.

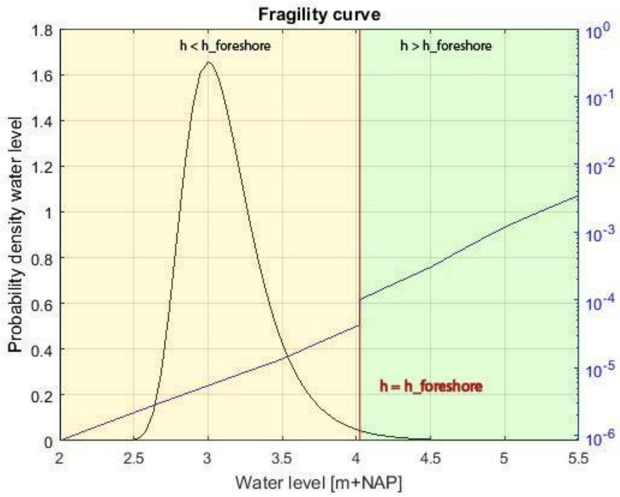


Figure 6.6: Fragility curve for Test Case 4 with foreshore using the WBI failure definition. Distinction has been made between water levels below and above foreshore

Remember that for the calculation all slip surfaces with an entrance point halfway the inner slope or further to the water side were taken into account. Figure 6.7 shows for two water levels, one above foreshore level and one just below foreshore level, the critical slip surfaces in the D-GeoStability software (using the design point values resulting from the FORM calculations as input). For both water levels, the critical slip surface has a similar shape. For the water level above foreshore this leads to little residual width; there is little dike left to retain the water. For the water level below foreshore, there is a lot of residual width; the complete foreshore and part of the dike are intact to retain the water. The shapes of the critical slip surfaces are for Test Case 4 for all water levels similar as shown in Figure 6.7. This means that for all water levels below foreshore level, there is this large amount of residual width.

Note: Figure 6.7 shows that the development of the phreatic line is different for water levels above foreshore and water levels below foreshore. This explains the step in the fragility curve at the level of the foreshore, as mentioned before.

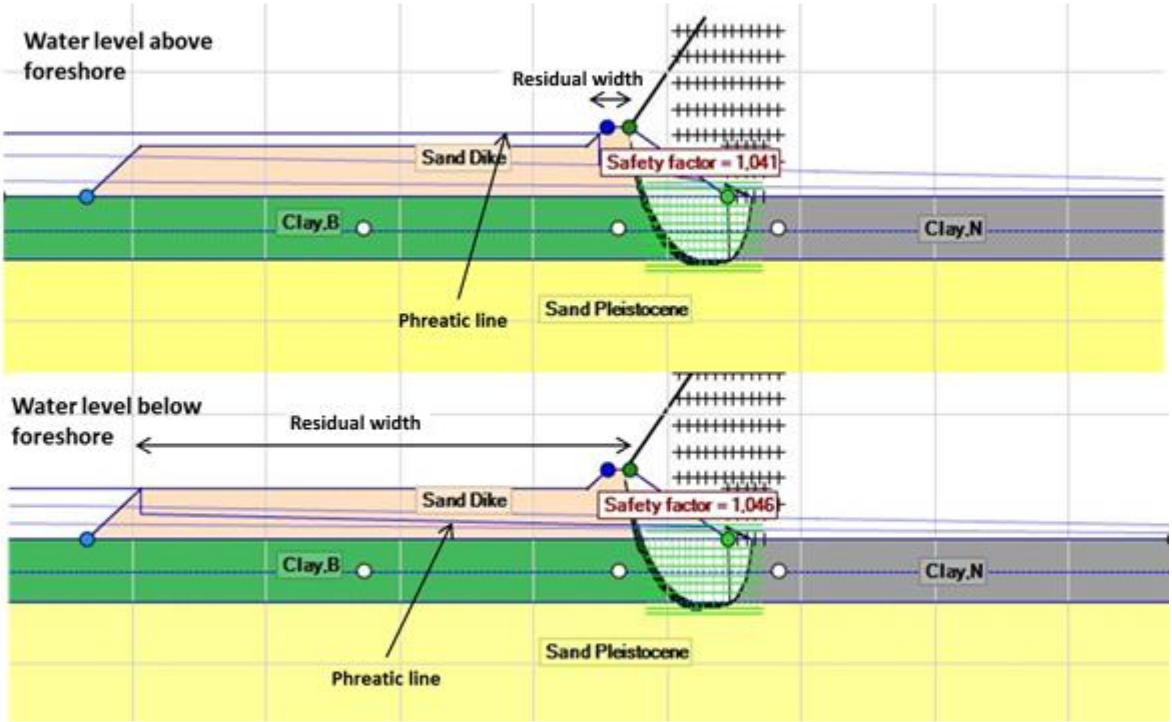


Figure 6.7: Critical slip surfaces for a water level above and below foreshore, showing the residual width of the dike in both situations

Concluding the above, the calculation using the WBI failure definition results in the probabilities shown in Table 6.2. However, in this calculation the residual width was neglected. In the next section, the calculation is performed again, this time taking into account the residual width.

P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
9.79E-06	7.68E-06	2.08E-06

Table 6.2: Results Test Case 4 using the WBI failure definition, showing the total failure probability and the contributions of the water levels above and below foreshore to the total failure probability

6.3.2. Residual width

To take into account the residual width in the calculation, the three scenarios are used (Section 6.1). This is illustrated in Figure 6.8:

- For the water levels above foreshore level (scenario 1), the WBI failure definition can be used, as the dike has no or little residual width for these water levels. This means that the results for these water levels do not change, and the green part of the fragility curve stays the same. Therefore, the contribution of these water levels to the total failure probability is still $2.08*10^{-6}$.
- For the water levels below foreshore (scenarios 2 and 3), the WBI failure definition neglects the large amounts of residual width (see Figure 6.7). Taking into account this residual width will decrease the (conditional) failure probabilities of these water levels, causing the fragility curve to decrease. This is illustrated in Figure 6.8 with the dashed line “residual width”. Note that this is just an illustration. The exact fragility curve after taking

into account residual strength will be determined below, by determining the probabilities for scenarios 2 and 3.

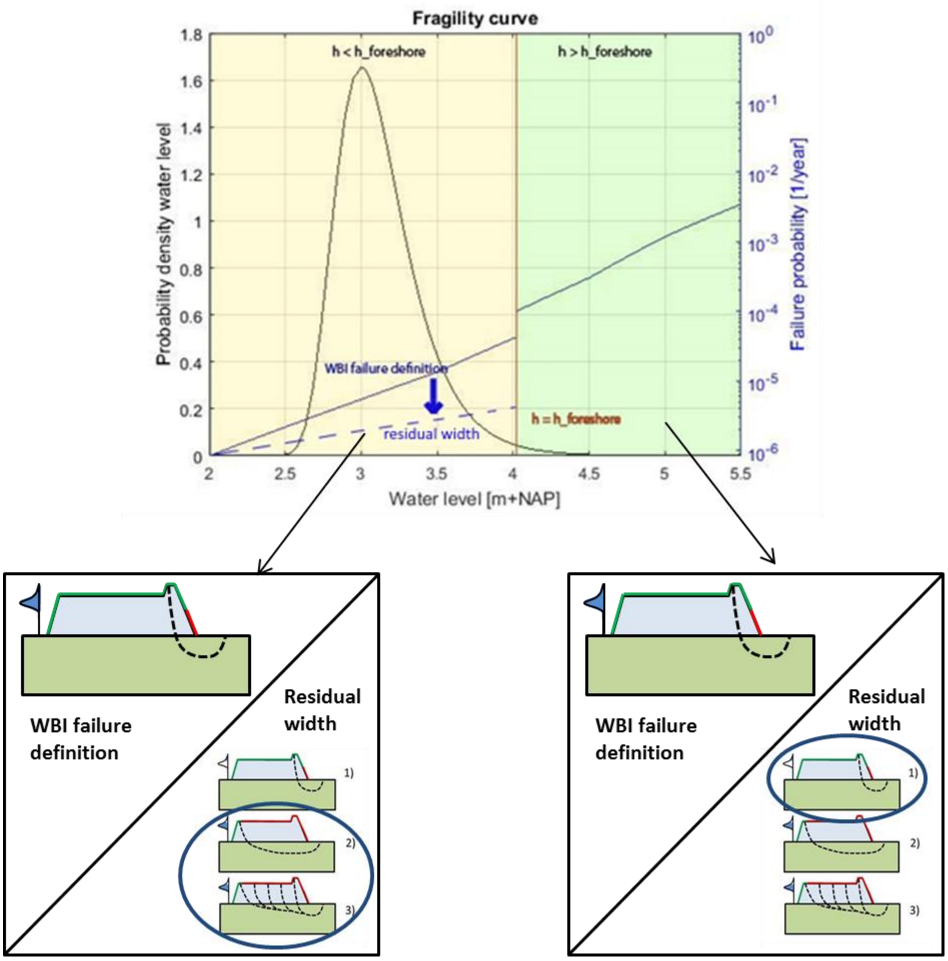


Figure 6.8: Illustration of the probabilistic calculation taking into account the residual width: for water levels above foreshore the WBI failure definition is used (scenario 1) and for levels below foreshore scenario 2 and 3 are used

The next step in the calculation is the analysis of scenario 2 (long and deep sliding plane) and scenario 3 (progressive failure). The calculation choices of these analyses have already been discussed in Section 4.5.4

As mentioned in Section 4.5.4, the analysis will be performed using only one water level: the water level for which the foreshore is just not flooded, in this case at 4.02m+NAP (the level of the foreshore). As mentioned before, the probabilities for progressive failure are determined first. Then, the probabilities for one long sliding plane are determined (see Example 6.1).

The slip surfaces of the progressive failure analysis for Test Case 4 are shown in Figure 6.9. As explained in Section 4.5.4 a stop criterion was established for this analysis, of β = 6.44. Once the reliability is higher than this value, the failure probability is negligible. Figure 6.9 shows that this is the case already after the third slide. However, as explained in Appendix B, at that point, a different shaped slip surface would still be possible. This is shown as the “last step”, which indeed has a lower reliability than the third slide (and does not exceed the stop criterion) and results in an extra loss of foreshore of 7m. It is assumed that after every individual slide (shown in Figure 6.9), a slip surface like this, taking away another 7m of foreshore, is possible without increasing the reliability (see Appendix B). Therefore, the amounts of foreshore losses, resulting from the progressive failure analysis, are for every soil slide equal to the corresponding amount shown in Figure 6.9 plus the amount of the "last step". These results are shown in Table 6.3.

Note: The foreshore loss uses the starting point of the foreshore (at the land side) as reference. A negative loss therefore means that the slip surface enters in the crest of the dike, and not yet in the foreshore.

Slide ID	β scenario 3 [-]	Foreshore loss [m]
1	3.93	-1.5
2	5.29	3.5
3	6.48	7.0

Table 6.3: Results scenario 3 (progressive failure) for Test Case 4, showing amounts of foreshore loss and the corresponding reliabilities

As a next step, for the three steps in Table 6.3, it has to be determined what the probability is of reaching the same amounts of foreshore loss due to one long sliding plane. The slip surfaces used for this analysis are shown in Figure 6.10. The reliability of the second step in scenario 2 is already higher than the stop criterion. Therefore, the reliability of the third step does not have to be calculated, because this will be even higher. The results of the analysis of scenario 2 are shown in Table 6.4.

ID	β scenario 2 [-]	Foreshore loss [m]
1	5.01	-1.5
2	6.68	3.5
3	-	7.0

Table 6.4: Results scenario 2 (one long sliding plane) for Test Case 4, showing amounts of foreshore loss and the corresponding reliabilities

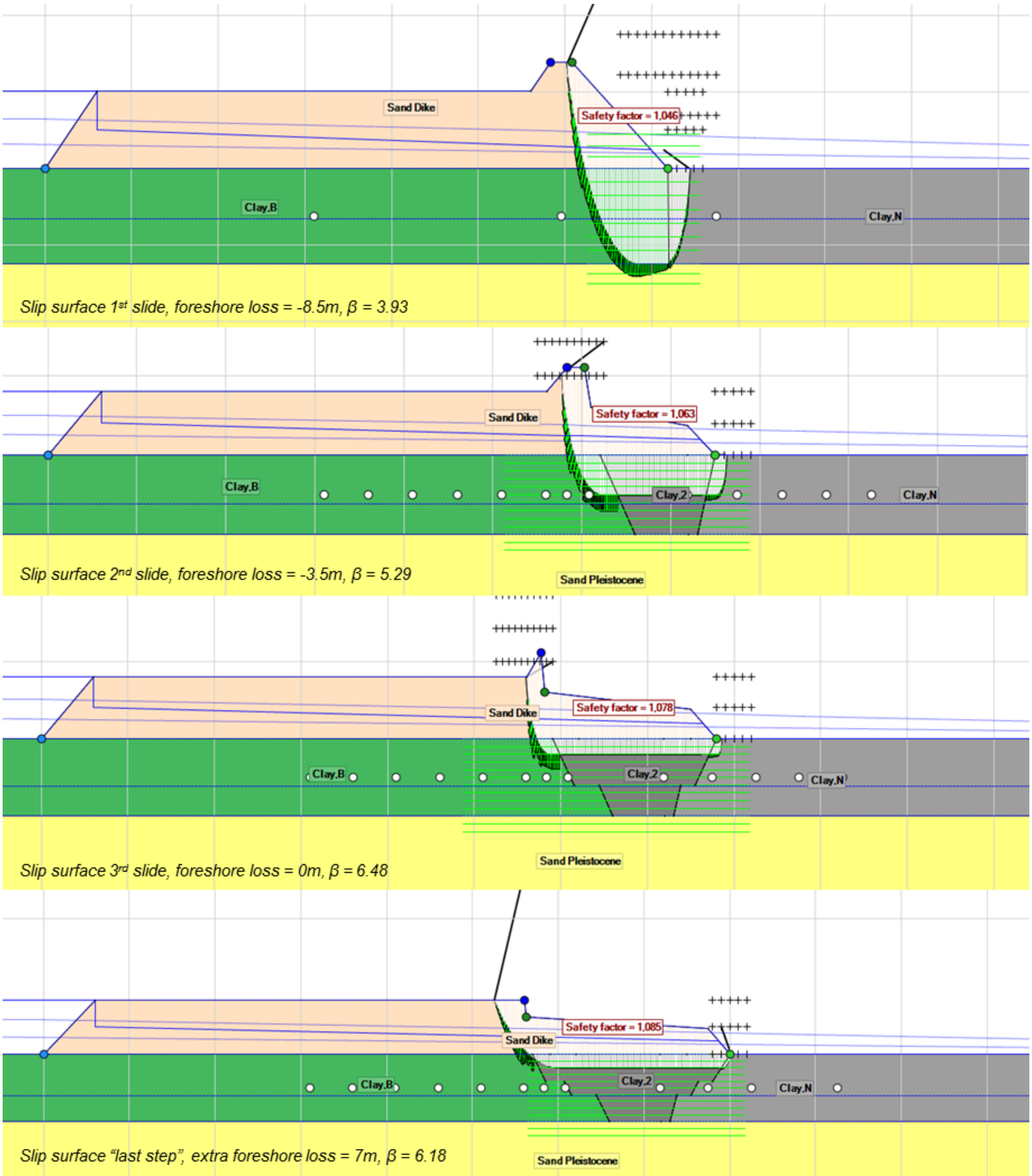


Figure 6.9: Results of the progressive failure analysis (scenario 3) of Test Case 4, showing the slip surfaces of the individual soil slides (taken from D-GeoStability)

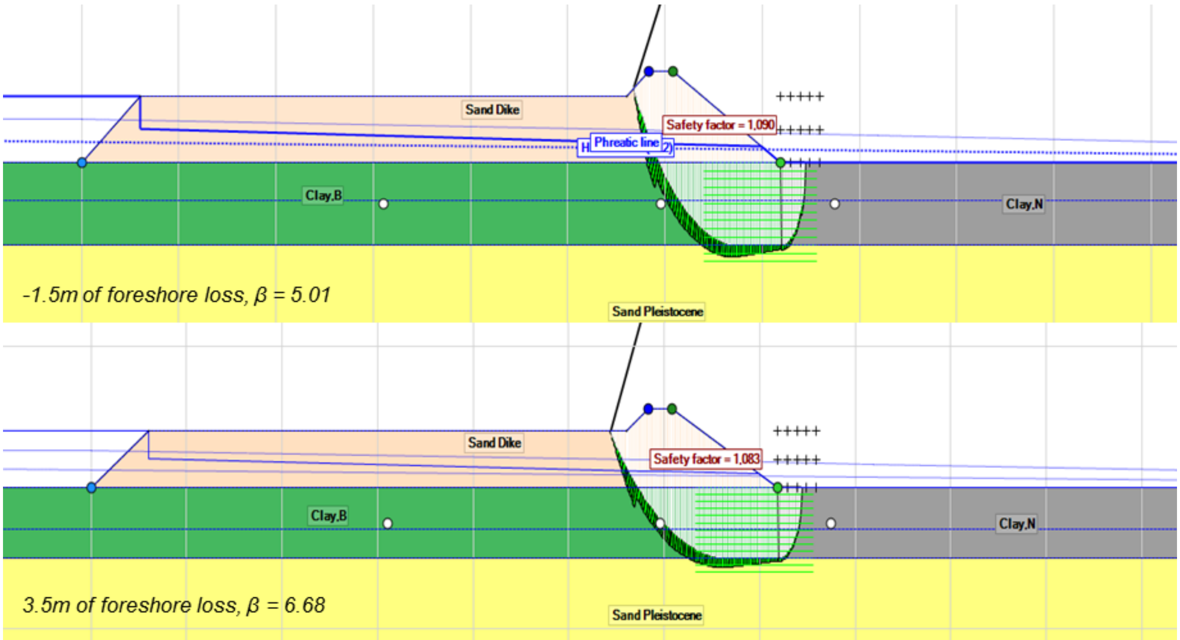


Figure 6.10: Results of the analysis of scenario 2 for Test Case 4, showing the slip surfaces that are used (taken from D-GeoStability)

Comparing the results in Table 6.3 and Table 6.4, it can be concluded that for Test Case 4, scenario 3 (progressive failure) is dominant for all steps, as the reliabilities of scenario 3 are lower than those of scenario 2 (and thus the probabilities are higher). The final results of the residual width analysis are shown in Table 6.5. From the table, it can be concluded that a foreshore loss of 7m already corresponds to a negligible probability. Note that Test Case 4 has a foreshore of 100m, so the conditional failure probability for the water level used in the assessment (4.02m+NAP) is negligible. In the analysis only one water level was used, but as described in Section 4.5.4, it is assumed that the conditional failure probabilities for the other water levels below foreshore level are equal.

ID	Foreshore loss [m]	β [-]	Probability [-]
1	-1.5	3.93	4.25E-05
2	3.5	5.29	6.12E-08
3	7	6.48	4.59E-11

Table 6.5: Results residual width analysis, showing the amounts of foreshore loss and the corresponding dominant reliability (lowest of scenario 2 and 3) and dominant probability

Concluding the analysis, the conditional failure probabilities for water levels below foreshore level are all negligible for Test Case 4. This means that the fragility curve of the yellow part is zero, after taking into account residual width(see Figure 6.11). Integration of that part therefore also leads to a probability of zero. As mentioned before, the green part is still the same as for the WBI failure definition.

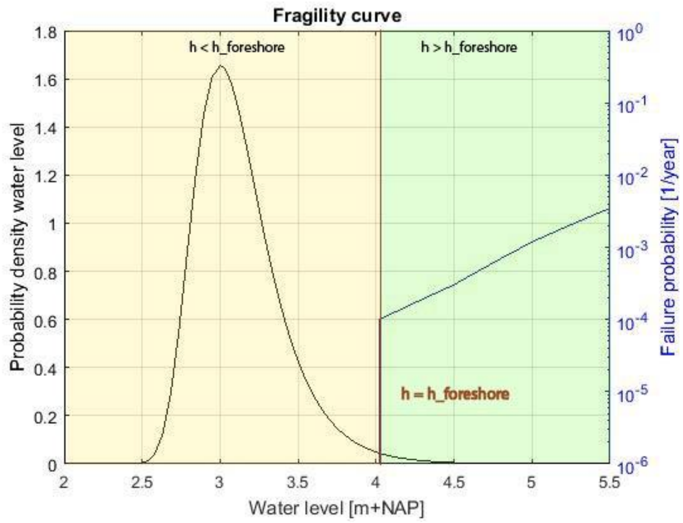


Figure 6.11: Fragility curve of Test Case 4 after taking into account residual width. Distinction has been made between water levels above and below foreshore

The final results after taking into account residual width are shown in Table 6.6. The table shows that the water levels below foreshore contribute nothing to the total failure probability. The contribution of the higher water levels, which is therefore equal to the total failure probability, is $2.08*10^{-6}$. Note that this contribution is the same as in Table 6.2, because this part of the fragility curve has not changed. This means that for Test Case 4, the dike is significantly safer for water levels below foreshore level than for higher water levels. Besides, comparing Table 6.6 to Table 6.2 shows that using the WBI failure definition, and thus neglecting residual width, leads to overestimation of the total failure probability; taking into account residual width leads to a total failure probability which is approximately four times smaller.

P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
2.08E-06	0	2.08E-06

Table 6.6: Results of Test Case 4 after taking into account residual width, showing amounts of foreshore loss and the corresponding reliabilities

6.4. Results

In this section, the results of the calculations for all test cases are discussed. First, the results using the WBI failure definition are covered, followed by the results after taking into account residual width.

6.4.1. WBI failure definition

This section covers the results of the calculations using the WBI failure definition for all water levels. The results of the individual calculations (including the fragility curves) can be found in Appendix F. The resulting failure probabilities are shown in Table 6.7 and Table 6.8, for the basic water level distribution (α = 4.5) and the narrow distribution (α = 7.5) respectively. Like in the example (Section 6.3),the tables show both the total failure probability and the contributions of water levels below and above foreshore to the total failure probability.

Test Case	P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
1	2.97E-02	2.88E-02	9.55E-04
2	8.36E-04	8.20E-04	1.51E-05
3	2.60E-03	2.20E-03	4.00E-04
4	9.79E-06	7.68E-06	2.08E-06
5	1.26E-05	1.22E-05	3.30E-07
6	6.82E-06	6.10E-06	7.11E-07
7	9.34E-06	9.29E-06	4.34E-08

Table 6.7: Results of all test cases using WBI failure definition for α = 4.5, showing the total failure probability and the contributions of the water levels below foreshore and above foreshore to the total failure probability

Test Case	P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
1	2.77E-02	2.77E-02	4.15E-05
2	7.82E-04	7.81E-04	6.73E-07
3	9.26E-04	9.08E-04	1.77E-05
4	6.66E-06	6.59E-06	6.90E-08
5	1.13E-05	1.13E-05	1.42E-08
6	4.62E-06	4.61E-06	5.91E-09
7	8.11E-06	8.11E-06	4.24E-10

Table 6.8: Results of all test cases using WBI failure definition for α = 7.5, showing the total failure probability and the contributions of the water levels below foreshore and above foreshore to the total failure probability

Appendix F describes for every test case and for every water level the entrance point of the critical slip surface. The entrance point of all critical slip surfaces lies in the middle of the crest or further to the land side. For the water levels below foreshore level, this means that there are large amounts of residual width, because all test cases have a foreshore of 100m. This means that, taking into account the residual width, the conditional failure probabilities of the water levels below foreshore level will be lower. In the example (Section 6.3) this was already explained. The failure probabilities after taking into account residual width will be determined in the next section.

6.4.2. Residual Width

The result of residual width analysis is a list of amounts of foreshore loss and a corresponding probability (see the example in Section 6.3). The individual results can be found in Appendix G. Figure 6.12 shows the results in a scatter plot. As can be seen in the figure, for all cases the stop criterion (negligible failure probability) is reached before the complete foreshore of 100m is lost. This means that for all test cases, the conditional failure probabilities for water levels below foreshore level are negligible. Only for Test Case 1 a substantial loss of foreshore could be possible: a loss of 80m can be reached with a probability which is not negligible (β < 6.44). However, note that this case has a very weak subsoil (see Section 4.3). For the other test cases, the maximum loss of foreshore is less than 20m. This means that for these test cases, a foreshore smaller than 100m would also lead to a negligible conditional failure probabilities; a foreshore of 20-25m wide would suffice.

Furthermore, Appendix G shows that for almost every step the progressive failure probability is dominant over one long sliding plane. However, note that the progressive failure probability is based on some (conservative) assumptions. If one would, based on further research, use more realistic modelling of progressive failure, it may be possible that one long sliding plane (scenario 2) becomes dominant.

Note: The stop criterion is based on an absolute lower limit of the target probability (see Appendix B). It is assumed that the conditional failure probabilities are negligible when they are 1% of the target probability or less. This means that if the target probability of the dike is higher than the lower limit (less strict), a negligible failure probability will be reached even sooner, and even less foreshore is needed for this.

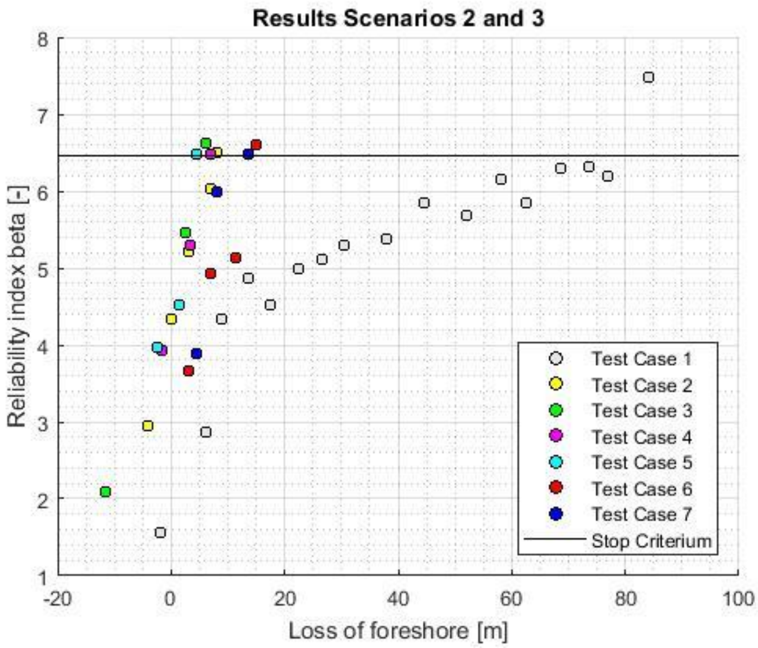


Figure 6.12: Scatter plot of the residual width analysis: showing for all test cases the amounts of foreshore loss and the corresponding reliability indexes

After taking into account the residual width in the probabilistic calculations, the failure probabilities are as shown in Table 6.9 and Table 6.10. The conditional failure probabilities for the water levels below foreshore are negligible for all test cases. Integration over these water levels leads therefore to a failure probability of zero, which means that these water levels contribute nothing to the total failure probability. As explained in the example (Section 6.3), the contribution of the water levels above foreshore level is the same as in the calculation using the WBI failure definition (Table 6.7 and Table 6.8).

Test Case	P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
1	9.55E-04	0	9.55E-04
2	1.51E-05	0	1.51E-05
3	4.00E-04	0	4.00E-04
4	2.08E-06	0	2.08E-06
5	3.30E-07	0	3.30E-07
6	7.11E-07	0	7.11E-07
7	4.34E-08	0	4.34E-08

Table 6.9: Results of all test cases after taking into account residual width for α = 4.5, showing the total failure probability and the contributions of the water levels below foreshore and above foreshore to the total failure probability

Test Case	P_f (total)	P_f ($h < h_{foreshore}$)	P_f ($h > h_{foreshore}$)
1	4.15E-05	0	4.15E-05
2	6.73E-07	0	6.73E-07
3	1.77E-05	0	1.77E-05
4	6.90E-08	0	6.90E-08
5	1.42E-08	0	1.42E-08
6	5.91E-09	0	5.91E-09
7	4.24E-10	0	4.24E-10

Table 6.10: Results of all test cases after taking into account residual width for α = 7.5, showing the total failure probability and the contributions of the water levels below foreshore and above foreshore to the total failure probability

Note: In the test cases considered, the conditional failure probabilities for water levels below foreshore level, after taking into account residual width, are negligible. If a situation would be found in which this is not the case (e.g. in case of a very weak subsoil and small foreshore), the fragility curve in the yellow part (Figure 6.11) is not equal to zero. However, the fragility curve can be determined based on the conditional failure probability that was found in the analysis of scenario 2 and 3: the probability of losing an amount of foreshore leading to a situation in which there is less than 3m of remaining width (defined as failure, see Appendix B). As it is assumed that all water levels below foreshore have an equal conditional failure probability, the fragility curve in the yellow part will be a horizontal line at this probability.

6.5. Conclusion

In the introduction of this chapter, the following hypotheses were stated:

- Taking into account residual width, the dike is significantly safer for water levels below the level of the foreshore than for higher water levels;
- Using the WBI failure definition, and thus neglecting the residual width, leads to overestimation of the failure probability.

Based on the results it can be concluded that, using the WBI failure definition, the dike is less safe for water levels below foreshore level than for higher water levels; the water levels below foreshore contribute more to the total failure probability than the water levels above foreshore (see Table 6.7 and Table 6.8).

After taking into account the residual width, the contribution of the water levels below foreshore level to the total failure probability decreases significantly, even to zero. In this case, the dike is significantly safer for water levels below foreshore than for higher water levels (see Table 6.9 and Table 6.10). Therefore, the first hypothesis is true, but only if one takes the residual width into account in the calculations.

Comparison of Table 6.7 and Table 6.8 on the one hand and Table 6.9 and Table 6.10 on the other hand shows that there is profit in taking into account the residual width. The contribution of the water levels below foreshore level to the total failure probability decreases significantly, causing the total failure probabilities to decrease significantly too. Based on this, it can be concluded that using the WBI failure definition leads to overestimation of the failure probability for dikes with foreshores. This means that the second hypothesis is also true.

Finally, for the test cases used in this Thesis, after taking into account residual width, the conditional failure probabilities for water levels below the level of the foreshore are negligible. Therefore, only the water levels above foreshore level contribute to the total failure probability. In all test cases used in this Thesis, a foreshore of 100m is present. However, the research showed that for most of the test cases, a foreshore of 20 to 25m would already lead to negligible conditional failure probabilities for the water levels below foreshore level. For these amounts of foreshore, the total failure probabilities would be the same (apart from some small differences due to a different development of the phreatic line). This shows that a relative small foreshore already leads to a significant reduction in failure probability.

7. Influence foreshore

7.1. Introduction

This chapter refers to the fifth research sub-question of this thesis: "*What is the influence of the presence of a foreshore on the failure probability for macro-stability of a dike?"*

Chapter 6 already concluded that a dike is significantly safer for water levels below the level of the foreshore than for higher water levels. The presence of a foreshore increases the safety of the dike. Based on that, there is the hypothesis that the presence of a foreshore has a significant influence on the failure probability of a dike, compared to a dike without a foreshore. Besides, in Section 1.2 it was predicted that the current semi-probabilistic assessment rule for macro-stability would lead to a very conservative result for dikes with high and wide foreshores, because water levels below the level of the foreshore, for which the dike is significantly safer (the conditional failure probabilities are lower), are not included in the semi-probabilistic assessment.

This chapter will study the influence of the presence of a foreshore on the total failure probability. Besides, the influence of different calculation methods on the resulting failure probability will be determined, by comparing the results of these different calculations. The different calculations are shown in the calculation matrix of Figure 7.1.

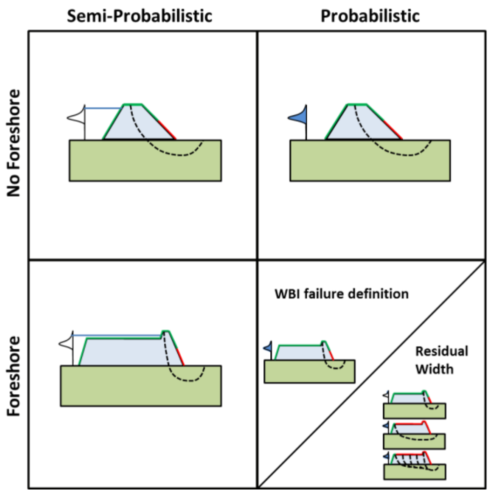


Figure 7.1: Calculation matrix showing all calculations types performed in this Thesis

7.2. Approach

Up to this point in the report, all calculations shown in the matrix of Figure 7.1 have been performed, except for one: the semi-probabilistic calculations of the dikes with foreshores. These calculations will be performed as a first step in this chapter. The calculations are based on the calculation choices as specified in Section 4.5. The next section will give an example of the semi-probabilistic calculations.

Once the results of all different calculations are known, they are compared to each other, to study the influence the different calculation types have on the resulting failure probabilities.

7.3. Example

This section will cover the semi-probabilistic calculation of a dike with foreshore for Test Case 4 as an example. This test case was already used as an example in Chapters 4, 5 and 6.

The semi-probabilistic calculation is performed using a design water level (5.04m+NAP, see Section 4.7.1). For the other parameters, design values are used too, as described in Section 4.7.2.

Figure 5.3 shows the result of the semi-probabilistic calculation in the D-GeoStability software. The critical slip surface is shown. The corresponding factor of safety (FoS) is 1.065. This FoS can be transformed into an approximated failure probability using the calibrated formula (2.20):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right)$$

Using the FoS = 1.065 and γ_d=1.06 (model factor Uplift Van), this leads to an annual failure probability of Pf= $3.67*10^{-5}$ (β = 3.965).

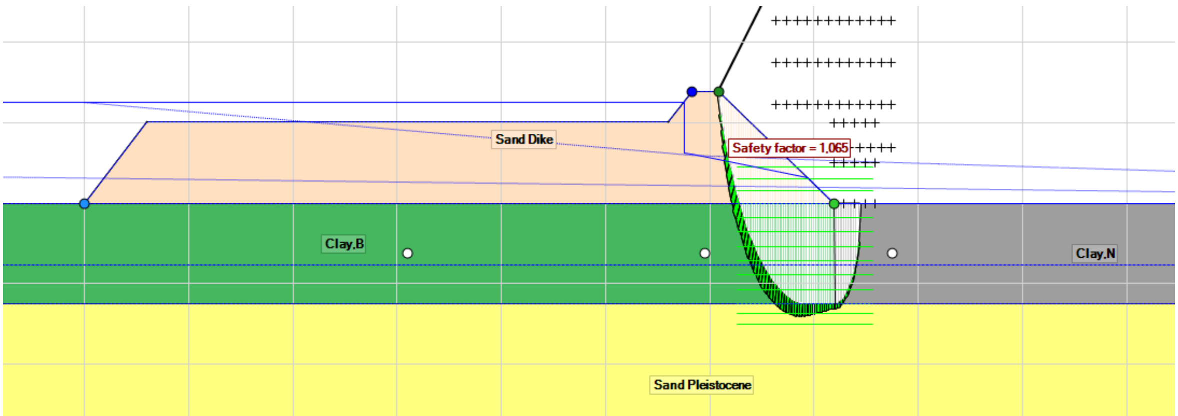


Figure 7.2: Result semi-probabilistic calculation Test Case 4, showing the critical slip surface and the corresponding FoS (taken from D-GeoStability)

7.4. Results

This section describes the results of the research performed in this chapter. First, the results of the semi-probabilistic method using dikes with foreshores are covered. Then, the different calculations are compared to each other.

7.4.1. Results semi-probabilistic calculation with foreshore

This section covers the results of the calculations using the semi-probabilistic method for dikes with foreshores. Figure 7.3 shows this calculation in the calculation matrix. The results are shown in Table 7.1 and Table 7.2 for the basic water level distribution (α = 4.5) and a narrower water level distribution (α = 7.5) respectively.

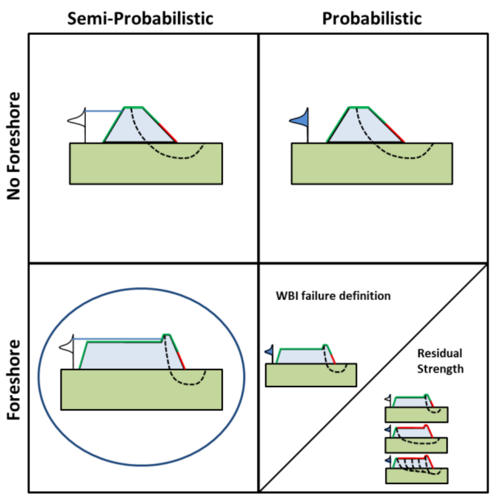


Figure 7.3: Calculation matrix showing the calculation type section 7.4.1. refers to

Test Case	Failure probability [-]
1	6.56E-03
2	2.02E-04
3	2.26E-03
4	3.67E-05
5	1.24E-05
6	2.46E-05
7	1.02E-05

Table 7.1: Failure probabilities semi-probabilistic method for all test cases with foreshores for α = 4.5

Test Case	Failure probability [-]
1	2.97E-03
2	1.52E-04
3	1.09E-03
4	8.61E-06
5	9.12E-06
6	3.51E-06
7	8.14E-06

Table 7.2: Failure probabilities semi-probabilistic method for all test cases with foreshores for α = 7.5

At this point, all the results of the different calculations are known. In the next sections the results will be compared.

Note:

Test Case 6 and Test Case 7 of Table 7.2, based on a water level distribution with less spreading (α = 7.5) are special cases. For these cases the design water level is below the level of the foreshore, as can be seen in Figure 7.4, which shows the result of Test Case 6 in the D-GeoStability software. This means that for the design water level there is a lot of residual width.

The WBI proposes a simple geometry check to verify whether macro-stability could be relevant for this case. However, one of the requirements in this check is that the water level must be lower than 2/3 of the dike height, which is not the case (design water level = 4.09m+NAP, dike height = 5.3m+NAP). There is no other way to take the residual width into account in a semi-probabilistic calculation. Therefore, the WBI failure definition is used, just like for the other cases. The slip surface shown in the figure is therefore considered as failure.

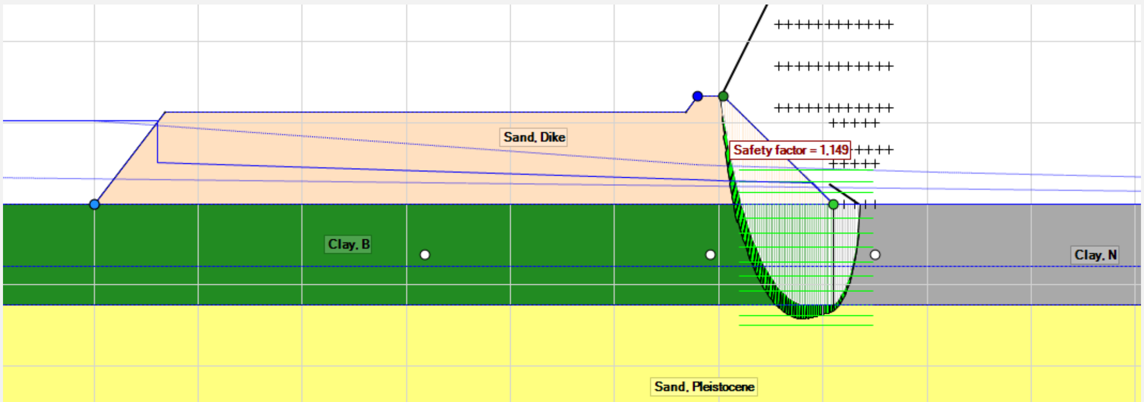


Figure 7.4: Result semi-probabilistic calculation Test Case 6 with foreshore (alpha = 7.5)

7.4.2. Semi-Probabilistic with foreshore vs. Semi-probabilistic without foreshore

Figure 7.5 shows the comparison between the semi-probabilistic calculations, both for cases with foreshore and without foreshore. The figure shows a scatter plot of the resulting failure probabilities. The plot also shows the line of equal probability. If a point is on this line, both calculations result in the same probability. Furthermore, for more clarity, the calculation matrix shows which calculations are compared.

Figure 7.5 shows that there is almost no difference between the two calculations; performing a semi-probabilistic calculation on a dike with foreshore and a dike without foreshore results in more or less equal failure probabilities. In other words: the semi-probabilistic method does not take the foreshore into account.

Note: As mentioned in Section 4.6, the standard phreatic line schematization proposed by the WBI was used in the calculations, and the time-dependence of the phreatic line development was not taken into account. For dikes with foreshores, this means that for a (design) water level above foreshore, the foreshore is assumed to be completely saturated, which is a conservative simplification. If one would take into account the time-dependence of the phreatic line development of the dikes with foreshores, the points in the scatter would probably shift in downward direction (decreased probability for dikes with foreshores) and the presence of the foreshore thus would have some influence on the failure probabiliy, as the points would then be below the line of equal probability.

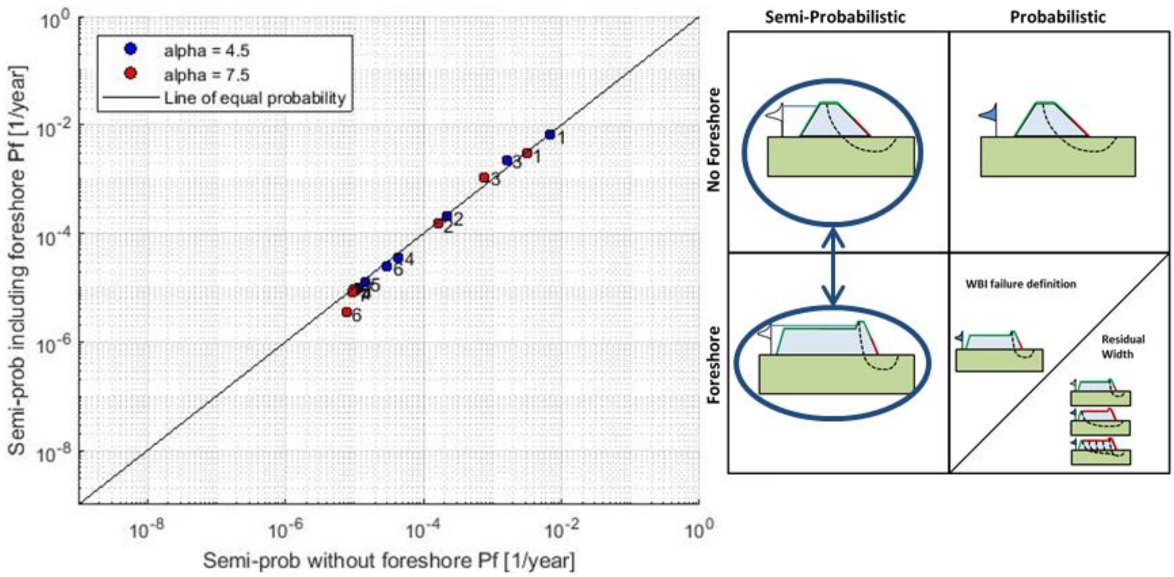


Figure 7.5: Scatter plot failure probabilities semi-probabilistic method with foreshore vs semi-probabilistic method without foreshore

7.4.3. Probabilistic with foreshore (WBI failure definition) vs. Probabilistic without foreshore

Figure 7.6 shows the comparison between the probabilistic calculations for dikes without foreshores and the probabilistic calculations for dikes with foreshores using the WBI failure definition for all water levels.

The figure shows that the failure probabilities for dikes with foreshores (using the WBI failure definition) are a little lower than the failure probabilities for dikes without foreshores. Based on this, it can be concluded that although the WBI failure definition overestimates the failure probability for dikes with foreshores, as concluded in Chapter 6, it still takes into account some influence of the foreshore.

Note: In Figure 6.6, this influence of the foreshore was already found as a step in the fragility curve. This step is probably caused by the phreatic line and the aquifer head, which for water levels below foreshore develop over a longer distance (100m extra due to the foreshore) than for higher water levels.

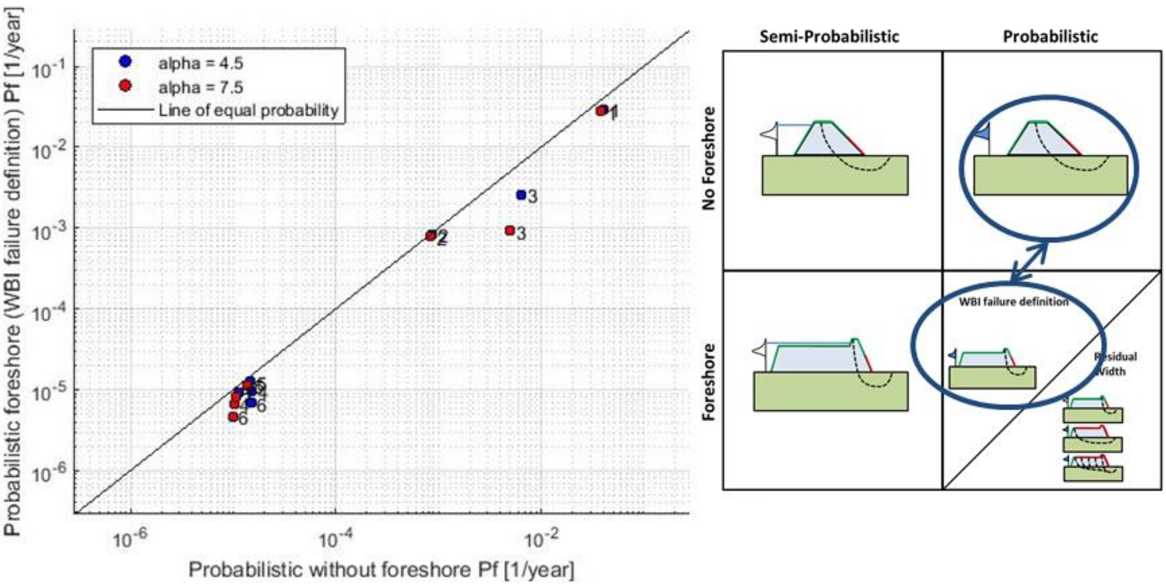


Figure 7.6: Scatter plot failure probabilities probabilistic method with foreshore (using WBI failure definition) vs probabilistic method without foreshore

7.4.4. Probabilistic with foreshore (Residual width) vs. Probabilistic without foreshore

Figure 7.7 shows the comparison between the probabilistic calculations for dikes without foreshores and the probabilistic calculations for dikes with foreshores after taking into account residual width.

The figure clearly shows that the failure probabilities for dikes with foreshores, after taking into account residual width, are significantly smaller than for dikes without foreshore. For the basic water level distribution (α = 4.5) the presence of a foreshore leads to a maximum reduction of the failure probability (compared to a dike without foreshore) of a factor 263 (for test case 7). This corresponds to a difference in reliability index of Δβ = 1.12. For the narrower water level distribution (α = 7.5) the maximum reduction is even higher: a factor of 25,336 (Δβ = 1.88). This shows that the presence of a foreshore has a significant influence on the failure probability of a dike.

Figure 7.7 also shows that the narrow water level distribution (α = 7.5) leads to more reduction in failure probability than the basic water level distribution, as the points are further away from the line of equal probability. This can be explained by the fact that the probabilities of occurrence of high water levels are lower for the water level distribution with less spreading. As was concluded in Chapter 6, for the test cases considered in this Thesis, only the water levels above the level of the foreshore contribute to the total failure probability. The probabilities of occurrence of these water levels are lower for the water level distribution with less spreading, leading to a smaller failure probability.

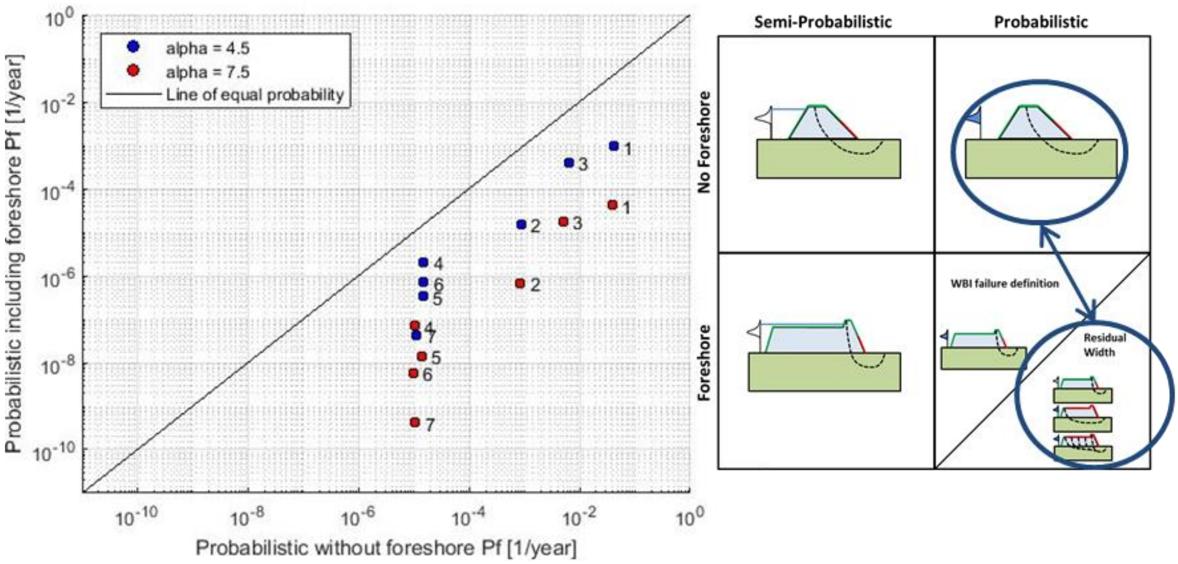


Figure 7.7: Scatter plot failure probabilities probabilistic method with foreshore (taking into account residual width) vs probabilistic method without foreshore

Figure 7.8 shows the results again, only for the basic water level distribution (α = 4.5). In this figure, the results are differentiated based on dike and foreshore material and based on the foreshore height. The left figure shows the differentiation based on dike material. Based on this figure, it can be concluded that the influence of the foreshore is higher for dikes and foreshores out of clay than for dikes out of sand, as the reduction of the failure probability is bigger. This can be explained by the fact that clay dikes have a relatively gentle fragility curve (see Appendix F). Water levels above foreshore level then have a relatively low conditional failure probability (just like the water levels below foreshore). As concluded in Chapter 6, only the water levels above foreshore contribute to the total failure probability. As their corresponding conditional failure probabilities are relatively small, the total failure probability is small too. For sand dikes, the fragility curves are steeper, which means that the conditional failure probabilities for higher water levels are higher, causing the total failure probability to be higher too.

The right figure shows the differentiation based on the foreshore height. Note that Test Case 6 and Test Case 7 are the same as Test Case 4 and Test Case 5 respectively, except for the foreshore height, which has been increased for Test Cases 6 and 7. The figure shows that a higher foreshore leads to a larger influence of the foreshore; the reduction of the failure probability (compared to a dike without foreshore) is bigger. This can be explained by the fact that for a higher foreshore there are less water levels above foreshore level, so there are also less water levels that actually contribute to the total failure probability, causing the total failure probability to decrease.

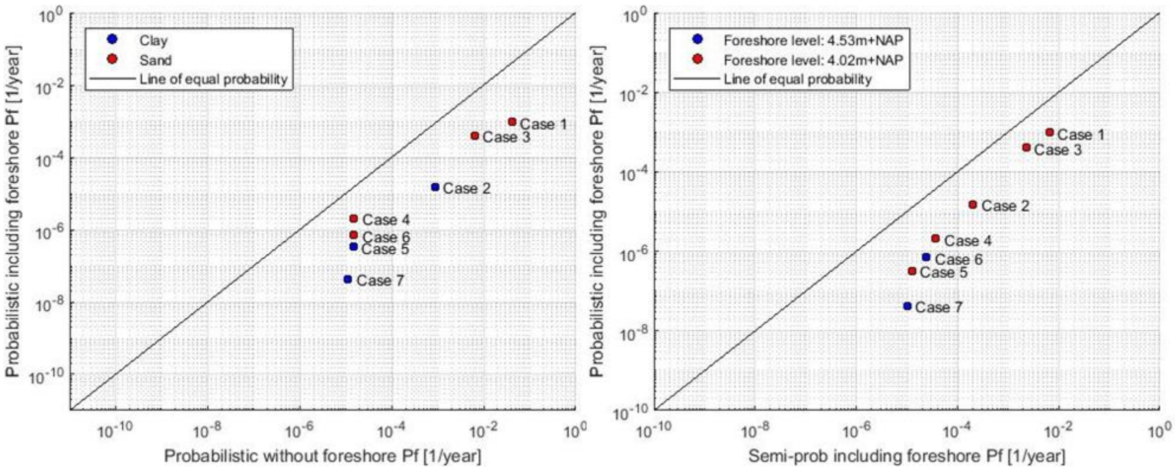


Figure 7.8: Scatter plot failure probabilities probabilistic method with foreshore (taking into account residual width) vs probabilistic method without foreshore, differentiated based on dike material and foreshore height

7.4.5. Probabilistic with foreshore (Residual width) vs. Semi-Probabilistic with foreshore

Figure 7.9 shows the comparison between the fully probabilitic calculations (after taking into account residual width) and the semi-probabilistic calculations, both for dikes with foreshores.

The figure shows that the semi-probabilistic method can lead to very conservative results for dike with high foreshores, as predicted in Section 1.2. The probabilistic failure probabilities are significantly lower than the semi-probabilistic probabilities, meaning that the semi-probabilistic method overestimates the failure probabilities. The maximum overestimation caused by the semi-probabilistic method for the basic water level distribution (α = 4.5) is a factor 235 (for Test Case 7). This corresponds to $\Delta\beta$ = 1.09, meaning that the semi-probabilistic method results in a reliability index which is 1.09 lower than in the fully probabilistic calculation. For the water level distribution with reduced spreading (α = 7.5) the maximum overestimation is a factor 19,192 ($\Delta\beta$ = 1.82). Furthermore, Figure 7.9 shows the same trends as Figure 7.7, which means that the same differentiations as used in Figure 7.8 are possible.

Note: The differences between the two calculations in Figure 7.9 are a little smaller than the differences in Figure 7.7. This can be explained by the fact that the semi-probabilistic rule for dikes in general for most test cases leads to underestimation of the failure probability (as concluded in Chapter 5). Therefore, the semi-probabilistic probabilities (both for dikes with and without foreshores) are for most cases smaller than the probabilistic probabilities for dikes without foreshores.

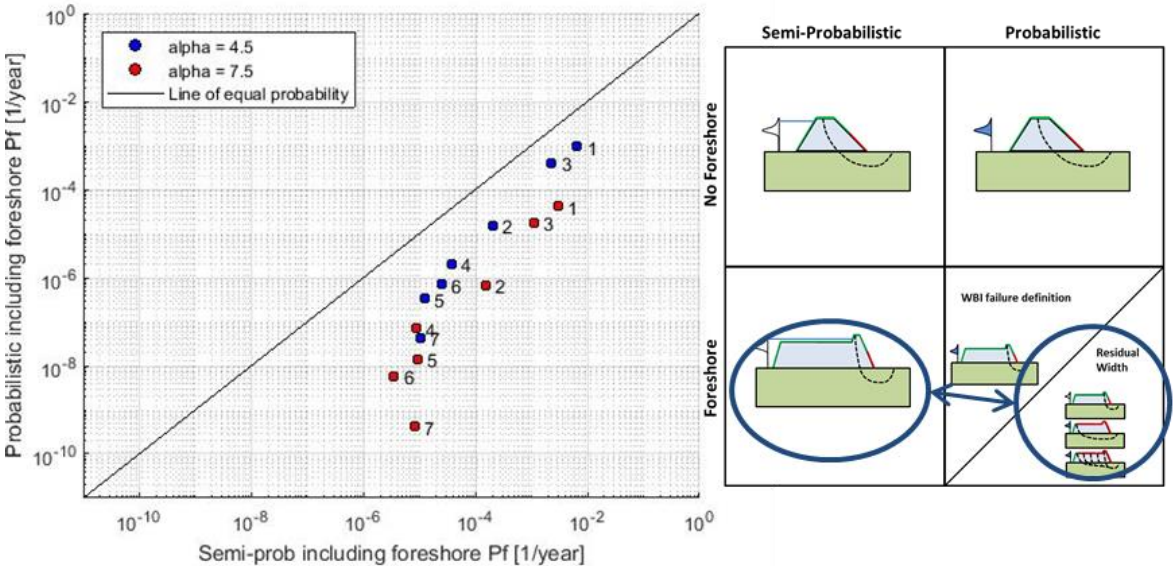


Figure 7.9: Scatter plot failure probabilities probabilistic method with foreshore (taking into account residual strength) vs semi-probabilistic method width

Figure 7.10 shows the results again, this time in a scatter plot of the reliability index β (result of the probabilistic calculation), against the safety factor γ_n (result of the semi-probabilistic calculation). This approach was already used in Section 5.4 for the dikes without foreshore, which are shown in red. The dikes with foreshore are shown in green. The points that were originally used for the calibration of the semi-probabilistic rule (Section 2.4.5) are shown in blue. The black line shows the calibrated (β-γ_n)-relation for the semi-probabilistic assessment (20%-fit). As mentioned before, for most test cases without foreshores, the semi-probabilistic method (calibrated rule) leads to an underestimation of the failure probability (points on the left of the black line). For all test cases with foreshores, the semi-probabilistic method leads to a conservative result (the points are on the right of the line).

All points, both for the test cases with and without foreshores, are within the scatter of the original calibration study. However, remember that the red points and the green points are actually the same cases, except for the presence/absence of a foreshore. Including a foreshore in the calculation leads to a significant shift to the right (higher reliabilities, lower failure probabilities).

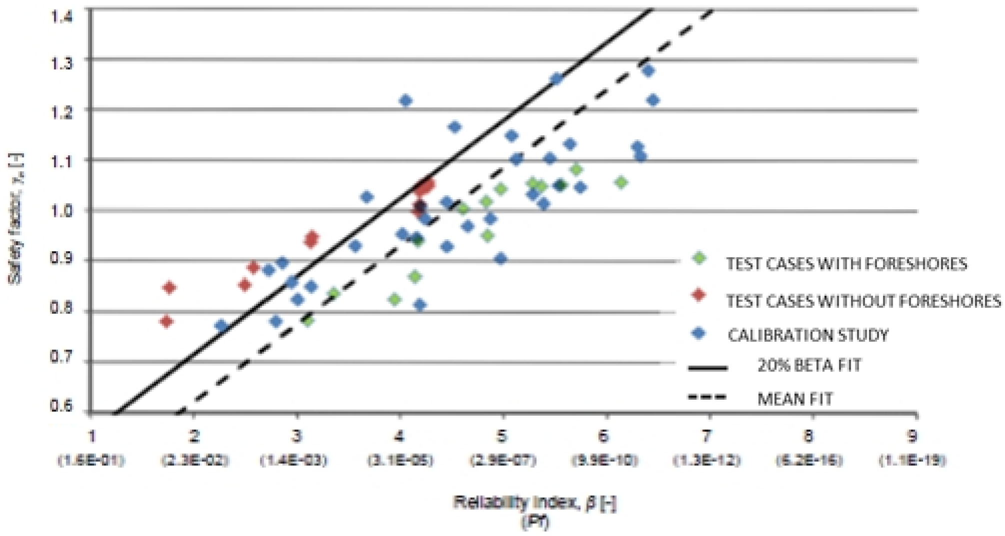


Figure 7.10: Scatter plot of the results of the semi-probabilistic method vs the probabilistic method for both dikes with and without foreshores calculated in this Thesis, and the scatter of the original calibration study

7.4.6. Influence conservative semi-probabilistic method on dike design

The previous section concluded that for dikes with foreshores, the semi-probabilistic method can lead to very conservative failure probabilities. As mentioned before, overestimations of the failure probability with a factor of 235 (maximum for water level distribution with α = 4.5) and even 19,192 (maximum for water level distribution with α = 7.5) were found. However, failure probabilities are a bit abstract; it is hard to immediately understand the impact these overestimations have on dike assessments and dike design. This section tries to give a better understanding of the impact the conservatism in the semi-probabilistic method has on a dike.

As concluded before, the semi-probabilistic method does not take the presence of the foreshore into account. It therefore does not account for the extra safety the foreshore adds to the dike. To reach the same amount of safety as in the fully probabilistic calculations, which do take into account the presence of the foreshore, one therefore has to apply a stability berm in the hinterland. This is illustrated in Figure 7.11.

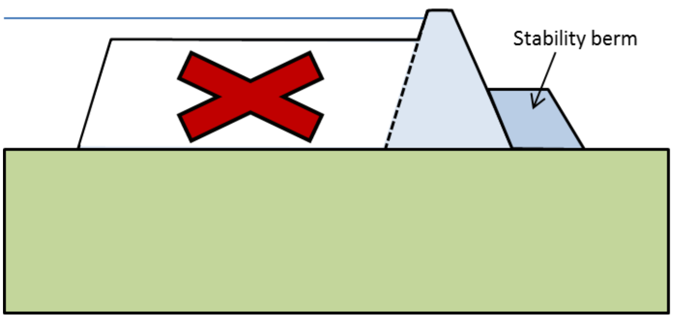


Figure 7.11: Illustration stability berm semi-probabilistic method

For example, for Test Case 7, using the basic water level distribution (α = 4.5), the semi-probabilistic method resulted in a failure probability of $1.02*10^{-5}$ per year. The fully probabilistic method, taking into account residual width, resulted in a failure probability of $4.34*10^{-8}$, which for now is assumed to be equal to the target probability, meaning that based on the probabilistic calculation the dike is just safe enough. However, if one only uses the semi-probabilistic method for the dike assessment, the dike will be assessed as unsafe. This means that the dike has to be reinforced by applying a stability berm.

As mentioned, the target probability of Test Case 7 is $4.34*10^{-8}$. The factor of safety (FoS), that is needed to reach this probability in the semi-probabilistic method, can be determined based on formula (2.20):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right)$$

Based on this formula, a FoS of at least 1.286 is needed to reach the target probability. Assuming a berm height of 2.5m, this leads to a berm width of at least 5.5m.

Performing the same approach for Test Case 7 using the narrower water level distribution (α = 7.5), the semi-probabilistic calculation should result in a FoS of at least 1.410 to reach the same failure probability as calculated using the fully probabilistic calculation ($P_f = 4.24*10^{-10}$). To reach this FoS, a berm of 8.5m wide is needed.

Concluding the above: if one uses the semi-probabilistic method for the assessment of dikes with high and wide foreshores, this could lead to significant reinforcements of the dike. However, if one would put in some more effort in the assessment and performs a fully probabilistic calculation, it may be possible that no reinforcement is needed at all. This proves that using the semi-probabilistic method for dikes with foreshores can lead to unnecessary expenses for dike reinforcements.

7.5. Conclusions

Based on the research performed in this chapter, it can be concluded that for the test cases considered in this Thesis, the presence of a foreshore has a large influence on the failure probability of a dike, compared to dikes without foreshore. This influence is bigger for clay dikes than for sand dikes. Besides, the influence increases for an increased foreshore height and for a decreased spreading in the water level distribution. The combination of the foreshore height and the water level distribution determines the probability of flooding of the foreshore. The lower this probability, the larger the influence of the presence of the foreshore on the failure probability.

The failure probabilities for dikes with foreshores are significantly smaller than for dikes without foreshores. However, this is only the case if one calculates the failure probabilities of dikes with foreshores using fully probabilistic calculations, and takes into account residual width. If one uses semi-probabilistic calculations, the presence of the foreshore is not taken into account at all: the failure probabilities for dikes with and without foreshores are more or less the same. If one uses probabilistic calculations using the WBI failure definition, the presence of the foreshore is (partly) taken into account, but this leads to only small differences compared to the dikes without foreshores.

Besides, it can be concluded that the semi-probabilistic method can lead to very conservative results for dikes with foreshores. The semi-probabilistic method overestimates the failure probabilities significantly. The research shows that if one uses the semi-probabilistic method for the dike assessments, this could lead to significant dike reinforcements, whereas, if one would use fully probabilistic calculations taking into account residual width, it may be possible that no reinforcements are needed at all. This proves that if one uses the semi-probabilistic method for the macro-stability assessment of dikes with high and wide foreshores, this could lead to unnecessary expenses for dike reinforcement.

8. Improvement Safety Assessment

8.1. Introduction

This chapter proposes possible improvements for the current safety assessment method for macro-stability of dikes with high and wide foreshores. This chapter refers to the last research sub-question of this Thesis: *"Can the current safety assessment method for macro-stability be improved, to account for the presence of a foreshore? If yes, what improvements can be made?"*

Chapter 7 already concluded that the current semi-probabilistic method can lead to significant overestimations of the failure probabilities for dikes with high foreshores. Using this method could therefore lead to unnecessary expenses for dike reinforcements. This proves the need for an improved assessment method, because currently the semi-probabilistic method is mostly used.

In this chapter, three possible improved methods are described. These methods are compared to determine what improvement will be best.

8.2. New Semi-Probabilistic Calibration

The semi-probabilistic method is a quick and easy-to-use method to assess the safety of a dike, and is therefore mostly used by the dike managers, who are responsible for the safety assessments of dikes. However, as mentioned before, the current semi-probabilistic method can lead to very conservative results for dikes with high foreshores. The dike managers therefore would profit from a new semi-probabilistic method for dikes with foreshores, which is less conservative, but still quick and easy to use.

Such a new semi-probabilistic method can be based on the same approach as the current one, but using a new calibrated (β-γ_n)-relation. The current method uses a relation, which is based on a calibration on the 20%-quantile of the reliabilities in the (β-γ_n)-scatter plot of Figure 8.1. This has led to the following calibrated relation (2.19):

$$\gamma_n = 0.15 * \beta_{T,cross} + 0.41$$

For the new semi-probabilistic method one could perform such a calibration again, only for dikes with foreshores. The scatter of dikes with foreshores will probably look similar to the scatter in Figure 8.1, but shifted to the right. After all, the results of the semi-probabilistic calculations (γ_n) for dikes with and without foreshore are more or less the same, as the semi-probabilistic method does not take the foreshore into account, whereas the reliabilities β, calculated using fully probabilistic calculations, are higher for dikes with foreshores (see Section 7.4.4). Performing such a calibration will lead to a new (β-γ_n)-relation. Because this new relation is calibrated specifically for dikes with foreshores, it will be not that conservative. However, the result of the semi-probabilistic method will still be an approximated failure probability based on a calibration, which can be either an underestimation or an overestimation of the failure probability. This is an important limitation of this method.

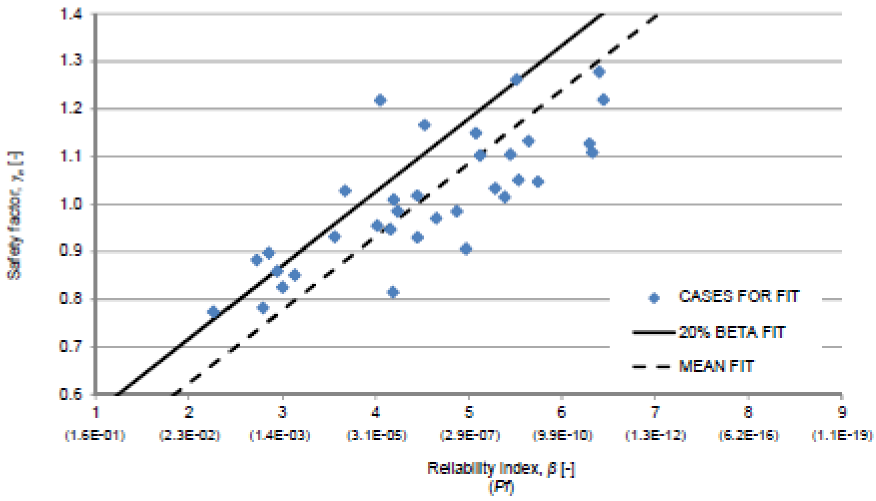


Figure 8.1: Scatter plot of the results of the calibration study (Kanning et al., 2016)

8.3. Two Assessment Rules

Another possibility for a new semi-probabilistic method is to create two separate assessment rules: one for water levels below foreshore level and one for water levels above foreshore level. This is in analogy to the approach that is used for the combination of macro-stability and overtopping. This approach is explained in Appendix I. The concept and theory behind the method of two separate assessment rules is explained below.

The principle of two separate semi-probabilistic assessment rules is based on the law of total probability. For the failure probability of a dike with foreshore, the law of total probability can be written as follows:

$$P_f = P_{f|h\leq h^*} * P_{h\leq h^*} + P_{f|h>h^*} * P_{h>h^*} \tag{8.1}$$

In which:

P_f	Total annual failure probability [-]
h^*	Level of the foreshore [m+NAP]
$P_{f\|h\leq h^*}$	Conditional failure probability given the water level is below foreshore [-]
$P_{h\leq h^*}$	Probability that the water level is below foreshore [-]
$P_{f\|h>h^*}$	Conditional failure probability given the water level is above foreshore [-]
$P_{h>h^*}$	Probability that the water level is above foreshore [-]

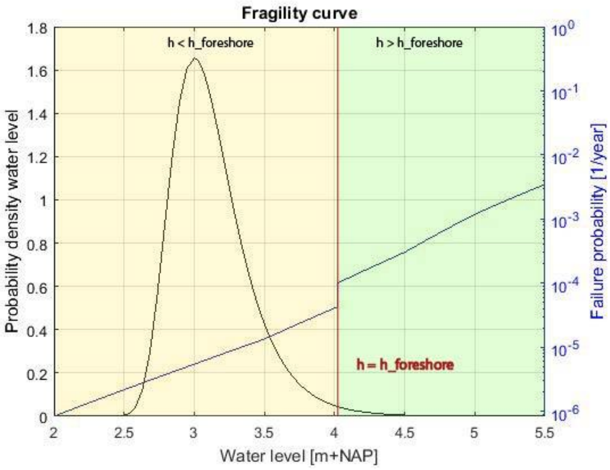


Figure 8.2: Fragility curve; water levels below foreshore shown in yellow, water levels above foreshore in green

Figure 8.2 shows the fragility curve of Test Case 4, together with the water level distribution. Distinction has been made between water levels below foreshore (shown in yellow) and water levels above foreshore (shown in green). The total failure probability is found by integrating the product of the fragility curve and the water level distribution over the water levels. The contribution of the water levels below foreshore to the total failure probability can be found by integrating this product over only the water levels that are below foreshore level (yellow part). For water levels above foreshore the same approach can be used, integrating over the water levels above foreshore (green part). Note that this approach has already been used in Chapter 6.

Equation (8.1) can be reduced to:

$$P_f = P_{f,lower\ water} + P_{f,higher\ water} \tag{8.2}$$

In which:

$P_{f,lower\ water}$	The contribution of the water levels below foreshore to the total failure probability, found by integration of the yellow part in Figure 8.2 [-]
$P_{f,higher\ water}$	The contribution of water levels above foreshore to the total failure probability, found by integration of the green part in Figure 8.2 [-]

Based on (8.1) and (8.2), the following equations can be defined:

$$P_{f,lower\ water} = P_{f|h\leq h^*} * P_{h\leq h^*} \tag{8.3}$$

$$P_{f,higher\ water} = P_{f|h>h^*} * P_{h>h^*} \tag{8.4}$$

In this Thesis, for all test cases the contribution of lower water levels to the total failure probability (after taking into account residual width) is negligible ($P_{f,lower\ water} = 0$, see Chapter 6). Equation (8.1) then reduces to:

$$P_f = P_{f|h>h^*} * P_{h>h^*} \tag{8.5}$$

The requirement for a safe dike is a failure probability which is lower than the target probability. The requirement can be written as follows, using equation (8.5):

$$P_{f|h>h^*} \leq \frac{P_{f,target}}{P_{h>h^*}} \tag{8.6}$$

Equation (8.6) is similar to the approach used for the combination of macro-stability and wave overtopping (see Appendix I).

In Equation (8.6) , the probability of flooding of the foreshore, which has a lot of influence on the failure probability, is included in the requirement of the dike. A semi-probabilistic rule for water levels above foreshore should lead to the conditional failure probability given the water level is above foreshore ($P_{f|h>h^*}$). A calibration has to be performed to be able to transform a FoS, resulting from the semi-probabilistic calculation, into $P_{f|h>h^*}$, which can then be compared to the requirement of (8.6).

Although this semi-probabilistic approach looks similar to the current semi-probabilistic method, the current (β-γ_n)-relation from the original calibration study cannot be used. After all, this calibration links the FoS to the total failure probability of the dike (P_f). In this new method, a relation is needed that links the FoS to the conditional failure probability given the water levels is above foreshore ($P_{f|h>h^*}$).

In reality, it may be possible that $P_{f,lower\ water} \neq 0$. The requirement of equation (8.6) then cannot be used, because the target probability cannot be completely used for the contribution of the water levels above foreshore, but part of the target probability will be used for the contribution of the water levels below foreshore. This is where the two separate assessment rules come into play.

The total safety requirement looks as follows:

$$P_{f,lower\ water} + P_{f,higher\ water} \leq P_{f,target} \tag{8.7}$$

The first assessment rule will be used to determine how much of the target probability is used for the contribution of the water levels below foreshore ($P_{f,lower\ water}$). Equation (8.3) can also be written as follows:

$$P_{f|h\leq h^*} = \frac{P_{f,lower\ water}}{P_{h\leq h^*}} = \frac{P_{f,lower\ water}}{1 - P_{h>h^*}} \tag{8.8}$$

A semi-probabilistic calculation using a (design) water level below foreshore should be performed, resulting in a FoS, which can then, using a calibrated relation, be transformed into $P_{f|h<h^*}$, which will lead to $P_{f,lower\ water}$ (using equation (8.8)). One needs to perform a calibration study to determine this relation.

The requirement for the first assessment rule is as follows:

$$P_{f,lower\ water} \leq P_{f,target} \tag{8.9}$$

After all, if only the contribution of the water levels below the foreshore already exceeds the target probability of the dike, the dike will never be assessed as safe, no matter what the contribution of the higher water levels is.

The requirement for the second assessment rule, for the water levels above foreshore level, now changes from equation (8.6) to:

$$P_{f|h>h^*} \leq \frac{P_{f,target} - P_{f,lower\ water}}{P_{h>h^*}} \tag{8.10}$$

Concluding the above, the method of two separate assessment rules starts with a semi-probabilistic calculation for the water levels below foreshore. Then, a semi-probabilistic calculation for water levels above foreshore is performed. For both calculations, a calibration has to be performed to be able to transform the FoS (resulting from the semi-probabilistic calculation) into $P_{f|h\leq h^*}$ and $P_{f|h>h^*}$ respectively. An example of such a calibration, performed for the test cases of this study, is given in Appendix J.

Furthermore, a clear approach for both semi-probabilistic calculations has to be established. A choice has to be made which water levels are used in the semi-probabilistic calculations. For example, for the water levels below foreshore one could use a water level equal to or just below foreshore level and for water levels above foreshore level one could use a water level x meter above foreshore. These choices should also be used in the calibration. However, this falls outside the scope of this research and is not further covered here. In further research, one can develop the concept described above to a proper assessment method.

The main advantage of the method of two separate assessment rules, compared to just calibrating a new (β-γ_n)-relation, is the fact that the probability of flooding of the foreshore, which has a significant influence on the failure probability, is included in the assessment. This way, the assessment better resembles reality. However, note that the results are still approximated failure probabilities based on calibration studies, which is still a limitation of this method.

8.4. Fully Probabilistic Method

Another option to improve the current assessment is switching to fully probabilistic calculations using fragility curves, as used in this Thesis. By using fragility curves, one can distinguish easily between water levels below foreshore level, for which one takes into account residual width, and water levels above foreshore, for which the WBI failure definition can be used.

The biggest advantage of using fully probabilistic calculations in the dike assessment, is the fact that they directly result in a failure probability. This as opposed to the semi-probabilistic methods, which use a calibrated relation to approximate the failure probability. Therefore, the fully probabilistic calculations better resemble the reality than the semi-probabilistic methods. Furthermore, the approach using fragility curves gives a lot of insight into the residual width a dike with foreshore has for different water levels, and which processes can lead to failure.

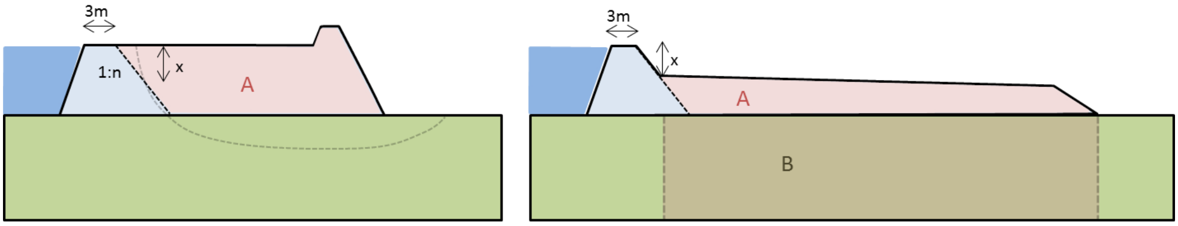
The main reason that a semi-probabilistic method is used in the current macro-stability assessment, is probably that in general perception, fully probabilistic calculations are difficult to perform. However, in fact, the differences between fully probabilistic and semi-probabilistic calculations are not that big. If one would produce clear guidelines for performing fully probabilistic calculations for macro-stability, like the guidelines the WBI proposes for the semi-probabilistic method, most people should be capable of performing such calculations. The remaining differences between the two calculation types is the fact that the fully probabilistic calculations are more time-consuming. However, in return, the result better resembles the real failure probability (see Section 4.6) than the result of a semi-probabilistic method.

To prevent unnecessary long calculation times, one could use the current semi-probabilistic method as a funnel. After all, this method is quick and easy to use and leads to conservative results for dikes with high foreshores. If a dike is assessed as safe based on this method, one does not have to perform the fully probabilistic calculations.

It may be possible to simplify the calculations for the water levels below foreshore in the fully probabilistic method. In the approach used in this Thesis, for every individual soil slide in the progressive failure analysis the probability was calculated. As mentioned before, the combined probability of the individual slides is generally equal to the individual probability of the last slide. Therefore, it should be possible to determine the failure probability for progressive failure using only the last slide leading to failure. However, for this analysis, a remaining profile after the last but one slide is needed. It may be possible to create a method for the schematization of this remaining profile based on assumptions. A possible schematization is shown in Figure 8.3

The schematization is based on (ENW, 2009), which describes the minimum remaining width a dike should have to be safe after sliding along a circular slip surface, taking into account that the top of this circular surface is unstable and a secondary straight sliding plane is possible (explained in Appendix B). The remaining profile after the last but one soil slide as schematized in Figure 8.3 is at the edge of failure as defined by (ENW, 2009): the remaining width is 3m. The slope 1:n is based on the slope of the secondary straight sliding plane (see Appendix B for values of n for different dike materials). The starting point of the remaining profile is at the line 1:n at a level of x below foreshore level (see Figure 8.3). (ENW, 2009) proposes half the foreshore height as conservative assumption for x. However, this is conservative for the secondary straight sliding plane, as a higher value of x will lead to more foreshore loss due to the secondary sliding plane. For the analysis of a new circular sliding plane, a lower value for x is conservative. In reality, values of 1m or 2m (of crest reduction x) are mostly observed. As a conservative choice one could choose a value of 1m.

In further research, it has to be validated whether these assumptions lead to a good approximation of the remaining profile.



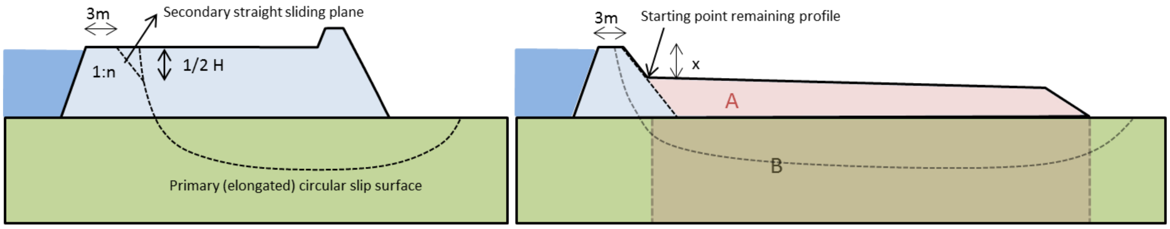
1) Initial dike profile: The remaining width is 3m, so the profile is at the edge of failure. A possible (elongated) circular slip surface is shown. The soil that will be redistributed for the remaining profiel (volume A), is bounded by the line starting from the remaining width of 3m, at a slope of 1:n. This slope is equal to the slope of a secondary straight sliding plane (ENW, 2009), as explained in Appendix B.

2) Remaining profile: The starting point of the remaining profile is at the line 1:n at the level of the foreshore minus x. Volume A will be redistributed starting from this point. The zone B shows the "disturbed clay zone", in which the shear strength is reduced due to previous slides. This zone starts at the same horizontal location as the starting point of the remaining profile. This is a conservative choice: in reality this zone will start at the intersection of the previous slip surface and the surface level. But, as this slip surface is unknown, the conservative choice will be used.

Figure 8.3: Schematization of the profile for the alternative approach progressive failure

Once a right schematization method for the remaining profile after the last but one slide has been established, the conditional failure probabilities for water levels below foreshore can be determined using the following approach:

1) Determine the probability of the critical long sliding plane in the initial profile, leaving a remaining width of less than 3m (scenario 2, see Figure 8.4). This remaining width is defined assuming the occurrence of a straight secondary slip surface at a slope of 1:n, crossing the primary (circular) long sliding plane at a level of half the foreshore height, as defined in (ENW, 2009) and Appendix B. The primary slip surface shown in the figure is just at the edge of failure, leaving a remaining width of 3m. The probability of the critical primary slip surface leading to less than 3m of remaining width, is the failure probability.
2) Determine the probability of the critical slip surface in the remaining profile after the one but last slide as defined above (scenario 3, see Figure 8.4). The critical slip surface must have an entrance point at the left of the starting point of the remaining profile, as this will lead to failure as defined by (ENW, 2009).
3) The highest of the probabilities above is assumed to be the conditional failure probability for all water levels below foreshore level.



Scenario 2 (one long sliding plane) Scenario 3 (progressive failure)

Figure 8.4: Schematization of the calculation of scenario 2 and 3 in the alternative approach

Note: For scenario 2, half the foreshore height is a conservative estimation for x (described above), as a higher value of x leaves less remaining width (the secondary straight sliding plane takes away a larger part of the foreshore). For scenario 3, lower values for x are conservative.

The simplification as described above can be a quick and easy alternative for the original analysis as described in Chapters 4 and 6. However, as mentioned in Section 4.6, the progressive failure analysis used in this Thesis, on which the simplification is based, has its limitations. More realistic modelling of the progressive failure analysis may prove the simplification as specified above wrong.

8.5. Comparison

This chapter proposed three possible improvements for the current semi-probabilistic method:

- Calibrating a new ($\beta - \gamma_n$) – relation, specifically for dikes with foreshores
- Creating two separate assessment rules: one for the water levels below foreshore and one for water levels above foreshore
- Switching to a fully probabilistic method

Table 8.1 shows the advantages and disadvantages of the three possible improvements.

Method	Advantage	Disadvantage
New ($\beta - \gamma_n$) – relation	• Quick and easy to use	• A lot of cases needed for a new calibration • Results in approximated failure probability
Two separate assessment rules	• Resembles the reality better than just a new ($\beta - \gamma_n$) – relation, because it includes the probability of flooding of the foreshore in the assessment • Quick and easy to use	• Results in approximated failure probability • New calibrations needed • Further research needed to establish approach for the semi-probabilistic calculations (choices for design water levels)
Fully probabilistic method	• No calibration needed, directly results in failure probability • Gives insight in different processes for water levels above foreshore and below foreshore	• Time-consuming • More difficult to perform • Progressive failure analysis has its limitations

Table 8.1: Comparison (dis)-advantages improved methods

An important limitation of the two semi-probabilistic methods is that the failure probability is approximated using a calibrated relation. Of these two, the method using two separate assessment rules resembles the reality in a better way, because the probability of flooding of the foreshore, which has a lot of influence on the failure probability, is included in the assessment. Therefore, this method is preferred above the other one, calibrating a new ($\beta - \gamma_n$) – relation.

The fully probabilistic method directly results in the failure probability; it is not based on a calibration, and therefore, it better resembles the reality than the semi-probabilistic calculations. Therefore, the best improvement for the safety assessment is switching to a fully probabilistic method, as used in this Thesis. However, there is still uncertainty in the progressive failure analysis that is used in this method, which has its limitations (see Section 4.6). Based on further research, more realistic modelling of progressive failure should be possible.

Besides the fully probabilistic method, one could further develop the semi-probabilistic method using two separate assessment rules as an alternative, in case the fully probabilistic method turns out to be too difficult to perform for the dike managers.

9. Conclusions

This chapter presents the most important findings of this Thesis. Furthermore, it will present a final answer to the main research question:

Is it possible to improve the safety assessment method for macro-stability for dikes with high and wide foreshores?

Note that the conclusions as presented below are based only on the test cases used in this Thesis, as specified in Section 4.3, which all have a relatively high foreshore and a simplified subsoil composition. Besides, the conclusions are based on the calculations performed in this research, and thus on their limitations and assumptions, as specified in Section 4.6. It is uncertain to what extent the conclusions of this Thesis apply in general sense.

Conclusions concerning the semi-probabilistic method for dikes without foreshores

For most of the test cases without foreshore considered in this Thesis, the semi-probabilistic method does not lead to a conservative result, but to an underestimation of the failure probability. As already explained, the test cases are biased, which makes it impossible to draw hard conclusions about the conservatism in the semi-probabilistic method for dikes without foreshore, based on this research.

Important to realize however, is the fact the semi-probabilistic method does certainly not always lead to a conservative result for dikes without foreshores. One should be aware of the fact that the semi-probabilistic method may lead to an underestimation of the failure probability and therefore one should use the semi-probabilistic rule with care.

Conclusions concerning the influence of the foreshore on the failure probability

The research showed that for the test cases considered in this Thesis, the presence of a foreshore leads to a significant reduction in failure probability, compared to a dike without foreshore. A maximum reduction of a factor 25,336 was found. This corresponds to a difference in reliability index of $\Delta\beta = 1.88$. Differentiating between clay dikes and sand dikes showed that the influence of the presence of the foreshore is larger for clay dikes. Furthermore, the research showed that the higher the level of the foreshore and the narrower the water level distribution, the larger the influence of the foreshore. In other words: the smaller the probability that the foreshore is flooded, the more influence the foreshore has on the failure probability.

The influence of the foreshore, described above, was only found using fully probabilistic calculations and taking into account the residual width of the dike. Using the WBI failure definition or the semi-probabilistic method led to little or no influence of the presence of a foreshore. After all, these calculation methods led to overestimations of the failure probability of dikes with high foreshores, as the research showed.

Furthermore, the research showed that for most test cases considered in this Thesis, a foreshore of 25m wide or more already leads to negligible conditional failure probabilities for water levels below foreshore. Therefore, a foreshore of 25m wide would already lead to the above mentioned reductions in failure probability. This shows that a relatively small foreshore already has a significant influence on the failure probability.

Conclusions concerning the semi-probabilistic method for dikes with foreshores

The semi-probabilistic method can lead to significant overestimations of the failure probability for dikes with high foreshores, compared to fully probabilistic calculations. A maximum overestimation of a factor 19,192 ($\Delta\beta = 1.82$) was found. In other words: the semi-probabilistic method resulted in a reliability index which was 1.82 lower than for the fully probabilistic calculations. For the semi-probabilistic method to result in the same amount of safety (same reliability index) as the fully probabilistic calculations, one has to apply a stability berm in the hinterland. Research showed that for the case in which the maximum overestimation was found, a stability berm of 2.5m high and 8.5m wide was needed. This shows that using the semi-probabilistic method, which is mostly used for the assessment, for dikes with high and wide foreshores can lead to unnecessary expenses for dike reinforcements.

To answer the research question, the above proves there is room for improvement in the safety assessment for macro-stability for dikes with high and wide foreshore.

10. Recommendations

This chapter discusses recommendations that can be made based on this Thesis, both for improving the safety assessment method and for further research. Based on the research performed in this Thesis, the following recommendations can be made:

1. For the safety assessment of macro-stability for dikes with high foreshores, it is recommended to switch to a fully probabilistic method. This method is based on fragility curves and takes into account the residual width of the foreshore. However, this method, as used in this Thesis, can be further improved:
    a. Using more accurate modelling of progressive failure (see recommendation (3)).
    b. If possible based on a), it is recommended to create a method to simplify the progressive failure analysis, by only considering the last soil slide leading to failure. Further research is needed to create a proper schematization method for the remaining profile after the last but one soil slide.
    c. It is recommended to include the time-dependence of the phreatic line in the assessment. This will have a positive effect on the safety of the dike, as it takes time for the foreshore body to get completely saturated.

2. Besides the fully probabilistic method, establish an alternative semi-probabilistic assessment, based on separate assessment rules for water levels below foreshore and above foreshore. The concept of such an assessment is described in Section 8.3. Further research is needed to establish a clear approach for the semi-probabilistic calculations. Besides, for both assessment rules, calibrations have to be performed.

3. The progressive failure analysis as performed in this Thesis is based on a lot of assumptions (see Section 4.6). It is highly recommended to perform the following further research to make more accurate modelling of progressive failure possible:
    a. Establish a more accurate way of analysing all possible pathways leading to failure, as the method used in this Thesis only considers two types of slip surface shapes and thus neglects a lot of possible slip surfaces.
    b. Establish a more accurate way of schematizing the remaining profile after a slide, as the remaining profile in this Thesis was based on a best guess, which is highly uncertain. Note that the remaining profile depends to a large extent on the slip surface, which may induce the need to combine a) and b).
    c. Investigate soil behaviour after a soil slide, to determine whether the strength of the soil is reduced and to what extent.
    d. Include dependence between successive slides in the calculations. In this Thesis, the successive slides are assumed to be completely correlated, which is a conservative choice. By including the dependence between the successive slides, the total probability of the combination of the slides can be decreased.

4. For the test cases without foreshores, the semi-probabilistic method in this Thesis resulted for most cases in an underestimation of the failure probability, instead of a conservative result. This could be an indication that the semi-probabilistic method is not as conservative as was expected (for dikes without foreshores). However, as mentioned before, the Test Cases are biased; they are unrealistically simplified. Therefore, it is recommended to collect the data from studies which calculate the failure probability using both the current semi-probabilistic method and fully probabilistic calculations, for real dikes (not biased). Based on these data, one can verify whether or not the current semi-probabilistic method is as conservative as expected and, if not, change the calibrated (β-γ_n)-relation.

5. It is recommended to clearly state in the WBI documentation that the current semi-probabilistic method results in an approximated failure probability, which is not always conservative, and that the real failure probability may be higher. Performing a fully probabilistic calculation can therefore lead to a worse result.

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Glossary

(β-γ_n)-relation	Calibrated relation which is used to approximate the reliability index (or failure probability) based on the result of the semi-probabilistic calculation (γ_n, related to FoS).
Aquifer	Permeable sand layer beneath the aquitard, through which groundwater flows.
Aquifer head	Head in the permeable sand layer. Measure for the pore pressures inside this layer.
Aquitard	(Almost) impermeable top layer(s) in the subsoil beneath a dike. The aquitard consists out of clay or peat.
Characteristic value	Value corresponding to a certain quantile of the probability distribution of a stochastic variable.
Conditional failure probability	The failure probability of the dike given occurrence of a certain event. In the fragility curves, the conditional failure probabilities are the failure probabilities given occurrence of certain water levels.
Conservative	“More negative than reality”. A conservative choice leads to overestimation of the failure probability and is used as a safe choice when the actual value is unknown.
Correlation	Measure of statistical dependence between two stochastic variables.
Critical slip surface	The macro-stability analysis considers different slip surfaces along which sliding of the soil is possible. The critical slip surface is the one leading to the least stability of the dike.
Cumulative distribution function (CDF)	Function which describes the probability that a stochastic variable is smaller than a certain value.
Design point	The combination of parameter input values that will most likely lead to failure.
Design water level	Water level used for the semi-probabilistic calculation. The design water levels is the water level with an exceedance frequency equal to the safety standard of the dike.
Deterministic	A deterministic parameter is modelled as only one value, without specifying uncertainty.
Dike section	Part of a dike trajectory with homogenous subsoil.

Dike trajectory	A chain of flood defences (often dikes) with approximately homogenous consequences in case of failure anywhere along the trajectory. The trajectories are regulated by law.
Drained behaviour	If soil shows drained behaviour, water flows out of the soil during loading.
Factor of Safety (FoS)	Factor describing the ratio between strength and load in the macro-stability mechanism. Is a measure of stability and the direct result of the semi-probabilistic calculation method.
Failure	Defined as loss of function of the dike, in terms of flooding.
Failure definition	Describes what situation is regarded as failure. For the macro-stability mechanism is determines which slip surfaces are considered as failure.
Failure probability	The annual probability of failure of a dike due to a certain failure mechanism, in this Thesis macro-stability. The failure probability is the probability of meeting a certain failure definition.
Foreshore	Land in front of a dike which is not flooded during daily situations. Only for high water levels, the foreshore is flooded.
FORM	First Order Reliability Method. Probabilistic calculation method used to calculate the conditional failure probabilities. The method linearizes the limit state function in the so called design point.
Fragility curves	Curves describing the relation between the water level and the corresponding conditional failure probability. By calculating the conditional failure probabilities for different water levels and constructing a fragility curve, the total failure probability can be calculated.
Fully probabilistic calculation	Calculation of the failure probability by using the complete probability distributions for all parameters. In this Thesis, the method of fragility curves is used. More calculation and iteration steps are needed for this calculation. The calculations result directly in a failure probability.
Influence coefficient (α)	Measure for the relative influence of a certain stochastic parameter on the total uncertainty, compared to other parameters.
Influence factor (α^2)	Measure for the relative influence of a certain stochastic parameter on the total uncertainty. Is equal to the influence coefficient squared.
Intrusion Length	Height of the aquitard (measured from the bottom) in which the variations in aquifer head can be felt.
Leakage Length	Measure for the distance over which the aquifer head develops.

Limit Equilibrium Models (LEM)	Used for the calculation of macro-stability. Compare the load against the maximum mobilizable strength.
Limit State Function (Z)	Used in probabilistic calculations to describe failure. Failure is defined as Z<0.
Macro-stability	Failure mechanism considering sliding of a volume of soil of the inner slope of the dike along a straight or circular sliding plane. Macro-stability is often caused by an increase in pore pressures in the soil.
Model factor	Partial safety factor describing the model uncertainty of the LEM models used in the macro-stability calculations.
Model uncertainty factor	Stochastic variable describing the model uncertainty of the LEM models used in the macro-stability calculations.
Phreatic line	Describes the development of the water level inside the dike. Shows which part of the dike is saturated.
Phreatic line	The water level inside the dike. The dike is saturated below the phreatic line.
Probability density function (PDF)	Function which describes the probability distribution of a stochastic variable.
Progressive failure	Successive shallow soil slides leading to loss of function of the dike.
Reliability index	Describes the safety (reliability) of the dike. Related to the failure probability by the cumulative standard normal distribution, which describes a standard probability distribution.
Remaining width	Width of the remaining dike at the height of the water level after sliding of the soil, after taking into account a secondary straight sliding plane. Is used to define which situation can be regarded as failure.
Residual width	The part of the dike and foreshore that is still intact after sliding of the soil and is still able to retain the water.
Safety	The resistance against sliding in land inwards direction, only considering slip surfaces meeting the failure definition. The higher the safety, the lower the failure probability.
Safety standards	The maximum allowed failure probabilities for the different dike trajectories in the Netherlands. The safety standards are regulated by law.

Semi-probabilistic calculation	Calculation of the failure probability using design values of the parameters. Only one calculation step needed, which results in a factor of safety (FoS). A calibrated relation is used to approximate the failure probability based on FoS.
Shear strength	The maximum shear stress the soil can withstand.
Stability	The resistance against sliding in land inwards direction. The higher the stability, the lower the probability of any soil slide occurring.
Stochastic variable	Uncertain variable described by a probability distribution function
Target probability	The maximum allowed failure probability of a dike section for a certain failure mechanism. Is found by distributing the safety standard over the failure mechanisms and the dike sections in the trajectory.
Uplift	The head in the aquifer exceeds the effective weight of the aquitard, causing the aquitard to float on top of the blanket. The shear strength in the uplift zone is then reduced to zero.
Undrained behaviour	If soil shows undrained behaviour, water cannot flow out of the soil during loading, and excess pore pressures are generated.
Water level distribution	Probability distribution of the annual maximum water levels.
WBI	Documentation describing the process of the safety assessment of the dikes and providing guidelines for the assessments.
WBI failure definition	All slip surfaces that enter halfway the inner slope of the dike or further to the water side are considered relevant (regarded as failure)

Appendix A. Soil parameters

Parameter	model	γ_dry/γ_wet	c	φ_mean	φ_dev	φ_cov	S_mean	S_dev	S_cov	POP_mean	POP_dev	POP_cov
Case 1												
Sand, Dike	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Clay, B	CuCalculated	15/15	-	-	-	-	0.25	0.03	0.12	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.25	0.03	0.12	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 2												
Clay, Dike (above PL)	Cphi	17/17	0	38.34	5.065	0.132	-	-	-	-	-	-
Clay, Dike	CuCalculated	17/17	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Clay, B	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 3												
Sand, Dike	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Clay, B	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 4												
Sand, Dike	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Clay, B	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 5												
Clay, Dike (above PL)	Cphi	17/17	0	38.34	5.065	0.132	-	-	-	-	-	-
Clay, Dike	CuCalculated	17/17	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Clay, B	CuCalculated	15/15	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 6												
Sand, Dike	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Clay, B	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.32	0.03	0.094	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-
Case 7												
Clay, Dike (above PL)	Cphi	17/17	0	38.34	5.065	0.132	-	-	-	-	-	-
Clay, Dike	CuCalculated	17/17	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Clay, B	CuCalculated	15/15	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Clay, N	CuCalculated	15/15	-	-	-	-	0.35	0.03	0.086	25	7.5	0.3
Sand, Pleistocene	Cphi	18/20	0	34	1.7	0.05	-	-	-	-	-	-

Table A.1: Subsoil parameters of all test cases

Note: Clay,B and Clay,N mean “Clay below dike” and “Clay next to dike” respectively. These are the two parts of the same layer, but due to different stress history below and next to the dike this layer is split into two parts, as explained in Section 4.5.

Appendix B. Calculation Choices

Section 4.5 briefly discussed the calculation choices that were made for the calculations performed in this Thesis. In this Appendix, these choices are covered in more detail.

First, the general calculation choices, like the parameters that are needed and the choices made for the schematization, are described. Then specific choices for the semi-probabilistic calculations, the fully probabilistic calculations and the progressive failure analysis are covered.

General

This section covers the general calculation choices that were made in this Thesis, like the parameters used in the calculations and the general schematization choices.

Parameters

Table B.1 shows the parameters that are used for the calculations in the WBI macro-stability kernel (in the D-GeoStability software). For each parameter, the table shows whether it is used for drained or undrained analysis and the distribution type. For some parameters it also shows the variation that is used; for these parameters theory gives standard values for the variation, which are always used. The mean values of the soil parameters and the variations that are not shown in the table are for each test case and for each soil layer separately depicted in Appendix A.

Symbol	Unit	Description	Drained	Undrained	Distribution	Used Variation
γ_{dry}	[kN/m^3]	Unit weight of soil above phreatic level	*	*	Deterministic	-
γ_{wet}	[kN/m^3]	Unit weigth of soil below phreatic level	*	*	Deterministic	-
c’	[kN/m^2]	Effective cohesion	*		Deterministic	-
ϕ	[°]	Effictive friction angle	*		Lognormal	Appendix
S	[-]	Undrained shear stress ratio		*	Lognormal	Appendix
m	[-]	Strength increase exponent		*	Lognormal	CoV = 0.03 (Rijkswaterstaat, 2016b)
σ'_{vy}	[kN/m^2]	Vertical yield stress		*	Lognormal	Appendix
λ_{in}, λ_{out}	[m]	Leakage length	*	*	Lognormal	Cov = 0.20 (Rozing, 2015)
IL	[m]	Intrusion length	*	*	Lognormal	CoV = 0.30 (Rozing, 2015)
PL1	[m+NAP]	Phreatic line schematization	*	*	Deterministic	-

WL	[m+NAP]	Water level	*	*	Gumbel	Appendix

Table B.1: Parameters used for the calulcations

Yield stress points

There are different methods for the calculation of the shear strength of undrained layers. In this study, the SHANSEP model using yield stress points is used (see Section 2.4.6). Yield stresses are measures of the stress history of the soil. They can be determined based on an in-situ cone penetration test or on the vertical effective stress in the yield stress point and the POP (pre-overburden pressure) value of a soil layer(determined in laboratory tests). As this study is based on theoretical test cases, and thus no cone penetration tests are available, the latter will be used in this study.

In each soil layer a yield stress point will be placed at three different locations in x-direction: one below the foreshore, one below the dike crest and one below the hinterland. The yield stresses in these points are calculated by adding the POP value of the soil layer in which the point is located to the effective stress value during daily conditions in that point.

The uncertainties in the vertical yield stresses are almost completely caused by the uncertainty in POP values (Schweckendiek et al., 2017). Therefore, the variation coefficient of the yield stress values is equal to variation coefficient of the corresponding POP value. Furthermore, in the lognormal distribution of the yield stress, a shift has been included equal to the value of the effective stress. This way, the yield stress uncertainty is modelled as exactly equal to the POP uncertainty.

The shear strength of the soil can be calculated based on the yield stresses using formula (2.22) (Section 2.4.6).

Soil layers

In general use in macro-stability calculations, soil layers are cut into two parts: a part below the dike and a part next to the dike, because both parts have a different stress history and therefore different strength or state parameters may be found. In this Thesis, all undrained layers (the aquitard) are split up in this way. The part next to the dike gets assigned one yield stress point and the part below the dike gets assigned the other two yield stress points. This way, the schematization better represents the local stress state. The different parts of the undrained aquitard layer and the location of the yield stress points are illustrated in Figure B.1.

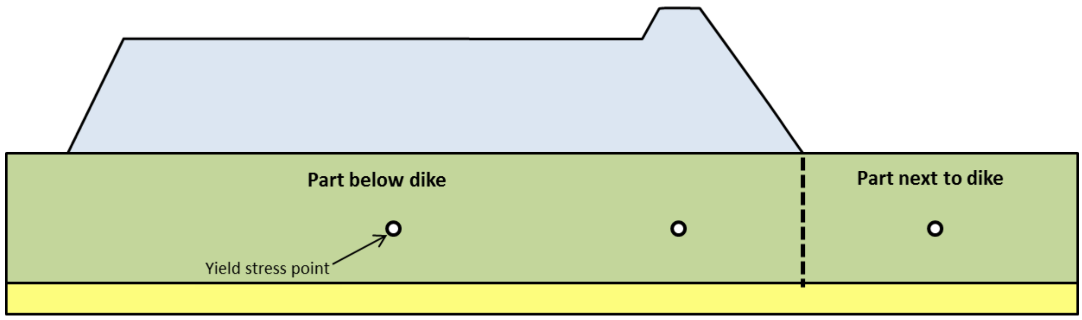


Figure B.1: Schematization different parts undrained aquitard

Furthermore, for cases with a dike body out of clay, the part of the clay dike that is above the phreatic line is modelled as separate layer. The reason for this is the fact that the strength of clay layers is usually calculated using undrained calculations, using the SHANSEP model. Undrained calculations are based on the fact that excess pore pressures are generated when the soil is loaded. However, the part of the dike above the phreatic line is dry, and therefore no excess pore pressures can be generated, because there are no pore pressures at all. Therefore, the strength of this part is calculated using the drained CSSM model (Section 2.4), based on the critical state friction angle φ_{cs} and the assumption of zero cohesion. Appendix H shows how φ_{cs} is determined.

Waternet Creator

In the WBI calculation kernel, the Waternet Creator is used, which creates the complete waternet inside a dike, based on the input of an outside water level, inwards and outwards leakage lengths, and an intrusion length. The output of the Waternet Creator consists of the phreatic line inside the dike and piezometric level (PL) lines in the soil layers beneath the dike.

The leakage length parameters are used to schematize the head in the aquifer. They are measures of length, which express the distance over which the head in the aquifer develops. Two leakage lengths have to be defined: an inward leakage length (in the direction of the land) and an outward leakage length (in the direction of the water). By modelling the leakage lengths as stochastic variables in the fully probabilistic calculations, the uncertainties in pore pressures are taken into account. The leakage length parameters can be calculated as follows (TAW, 2004):

$$\lambda = \frac{k * D * d}{k'} \tag{B.1}$$

In which:

λ	Leakage length [m]
k	Hydraulic conductivity aquifer [m/s]
D	Thickness aquifer [m]
d	Thickness aquitard [m]
k'	Hydraulic conductivity aquitard [m/s]

In the test cases used in this study, it is assumed that the hydraulic conductivity of the aquitard is $1.0*10^{-7}$m/s and the hydraulic conductivity of the aquifer is $1.0*10^{-4}$m/s for every case.

The intrusion length is the height of the aquitard over which the pore pressure variations in the aquifer can be felt. (Rijkswaterstaat, 2016b) gives standard values for the intrusion length, based on the soil composition of the aquitard. These values are used in the calculation.

The schematization choices that are used in the Waternet Creator are described in more detail in Appendix C.

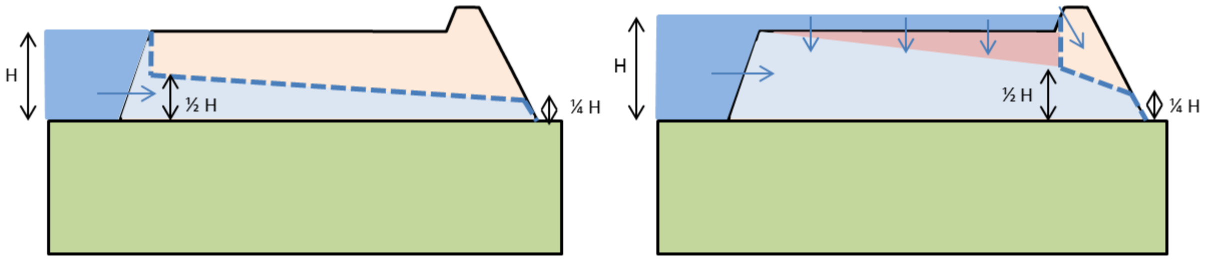
Phreatic line schematization

The schematization of the Phreatic Line is done automatically by the Waternet Creator, based on different standard dike cases as proposed by (TAW, 2004). These standard cases are based on dike and subsoil material, and the corresponding schematizations are described in Appendix C.

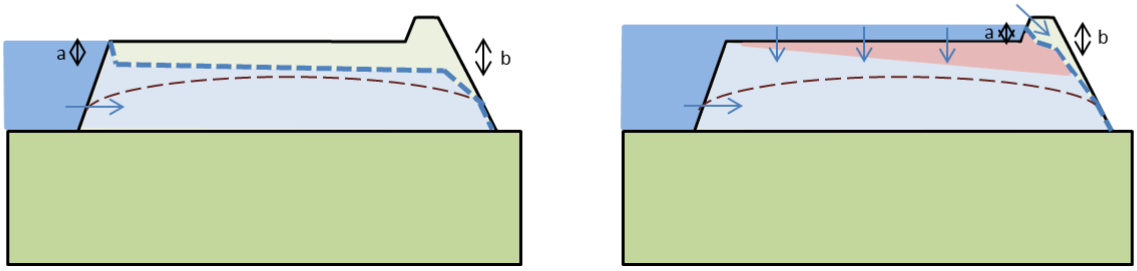
For clay dikes, the standard schematization of the phreatic line is limited by the phreatic line during daily conditions, based on precipitation and daily water level. This daily phreatic line will be approximated by a simple model in Plaxis software, based on a precipitation of 0.7m/year, which is the average amount of precipitation in the Netherlands (TAW, 2004).

The standard schematizations of the phreatic line, used by the Waternet Creator, are conservative estimates. These standard schematizations are created in such a way, that they lead to a little conservative, but safe result for dikes in general. However, this will probably not affect the results of this study a lot, because this study will compare the results from different types of calculations (see Section 4.2), and in all calculations the same schematization standards will be used. The accuracy of the phreatic line will be more important for actual design.

For the specific case of dikes with high and wide foreshores, the standard schematizations of the phreatic line are probably even more conservative than for dikes in general. After all, if the water level rises from a level below foreshore level (daily situation) to a water level above foreshore level, it takes time before the complete foreshore is saturated. How much time this takes, depends on the permeability of the foreshore. It may very well be possible that the foreshore does not get completely saturated before the end of the high water situation. The standard schematization ignores this fact, as it assumes a completely saturated foreshore in case the water level is above foreshore level. This is illustrated in Figure B.2, both for a sand dike and for a clay dike. In reality, the foreshore has a positive effect on the stability of the dike, because it takes time for the foreshore to get saturated, and thus it takes time for the pore pressures to rise in the foreshore. However, this is not accounted for in the standard schematization of the phreatic line, as used in this study. In further research, the influence of the simplification can be studied, by taking time-dependence of the development of the phreatic line into account.



Phreatic line in a sand dike (blue dashed line), schematized according to WBI "Sand dike on Clay": the phreatic level is reduced to half of the original water level at the entrance point and to a quarter at the end of the dike. The blue arrows show where the water infiltrates into the foreshore and the dike body. Note that the schematization is conservative: it assumes that for a water level above the foreshore the complete foreshore is saturated. However, the water that is on top of the foreshore takes time to infiltrate into the foreshore. Probably, part of the foreshore is not saturated (the red part) during the high water situation. The amount of foreshore that is saturated depends on the high water duration and the permeability of the foreshore.



Phreatic line in a clay dike, schematized according to WBI "Clay dike on Clay": the offsets at the entrance point and below the inner crest are a=1m and b=1.5m respectively. However, this schematization is limited by the phreatic line during daily situations. This daily phreatic line in a clay dike has a bulge (illustrated by the red dashed line), due to precipitation which can not flow out. The phreatic line during high water level can never be lower than this daily phreatic line. Again, as was the case for the sand dike, the phreatic line schematization is probably conservative (illustrated by the red part of the foreshore).

Figure B.2: Illustration schematization phreatic line and phreatic line development

Semi-probabilistic calculation choices

This section covers the specific choices that are made for the semi-probabilistic calculations.

Calculation Approach

The semi-probabilistic calculation is performed as proposed by the WBI (Rijkswaterstaat, 2016a). Using a design water level with an exceedance frequency equal to the maximum allowed failure probability of the dike trajectory (safety standard), and design (characteristic) values for all other parameters, a critical slip surface with a certain Factor of Safety is obtained, which is transformed into an approximated failure probability using formula (2.20):

$$P_f = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.41}{0.15}\right)$$

Parameters

As mentioned before, the semi-probabilistic calculation is performed using design values for the different parameters. Design values are obtained by multiplying some characteristic value by a safety factor (material factor). However, as mentioned in Section 2.4, these safety factors are 1.0 for all parameters. This means that the design value is just a characteristic value, i.e. a value for a certain quantile of the distribution of the parameter. Table B.2 gives the characteristic values to be used for the calculation, as proposed by WBI (Rijkswaterstaat, 2016b).

Symbol	Unit	Distribution	Characteristic value (quantile)
γ_{dry}	[kN/m^3]	Deterministic	-
γ_{wet}	[kN/m^3]	Deterministic	-
c'	[kN/m^2]	Deterministic	-
ϕ	[°]	Lognormal	5%
S	[-]	Lognormal	5%
m	[-]	Lognormal	5%
σ'_{vy}	[kN/m^2]	Lognormal	5%
λ_{in}, λ_{out}	[m]	Lognormal	50%
IL	[m]	Lognormal	50%
PL1	[m+NAP]	Deterministic	-
h	[m+NAP]	Gumbel	Exceedance frequency equal to safety standard dike trajectory

Table B.2: Quantiles of probabilistic calculations to be used in the semi-probabilistic calculations for the different parameters

Model factor

In the calibration formula, a model factor (γ_d) is used to transform the calculated Factor of Safety into an approximated failure probability. This model factor accounts for model uncertainties. As said before, the Uplift Van slip surface model will be used for the calculation. The corresponding model factor is 1.06, as mentioned before in Section 2.4.

Fully Probabilistic calculation choices

This section covers the specific choices that were made to perform the fully probabilistic calculations.

Calculation approach

The fully probabilistic calculation will be performed using fragility curves (see Section 2.3.3). This means that for a range of water levels the conditional reliabilities (β) will be computed. By linear interpolation between the calculated points, a (β-h)-curve is obtained, which shows for all water levels the corresponding conditional reliability. This (β-h)-curve is then transformed into a fragility curve, which shows for all water levels the corresponding conditional failure probability. By integrating the product of this fragility curve and the water level distribution over the water levels, the total annual failure probability is obtained.

To construct a (β-h)-curve, for a range of water levels the corresponding reliabilities have to be computed. For this, a FORM calculation is used (see Section 2.3.2). FORM linearizes a limit state function in the so-called design point. Every step, it computes the reliability β, and new design point values. In an iterative process, these steps are repeated until there is convergence between the input design point values and the calculated design point values. When convergence is reached, the resulting β is the conditional reliability corresponding to a certain water level. The resulting design point values (for all parameters) represent the combination of parameter values that will most probably lead to failure.

As explained in Section 2.4.3, the macro-stability calculation models try a lot of different slip surfaces, and then determines the critical slip surface. However, to limit calculation time, the slip surface is fixed for the FORM calculation. This means that the FORM calculation only uses one slip surface, and computes the reliability given that single slip surface. However, it is not certain that this slip surface will also be the critical one. Therefore, after performing the FORM calculation, a single-run is performed in the D-GeoStability software, not anymore fixing the slip surface, and using the design point values (resulting from the performed FORM calculation) as parameter input. The D-GeoStability software then returns the critical slip surface for these parameter values. If this resulting critical slip surface is the same as the one used in the FORM calculation, the fixed slip surface used in FORM was actually the critical one and the resulting reliability is the end result. If not, the FORM calculation is repeated, now using the resulting critical slip surface from the single-run as fixed slip surface. This process is repeated, until convergence is reached between the FORM fixed slip surface and the critical slip surface resulting from the single-run. This approach is illustrated in Figure B.3.

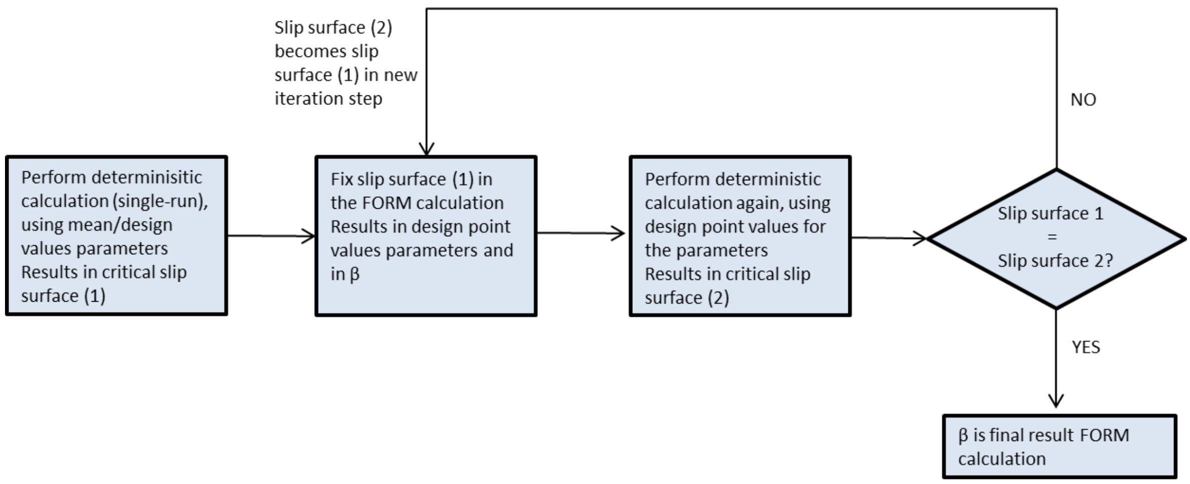


Figure B.3: Flowchart approach fixing slip surface

Note: In reality, every (relevant) possible slip surface contributes to the failure probability, and not just the critical slip surface. After all, all possible slip surfaces together form a series system; if one of the relevant slip surfaces occurs, failure occurs. However, in this study it is assumed that only the critical slip surface determines the probability. (Schweckendiek et al., 2017) states that this is a reasonable approximation, because different slip surfaces close to each other are (almost) fully correlated and completely different slip surfaces differ large in failure probability, and summing up smaller probabilities has only little influence on the total failure probability.

The final result of a FORM calculation for a certain water level consists of:

- The reliability factor β;
- The critical slip surface;
- Design point values for all parameters;
- Influence factors α^2 for all parameters, indicating the contribution of a certain variable to the total uncertainty.

Limit state function

To be able to perform the FORM calculation, a limit state function (Z) has to be defined (see Section 2.3.1). Z is defined in such a way, that for Z<0, one can speak of failure.

The result of a slip surface calculation in D-GeoStability is a critical slip surface with a corresponding factor of safety (FoS), which can be seen as the ratio between resistance and load. In an ideal model (without model uncertainties), one would speak of failure if the load exceeds the resistance (i.e. FoS < 1). However, as the Uplift Van model is not without uncertainties, the FoS has to be corrected using a model uncertainty factor m_d. This leads to the following limit state function:

$$Z = FoS * m_d - 1 \tag{B.2}$$

As mentioned above, FoS is the result from the slip surface calculation in the D-GeoStability software. The model uncertainty factor m_d is in fact the inverse of the model factor γ_d as used in the semi-probabilistic method. (Schweckendiek et al., 2017) proposes to model m_d as a lognormal distribution, with mean 0.995 and standard deviation 0.033.

Note: The above mentioned distribution leads to a characteristic value of 0.942. The inverse thereof is 1/0.942 = 1.06, which is the value used for γ_d.

Parameters

The parameters from Table B.1 are put in as stochastic values: their input consist of the distribution type, mean value, and variation coefficient / standard deviation. Note that this does not hold for the water level; to be able to construct a fragility curve, the FORM calculation is performed several times for a range of water levels, which are put in as deterministic values.

By modelling the leakage lengths and intrusion length as stochastic values, the uncertainties in pore pressures in the dike body and subsoil are taken into account.

Correlations

In the fully probabilistic calculation, correlation between different parameters has to be specified. Correlation is a measure of mutual dependence between two variables. A correlation coefficient of zero (ρ=0) means the parameters are completely independent and a correlation coefficient of one (ρ=1) means that the parameters are completely dependent (Jonkman et al., 2016).

Correlation is important for the soil strength parameters. As mentioned before, undrained layers have been split up into a part below the dike and a part next to the dike, because they usually have a different stress history. However, one should keep in mind that in reality it is one layer. Therefore, the strength parameters S and m of the two parts (of the same layer) are modelled as completely correlated (ρ=1). Because of their different stress history the state parameters POP are independent. The POP values are used to calculate the effective yield stress values in the yield stress points. Therefore, the yield stress points are modelled as independent (ρ=0).

Furthermore, also mentioned before, for clay dikes the part above the phreatic line is modelled as separate layer. The shear strength of this part is calculated using the drained CSSM model based on the critical state friction angle φ_{cs}. The parameter φ_{cs} is modelled as completely correlated with the undrained shear strength ratio S of the saturated part of the dike, of which the shear strength is calculated using the undrained SHANSEP model. After all, if the clay of the dike body is weak, it will probably be weak both in the saturated and in the unsaturated part.

Note: In some cases the fully probabilistic FORM calculation did not lead to convergence. For these cases, the undrained aquitard layer was split up in horizontal direction into an upper part and a lower part, each containing a separate yield stress point. This way, convergence was reached most of the time

Progressive failure calculation choices

This section covers the specific choices that are made for the progressive failure calculations. Progressive failure refers to scenario 3 of Chapter 3. For this analysis, a lot of assumptions are made. It is always tried to make conservative assumptions, in order to prevent underestimation of the failure probability.

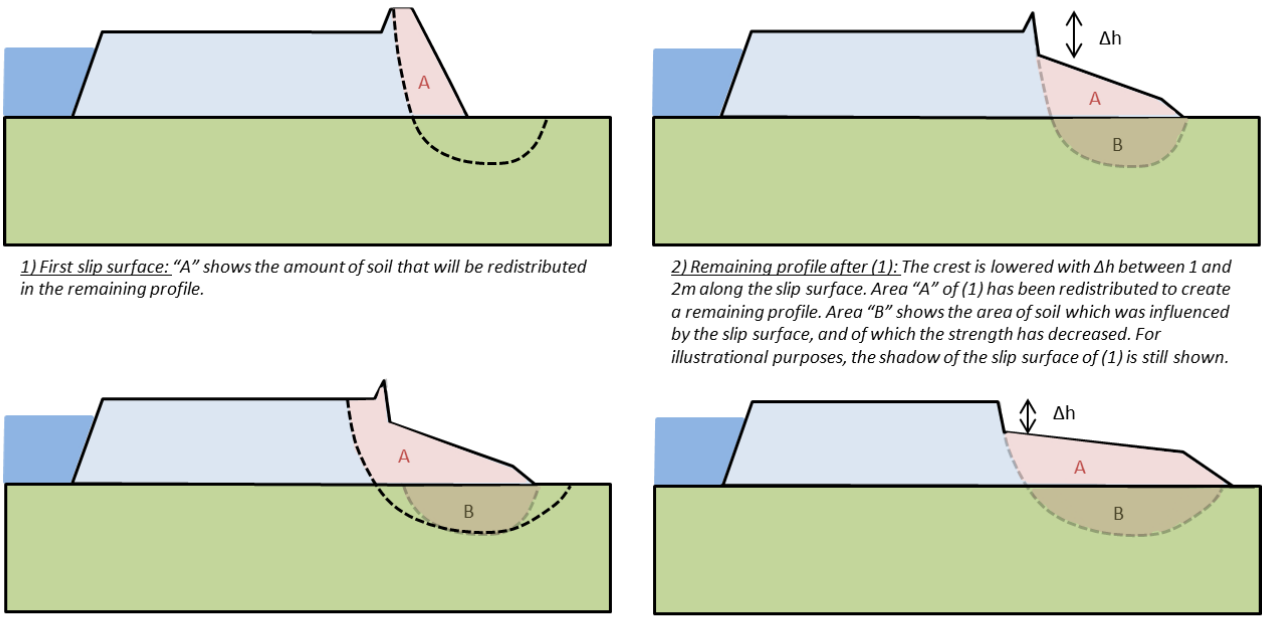
Water level

Scenario 3 holds for all water levels below foreshore level. For simplicity reasons, the progressive failure analysis is performed using only one water level, the highest water level for which the foreshore is not flooded. It is assumed that the conditional failure probability that is found for this water level, is also the conditional failure probability for the other water levels below foreshore level. This is a conservative assumption, because in reality the failure probability decreases for decreasing water level.

Remaining profile progressive failure

For the progressive failure analysis, after a sliding of soil has occurred a remaining profile has to be schematized, which will be used for the next slide.

When a volume of soil slides, along a slip surface, the soil on the left side of this slip surface will be lowered, whereas the soil at the right side will be pushed up; the soil rotates along the slip surface. This rotation continues until a new equilibrium is found and the soil is stable again. At the left side of the slip surface, a lowering of the surface level between 1m and 2m is mostly observed ('t Hart et al., 2016; ENW, 2009). This will be the starting point of the remaining profile. The rest of remaining profile is then found by redistributing the soil that rotated (only the part above surface level), starting from the starting point. This is illustrated in Figure B.4. Note that following this approach will always lead to a remaining profile that is more gentle than the initial profile. A more gentle profile generally also leads to more stability. This means that with every soil slide, the dike becomes more stable.



1) First slip surface: "A" shows the amount of soil that will be redistributed in the remaining profile.

2) Remaining profile after (1): The crest is lowered with Δh between 1 and 2m along the slip surface. Area "A" of (1) has been redistributed to create a remaining profile. Area "B" shows the area of soil which was influenced by the slip surface, and of which the strength has decreased. For illustrational purposes, the shadow of the slip surface of (1) is still shown.

3) Second slip surface: In the remaining profile of (2) a new volume of soil slides off along the drawn slip surface. Again, area "A" will be redistributed in the remaining profile.

4) Remaining profile after (3): Again, the crest is lowered with Δh between 1 and 2m, and area "A" of (3) has been redistributed. Note that area "B" (the affected soil) has grown due to the new slip surface.

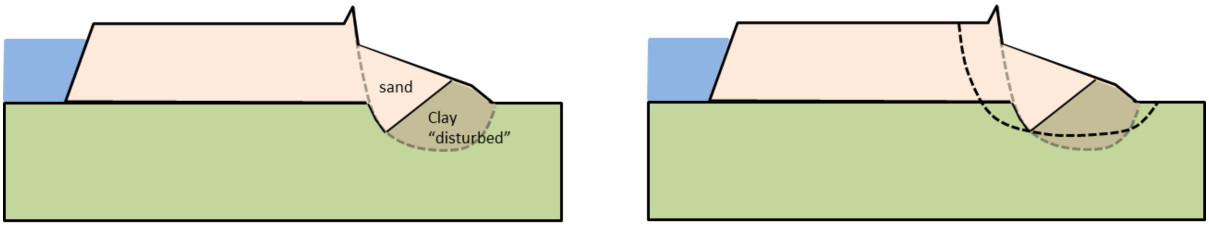
Figure B.4: Illustration approach schematization remaining profile

Besides, Figure B.4 shows a zone that was influenced by the slip surface, and in which the strength parameters are reduced. This is described below.

Normally, in the original situation the subsoil parameters are known from site investigation. However, after a sliding has occurred, the strength parameters of the soil may change; the soil may show brittle behaviour, meaning that the strength of the soil has decreased after the soil has deformed. Without extensive research it is impossible to predict the exact behaviour of the subsoil after a slide has occurred. Therefore, the progressive failure calculation is based on some conservative assumptions.

To account for brittle behaviour of the subsoil after sliding, a new zone (soil layer) is created in the subsoil at the location of the slip surface along which sliding has occurred. In this new zone the undrained shear strength ratio S is assumed to have a mean value of 0.15 and a standard deviation of 0.06, which leads to low, conservative, values for S. Furthermore, inside this new zone the POP (pre-overburden pressure, section 2.4.6) is decreased to zero, because the initial soil profile has been disrupted and there has not yet been a pre-overburden pressure in the new zone (there is no stress history).

As mentioned before, when a volume of soil slides, it rotates along the slip surface. In reality, this would mean that the dike material would be lowered at the left of the slip surface (end up below surface level) and at the right the aquitard material would be pushed up. Note that this is not done in the approach of Figure B.4, where only the material above the surface has been redistributed. The reason for this is illustrated in Figure B.5: In case the dike consists of sand, the slip surface of the next soil slide will always just go around the tip of the sand (that ended up below surface level), as the sand is significantly stronger than the “disturbed” clay (zone where the strength has been reduced). Therefore, the stability of the new slide depends very much on the location of the sand in the subsoil, which is an assumption. Therefore, the approach of Figure B.4 is used. Note that this leads to less stability, as the stronger material is kept above surface level. It is therefore a conservative choice.



Remaining profile of a sand dike, where the sand was lowered at the beginning of the slip surface and the clay was pushed up at the end. Note that this clay belongs to the part of which the strength has been reduced (disturbed clay).

A new slip surface will always go just around the tip of the sand area, because the sand is significantly more stable than the “disturbed”clay.

Figure B.5: Illustration remaining profile using lowering sand layer

Parallel System

Note that for failure to occur due to progressive failure, the successive soil slides must lead to a situation where there is (almost) no foreshore left, which is defined as failure (the exact definition of failure will be given later on in this appendix). All the individual successive slides up to the point of failure each have a probability of occurrence. Based on these probabilities, the total failure probability for progressive failure can be determined. Progressive failure forms a parallel system, meaning that for failure to occur, all individual soil slides have to occur. This is illustrated in the fault tree of Figure B.6. The total failure probability of the system is based on the correlation between the individual soil slides. The correlation coefficients will lie somewhere between 0 (fully independent) and 1 (fully dependent). The corresponding limits of the total failure probability are the following (Jonkman et al., 2016):

- Lower limit: The lower limit of the total failure probability is equal to the product of all individual failure probabilities. This limit corresponds to fully independent individual soil slides.
- Upper limit: The upper limit of the total failure probability is equal to the minimum of all the individual probabilities. This limit corresponds to fully correlated individual soil slides.

The actual total failure probability will be somewhere in between these limits, based on the correlations between the individual slides. The exact correlations are unknown, but it can be motivated that the individual slides are not completely independent, because the slip surfaces do for a large part cross the same subsoil. Therefore, there is some correlation between the individual slides, but it is unknown how large the correlation is. Therefore, as a safe and conservative choice, full correlation is assumed and the total failure probability is assumed to be equal to the upper limit, the minimum of all the individual probabilities. Note that in general, after each slide the dike becomes more stable, because every slide leads to a gentler remaining profile. Therefore, the minimum individual failure probability, and thus the total failure probability for progressive failure is generally equal to the individual probability of the last slide.

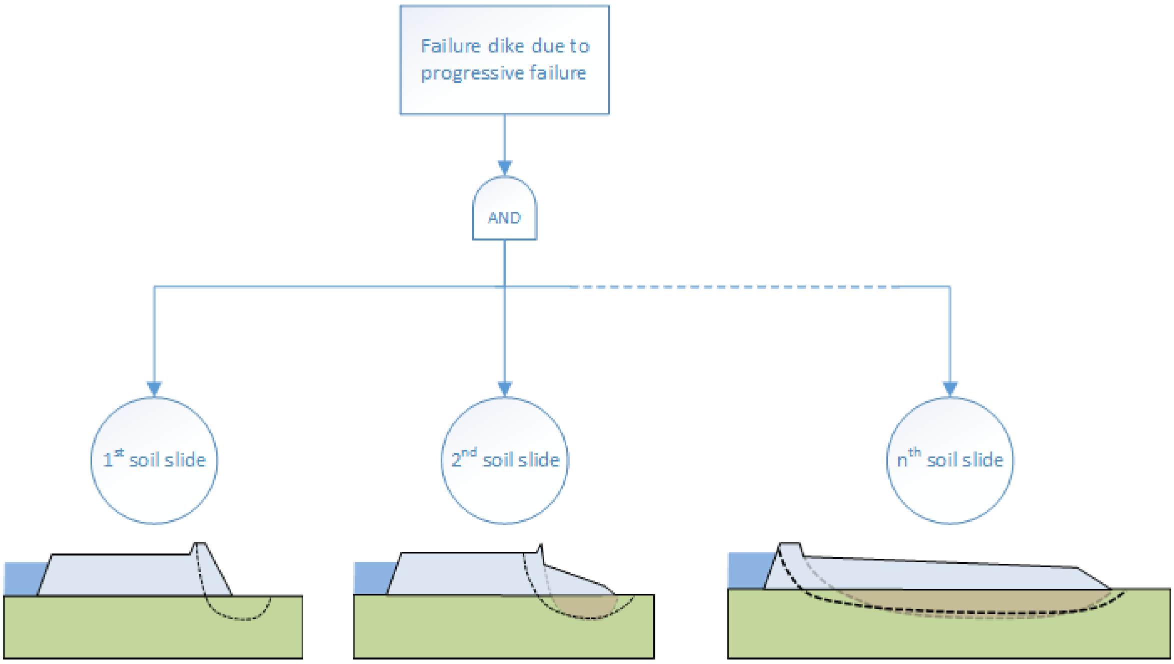


Figure B.6: Fault tree progressive failure

Stop Criterion

As mentioned before, after every soil slide the dike becomes more stable. This means that the probability of a new soil slide decreases with every slide, and thus the total failure probability decreases too. One can imagine that if the foreshore is sufficiently large, after a couple of slides the dike is so stable, that the probability of a new slide is negligible, leading to a negligible total failure probability for progressive failure. For this, one has to specify what probability can be considered as negligible. This depends on the target probability (maximum allowed failure probability) for macro-stability of a certain cross section. To meet safety requirements, the failure probability must be smaller than this target probability. It is assumed that if the failure probability for progressive failure is smaller than 1% of the target probability, one can speak of a negligible failure probability for progressive failure, because progressive failure then will not have a significant influence on the safety of the dike.

For this 1% value, an absolute lower limit has been determined to know when it is safe to stop the progressive failure analysis: *the stop criterion*. After all, if one is sure progressive failure will not lead to failure, one does not have to continue the analysis until the point of failure is reached. The stop criterion is based on an assumed lower limit for the target probability for macro-stability. Assuming a maximum allowed total failure probability of a dike trajectory of 1/100,000 and a dike trajectory length of 100km, formulas (2.3) and (2.17) lead to a lower limit target probability of $6.0*10^{-9}$ per year. For this lower limit target probability, the progressive failure will be negligible at $6.0*10^{-11}$, which corresponds to a reliability of $\beta = 6.44$. This is defined as the stop criterion. Once the probability of an individual soil slide is lower than the stop criterion, the failure probability for progressive failure is negligible, no matter what the actual target probability is, and the progressive failure analysis can be stopped.

Slip surfaces

The progressive failure analysis consists of different steps: the individual successive soil slides. In every step, a lot of different slip surface shapes are possible, leading to a soil slide. This means that there is an infinite amount of possible pathways leading to progressive failure. This is illustrated in the fault tree of Figure B.7.

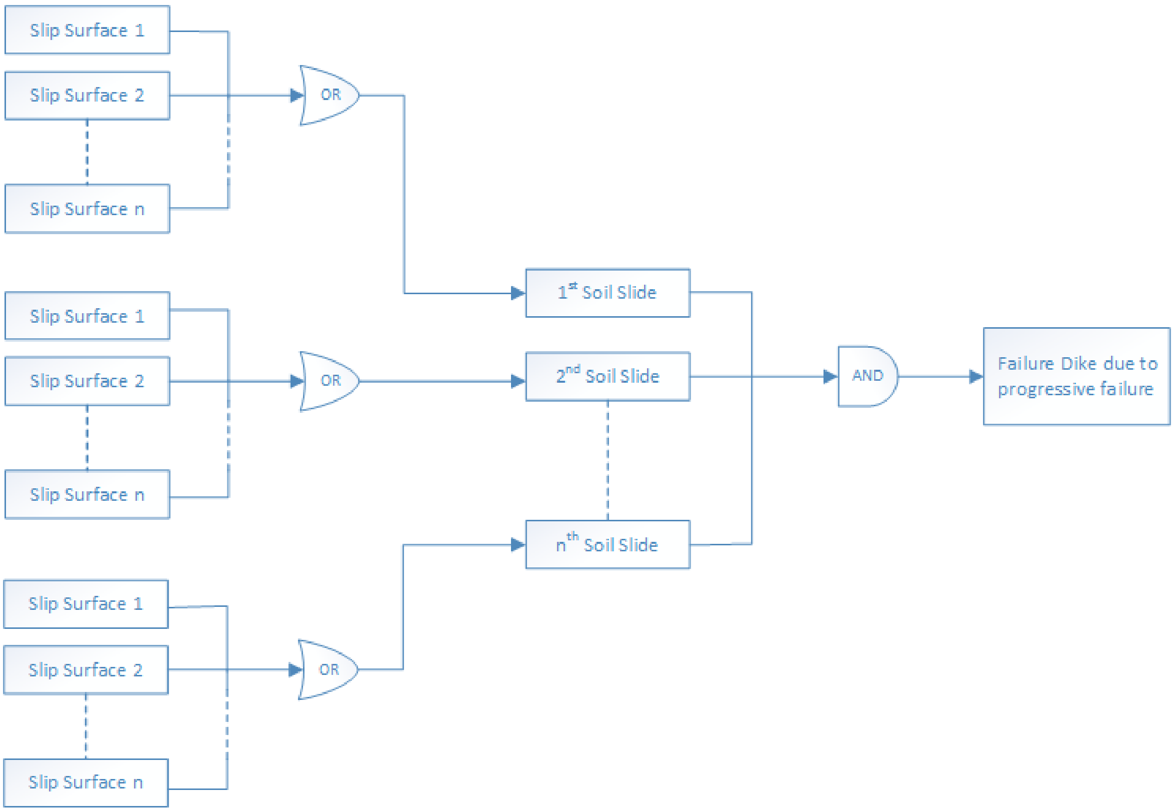
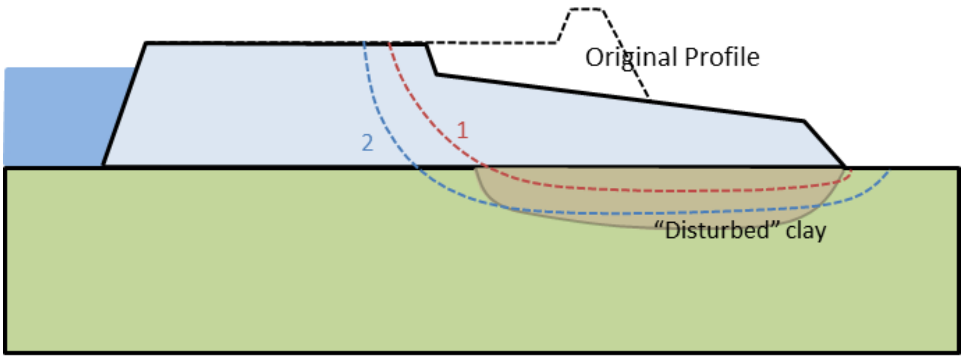


Figure B.7: Possible pathways leading to progressive failure

In a normal stability analysis, there are a lot of different possible slip surface shapes too. One simply choses the critical slip surface, which has the highest probability. After all, this slip surface leads to the most dangerous situation. However, in the progressive failure analysis the critical slip surface does not necessarily lead to the most dangerous situation. This is illustrated in Figure B.8.



Two possible slip surfaces in the same remaining profile. 1) has the largest probability of occurence, as it crosses the "disturbed clay" completely. 2) has as smaller probability of occurence because it also crosses the stronger undisturbed clay. However, 2) leads to a more dangerous situation for the following slides, because it increases the "disturbed" clay zone, while 1) does not.

Figure B.8: Illustration of the different slip surfaces used in the progressive failure analysis

Only looking at slip surfaces like (1), which are critical, will rather soon result in a stable situation, as the zone with “disturbed” clay does not grow. Slip surfaces like (2) may have a lower probability of occurrence, but they lead to a more dangerous situation for the next soil slides, as they grow the “disturbed” clay zone.

In the analysis, the following approach is used. Every step, slip surfaces like (2) are assumed, which correspond to a lower probability but cause the disturbed zone of clay to grow. This is repeated until the stop criterion is reached. Then, using the remaining profile after the last slip surface, one more soil slide along a surface like (1) is considered. This will take away another part of the foreshore, without further decreasing the failure probability; the probability of this final step should be higher than the probability of the previous step (which exceeded the stop criterion). The amount of foreshore that is lost due to this last slide (1), is for every step added to the amount of foreshore that would be lost using just slides like (2). This is illustrated in Example B.1.

Example B.1:

Imagine a progressive failure analysis on a certain dike. Just using slip surfaces like (2), leads to the following results:

Soil Slide	Probability	Foreshore loss
1st slide	1.0E-03	5m
2nd slide	1.0E-04	12m
3rd slide	1.0E-08	20m
4th slide	1.0E-11	30m

Note that for the 4th slide the stop criterions is exceeded (6.0E-11).

Imagine that a slip surface like (1) takes away another 5m of foreshore, corresponding to a probability of 1.0E-10 (which is higher than the probability of slide 4, which was required for this last step). The final results, including this extra loss due to slip surface (1) are:

Soil Slide	Probability	Foreshore loss
1st slide	1.0E-03	10m
2nd slide	1.0E-04	17m
3rd slide	1.0E-08	25m
4th slide	1.0E-11	35m

Failure Criterion

If the stop criterion is not exceeded, the progressive failure analysis will be continued until failure is reached. For this, it has to be defined what will be regarded as failure. Reference is made to (ENW, 2009), which states how wide the remaining dike (or in this case foreshore) has to be to still be safe.

Figure B.9 shows the schematization that is used in (ENW, 2009). The approach used is based on the assumption that after a primary circular slip surface, a secondary straight slip surface will occur, which will take away another part of the crest (or foreshore). This assumption is based on the fact that the upper part of the remaining profile after a circular slip surface is very steep, and thus unstable. To be sure the dike is safe, there has to be sufficient foreshore left after the a circular slip surface to make sure that a secondary straight slip surface will not lead to failure in terms of flooding. Note that this secondary slip surface is not the same as slip surface shape (2) in the previous section, which was just used to cover all possible circular slip surfaces. The progressive failure analysis, as described before, only considers the (elongated) circular slip surfaces (primary slip surfaces). The secondary straight slip surface is only used to define what is regarded as failure; it is used to determine the remaining width.

The secondary straight slip surface will be at an angle of 1:n, crossing the primary circular slip surface at a level of half the foreshore height (see Figure B.9). The requirement for the remaining profile is that the remaining width (the distance between the outer slope of the dike and the line 1:n) at a level equal to the water level must be at least 3m. Once this remaining width after a couple of successive (circular) soil slides is less than 3m, the situation is regarded as failure.

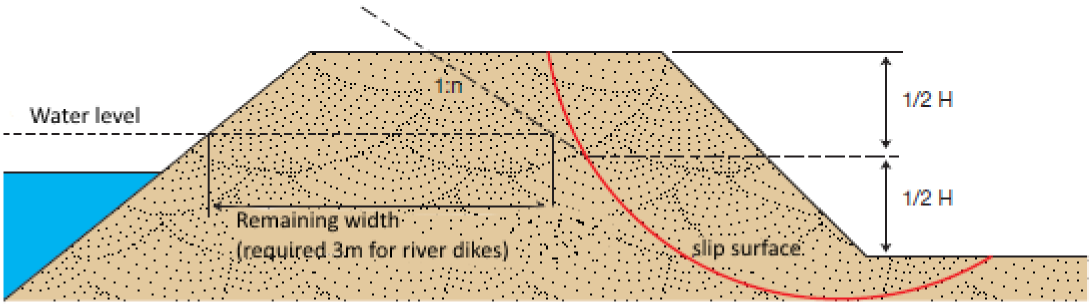


Figure B.9: Required remaining width schematization (ENW, 2009)

The slope of 1:n depends on the foreshore material. For a foreshore out of sand, the slope is at 1:4. For a clay foreshore, the slope is at 1:2 (but with the extra requirement that the undrained shear strength from laboratory studies is higher than 3.5*H, which is assumed to be the case in this Thesis).

Appendix C. Waternet Creator

Introduction

This Appendix covers the choices that are used by the Waternet Creator to schematize the waternet inside the dike and foreshore body and the subsoil. The waternet is created using different PL-lines. These PL-lines will be described separately. Finally, the interpolation between the different PL-lines will be described.

PL1 (Phreatic line)

The first PL-line is PL1, the phreatic line. As mentioned in Appendix B, standard schematizations are used for the phreatic line, following four different base cases. These cases are (*Basis Module Macrostabiliteit - Gebruikershandleiding*, 2016):

- Case 1A: “Clay dike on clay”
- Case 2A: “Sand dike on clay”
- Case 1B: “Clay dike on sand”
- Case 2B: “Sand dike on sand”

The standard schematizations for these cases are shown in Figure C.1 up to Figure C.3.

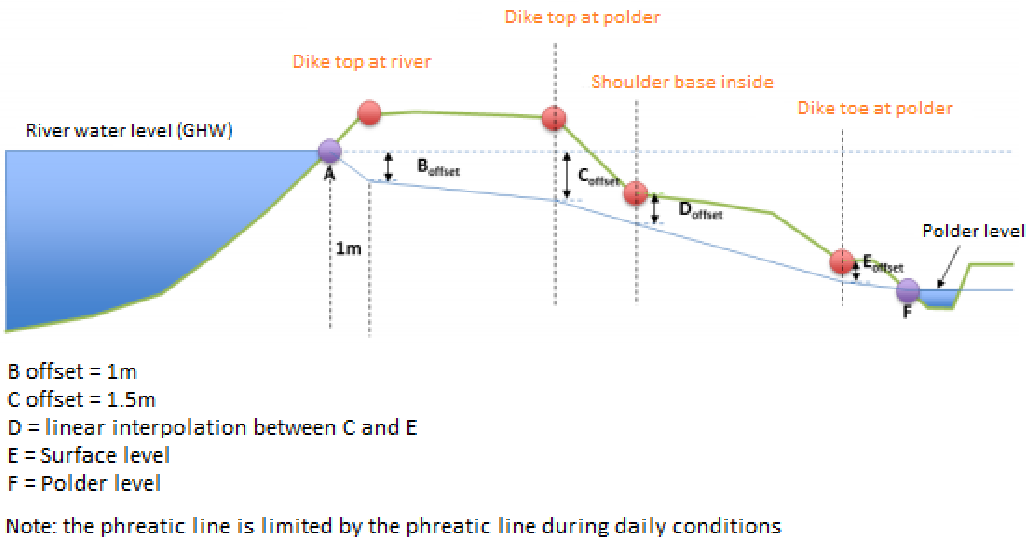


Figure C.1: Case 1A and 1B (*Basis Module Macrostabiliteit - Gebruikershandleiding*, 2016)

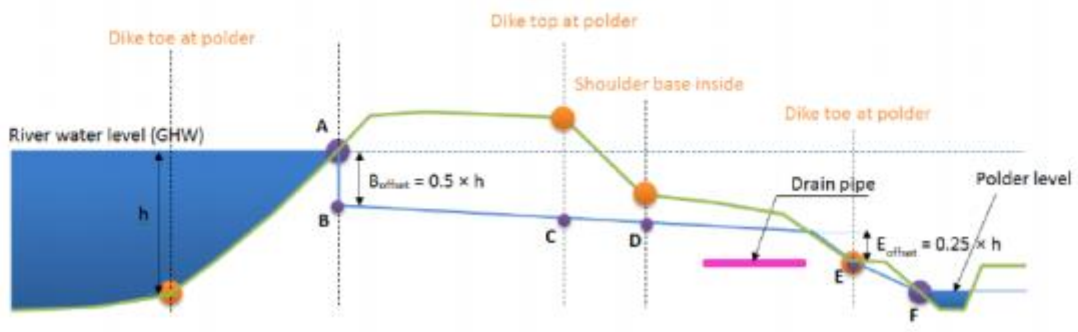


Figure C.2: Case 2A (*Basis Module Macrostabiliteit - Gebruikershandleiding*, 2016)

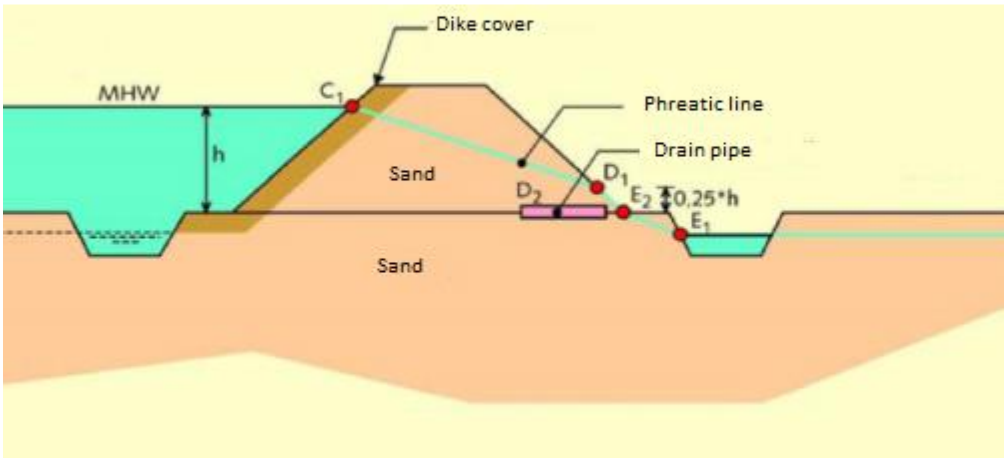


Figure C.3: Case 2B (*Basis Module Macrostabiliteit - Gebruikershandleiding*, 2016)

PL2

The next PL-line that will be discussed is PL2: the aquifer head during daily conditions. As an input for the Waternet Creator, one has to specify the value of PL2 at the left and the right boundary of the dike geometry, after which the Waternet Creator schematizes PL2 by using linear interpolation between these values. In this study, the PL2 values at the boundaries are determined using the following analytical equations (TAW, 2004):

$$W_1 = \frac{\lambda_1}{kD}$$

$$W_3 = \frac{\lambda_3}{kD}$$

$$\varphi_2 = \varphi_3 + (\varphi_0 - \varphi_3) * \frac{\lambda_3}{\lambda_1 + L_2 + \lambda_3}$$

$$\varphi_1 = \varphi_3 + (\varphi_0 - \varphi_3) * \frac{L_2 + \lambda_3}{\lambda_1 + L_2 + \lambda_3}$$

In which:

φ_0	Water level river (Daily water level) [m+NAP]
φ_3	Polder level [m+NAP]
φ_1	PL2 value outside toe dike/foreshore [m+NAP]
φ_2	PL2 value inner toe dike [m+NAP]
k	Hydraulic conductivity aquifer [m/s]
D	Thickness aquifer [m]
λ_1	Outer leakage length [m]
λ_3	Inner leakage length [m]
L_2	Distance between inner toe dike and outside toe dike/foreshore [m]

The value of PL2 can then be determined at each point using the following formulas:

Outside direction: $\varphi(x) = \varphi_0 - (\varphi_0 - \varphi_1)e^{(a+x)/\lambda_1}$

Inside direction: $\varphi(x) = \varphi_3 + (\varphi_2 - \varphi_3)e^{(a+x)/\lambda_3}$

The reference points for the coordinates are shown in Figure C.4. By choosing the left boundary and right boundary as x-coordinates respectively, the values of PL2 needed for the input of the Waternet Creator are obtained.

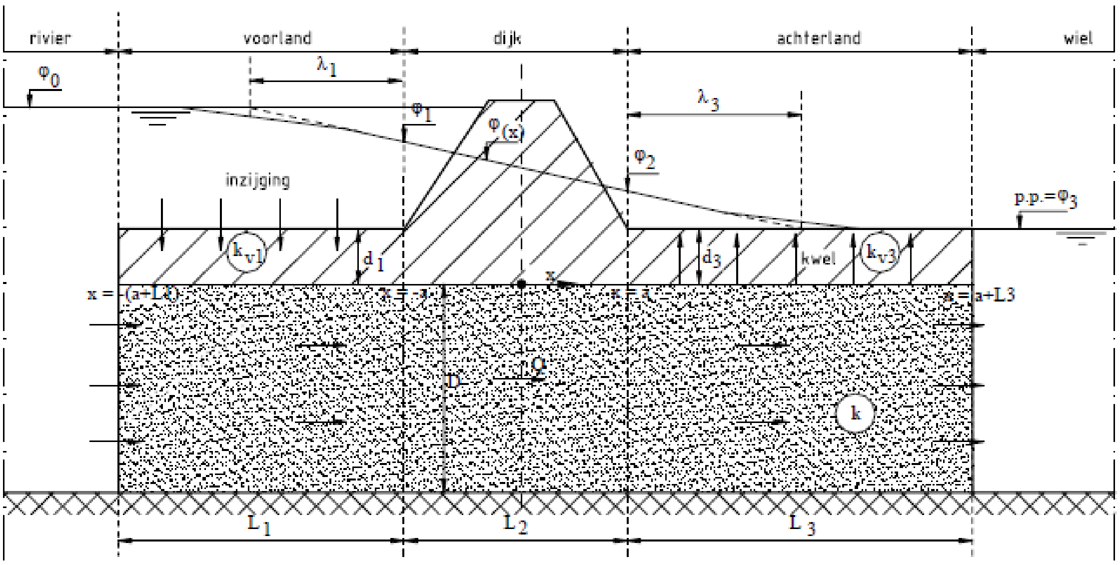


Figure C.4: Reference coordinates analytical calculations of aquifer head development (TAW, 2004)

PL3

The last PL-line is PL3, which is the head in the aquifer during the high water condition used in the assessment. The PL3 is generated automatically by the Waternet Creator, based on the leakage lengths and PL2. For this, Waternet Creator uses the following formulas (*Basis Module Macrostabiliteit - Gebruikershandleiding*, 2016):

$$\varphi_{3;dike\ top\ river} = \frac{MHW - GHW}{1 + \frac{\lambda_{outer}}{\lambda_{inner}}} + \varphi_2(X_{dike\ top\ river})$$

$$\varphi_3(X) = \left[\varphi_{3;dike\ top\ river} - \varphi_2\left(X_{dike\ top\ river}\right)\right] * e^{\left(-\frac{\Delta X}{\lambda_{inner}}\right)} + \varphi_2(X)$$

In which:

λ_{outer}	Outer leakage length [m]
λ_{inner}	Inner leakage length [m]
MHW	Water level used in assessment [m+NAP]
GHW	Mean high water (daily situation) [m+NAP]
$\varphi_2(X)$	Value of PL2 at location X [m+NAP]
$\varphi_{3;dike\ top\ river}$	Value of PL3 at the location of dike top river [m+NAP]
$\varphi_3(X)$	Value of PL3 at location X [m+NAP]

Furthermore, the Waternet Creator automatically reduces PL3 in case of uplift conditions. Uplift conditions are present when PL3 exceeds the effective weight of the aquitard (limit potential) at a certain location, causing the aquitard to be lifted up and "float" on top of the aquifer. The shear strength in the uplift zone is then reduced to zero. To avoid negative effective stresses in the soil, the PL3 is reduced to the limit potential, leading to effective stresses of zero.

PL lines per layer

Up to this point, the development of the different PL-lines in x-direction has been discussed. This section discusses how the PL-lines are used to obtain the development of the pore pressures in vertical direction at a certain x-coordinate. This development is illustrated in Figure C.5, and can be described as follows (from bottom to top):

- In the aquifer the pore pressures are hydrostatic based on the PL 3 head line.
- In the intrusion zone of the aquitard (with a thickness equal to the intrusion length, Appendix B) the pore pressures develop linearly from PL3 (at the bottom) to PL2 (at the top).
- In the rest of the aquitard the pore pressures develop linearly from PL2 (at the bottom) to PL1 (at the top).
- In the dike body the pore pressures are hydrostatic based on the PL1 head line.

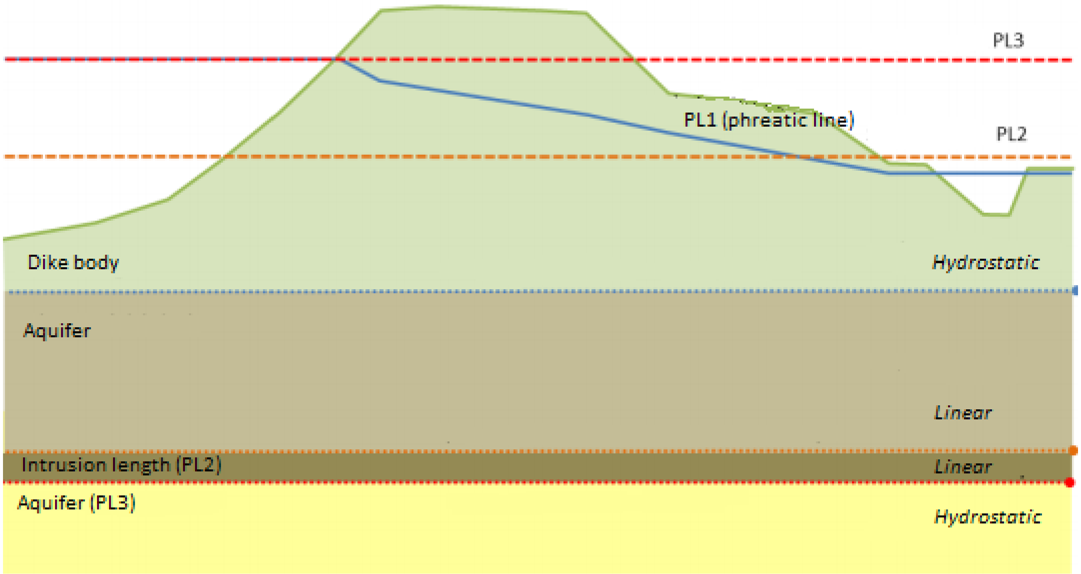


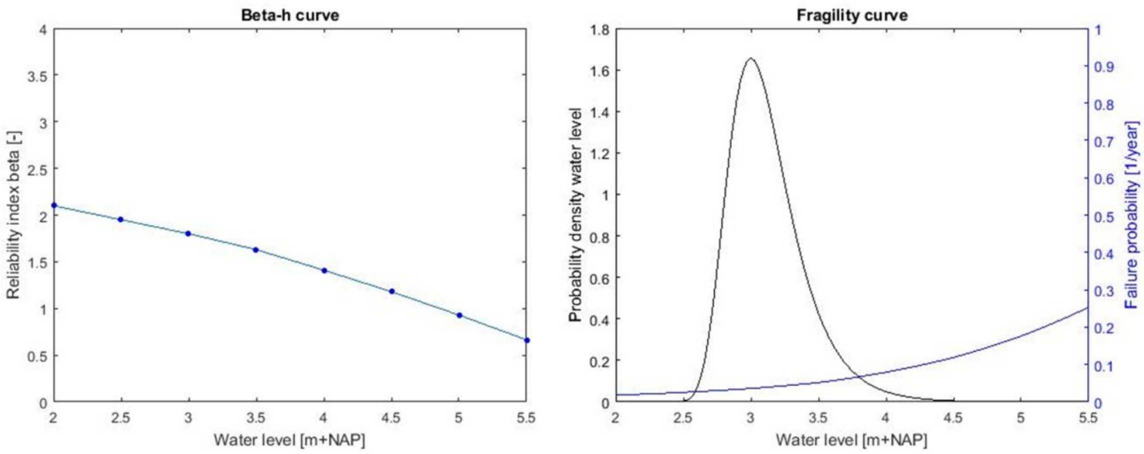
Figure C.5: Illustration PL lines per layer

Appendix D. Results fully probabilistic calculations dikes without foreshores

This Appendix shows the results of the fully probabilistic calculations for the test cases without foreshores. For every test case the results of the FORM calculations for all water levels, the (β-h)-curve and the fragility curve are shown.

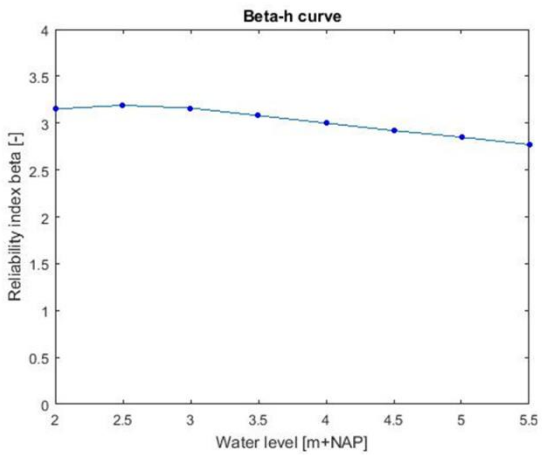
Test Case 1

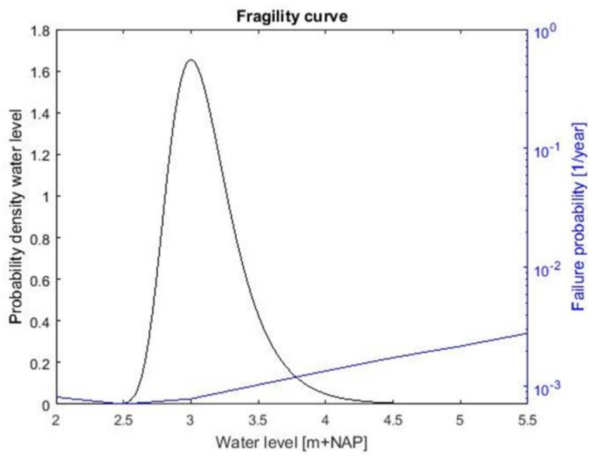
	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	2.1	1.95	1.8	1.63	1.41	1.18	0.91	0.664
Failure probability [-]	1.78 E-02	2.56 E-02	3.59 E-02	5.16 E-02	7.93 E-02	1.19 E-01	1.76 E-01	2.53 E-01
Influence factor (α^2) [%]								
λ_{in}	0	0.027	0.109	0.255	0.47	0.888	1.431	2.157
λ_{out}	0	0.018	0.075	0.175	0.323	0.603	0.899	1.508
Intrusion length	0	0.029	0.084	0.265	0.871	1.088	2.152	3.789
Yield (39.12)	14.611	15.228	15.846	16.539	17.352	17.456	18.132	18.992
Yield(70)	3.076	3.159	3.232	3.297	3.349	4.153	4.148	4.122
φ (Sand dike)	0.34	0.333	0.328	0.318	0.297	0.268	0.243	0.249
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay.B; Clay.N)	73.095	72.366	71.52	70.411	68.745	67.035	64.683	61.123
m (Clay.B; Clay.N)	1.042	1.095	1.156	1.227	1.307	1.489	1.597	1.839
m_d	7.836	7.745	7.65	7.513	7.286	7.021	6.715	6.221



Test Case 2

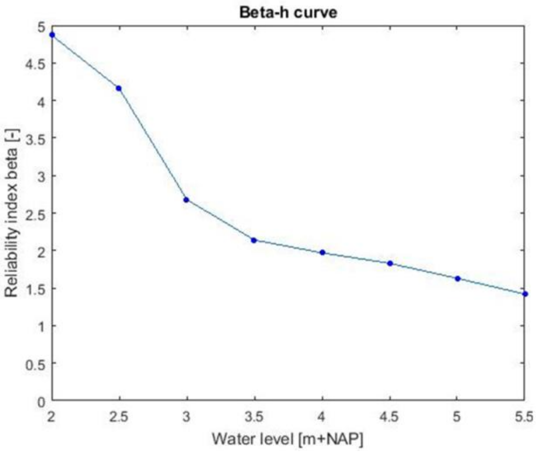
	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	3.15	3.19	3.16	3.08	3	2.92	2.85	2.77
Failure probability	8.16	7.11	7.89	1.04	1.35	1.75	2.19	2.80
[-]	E-04	E-04	E-04	E-03	E-03	E-03	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0	0.08	0.195	0.385	0.656	1.33	1.532
λ_{out}	0	0	0.048	0.12	0.24	0.42	0.856	2.74
Intrusion length	0	0	0.447	1.273	2.316	5.017	7.507	10.102
Yield Dike(39.12)	1.672	1.486	0.953	1.244	1.583	1.82	2.175	2.6
Yield Aquitard (39.12)	28.04	26.557	22.634	22.774	22.879	22.221	20.989	20.466
Yield(175)	5.213	5.302	5.725	5.691	5.641	4.957	5.65	5.119
φcs (Clay dike)	5.767	5.32	3.768	3.98	4.251	4.421	4.614	5.014
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	48.282	50.159	55.071	53.42	51.369	49.282	45.809	41.476
m (Clay Dike)	0.014	0.013	0.008	0.016	0.035	0.061	0.112	0.208
m (Clay,U; Clay,N)	0.498	0.545	0.635	0.736	0.868	0.883	1.116	1.389
m_d	10.514	10.619	10.631	10.552	10.435	10.262	9.841	9.353

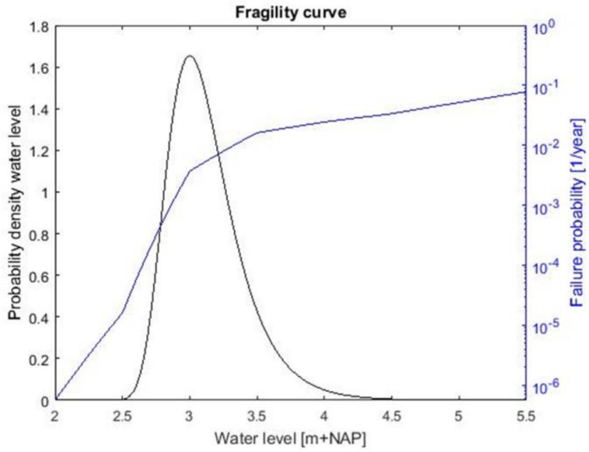




Test Case 3

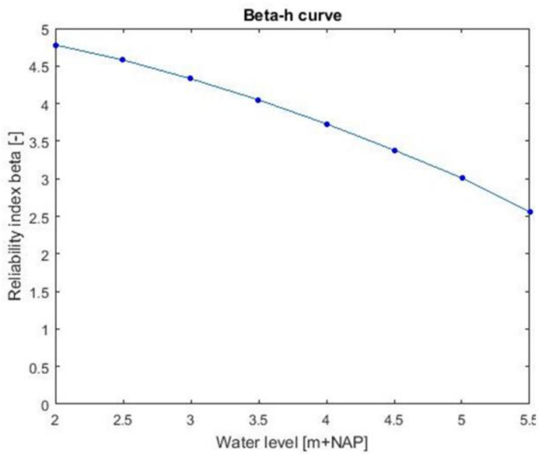
	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	4.87	4.16	2.68	2.14	1.97	1.83	1.63	1.42
Failure probability	5.58	1.59	3.68	1.61	1.79	3.36	5.16	7.78
[-]	E-07	E-05	E-03	E-02	E-02	E-02	E-02	E-02
Influence factor (α^2) [%]								
λ_{in}	0	2.616	0.001	0	0	0	0	0
λ_{out}	0	1.387	16.067	0.012	0.016	0.025	0.033	0.043
Intrusion length	1.427	5.782	1.386	5.093	6.492	8.269	9.328	10.144
Yield (139.12)	29.542	24.982	29.081	48.62	49.4	50.686	51.694	52.806
Yield(175)	1.095	0.485	0.297	0	0	0	0	0
φ (Sand dike)	2.058	0.938	0.647	0.457	0.399	0.519	0.442	0.374
φ (Sand Pleistocene)	0	1.744	0.491	0	0	0	0	0
S (Clay,B; Clay,N)	40.639	39.406	35.826	36.73	35.1	32.005	30.497	29.076
m (Clay,B; Clay,N)	14.169	7.059	5.662	2.061	1.992	2.038	1.994	1.958
m_d	11.069	15.602	10.544	7.026	6.601	6.459	6.013	5.598

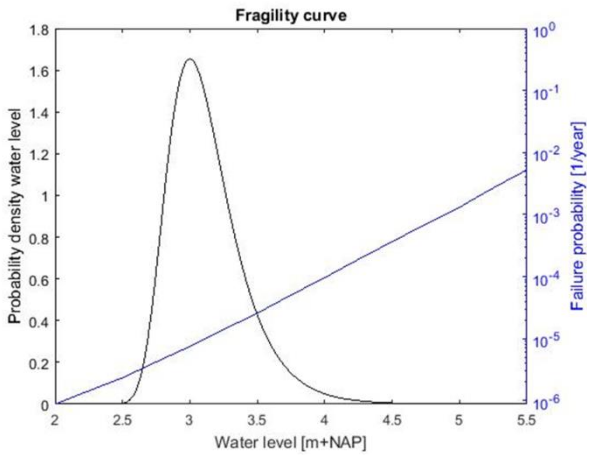




Test Case 4

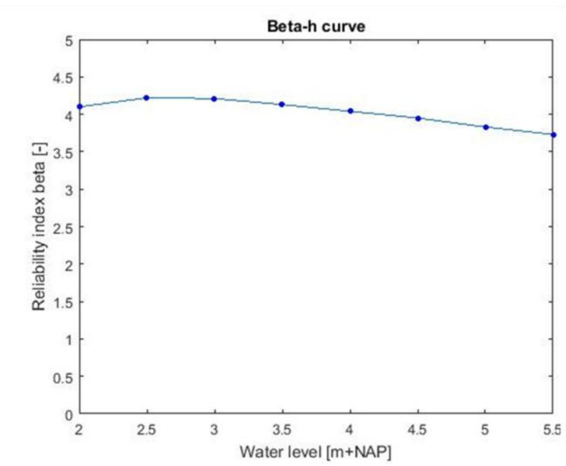
	Water level [m+NAP]							
	2	2.5	3	3.5	4	4.5	5	5.5
Failure analysis								
Reliability β [-]	4.78	4.58	4.33	4.05	3.73	3.38	3.01	2.56
Failure probability	8.76	2.32	7.46	2.56	9.57	3.62	1.31	5.23
[-]	E-07	E-06	E-06	E-05	E-05	E-04	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.045	0.189	0.462	0.893	1.482	2.891	14.479
λ_{out}	0	0.031	0.13	0.319	0.616	1.012	1.915	4.249
Intrusion length	0.117	0.404	0.044	0.722	3.04	7.333	7.468	13.38
Yield (139.12)	14.162	14.617	15.435	16.272	17.089	18.097	18.449	15.822
Yield(175)	3.784	3.887	4.019	4.103	4.111	4.509	5.273	4.019
φ (Sand dike)	0.54	0.519	0.498	0.468	0.427	0.361	0.314	0.218
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	68.223	67.452	66.735	64.97	61.676	55.957	52.677	39.333
m (Clay,B; Clay,N)	1.125	1.185	1.271	1.374	1.475	1.666	2.094	1.908
m_d	12.049	11.86	11.678	11.31	10.672	9.584	8.918	6.592

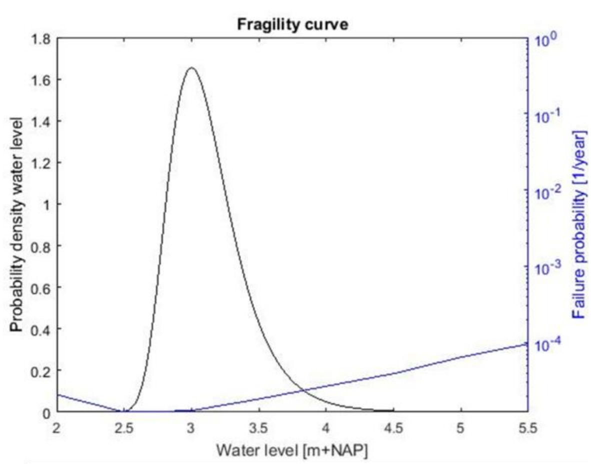




Test Case 5

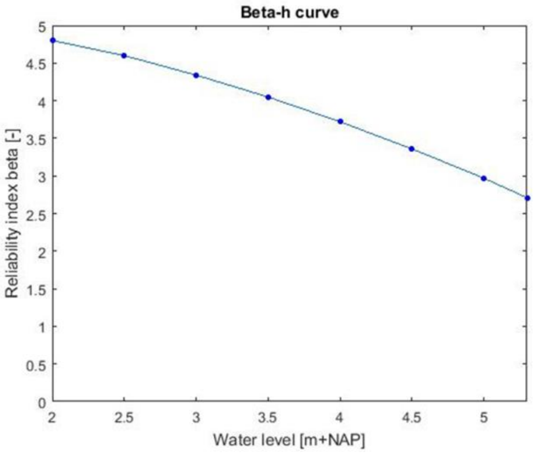
	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	4.1	4.22	4.21	4.13	4.04	3.95	3.83	3.73
Failure probability	2.07	1.22	1.28	1.81	2.67	3.91	6.41	9.57
[-]	E-05	E-05	E-05	E-05	E-05	E-05	E-05	E-05
Influence factor (α^2) [%]								
λ_{in}	0	0	0.105	0.263	0.555	1.183	1.922	1.088
λ_{out}	0	0	0.064	0.165	0.36	0.787	1.289	3.326
Intrusion length	0.149	0.01	0.671	1.547	4.941	10.371	16.455	17.285
Yield Dike(39.12)	2.776	1.662	1.06	1.354	1.798	2.067	2.289	2.764
Yield Aquitard (39.12)	30.849	25.334	21.679	21.876	21.66	20.134	18.941	19.458
Yield(75)	6.058	5.659	6.069	6.056	5.919	6.598	5.593	5.345
φcs (Clay dike)	7.24	5.826	4.135	4.288	4.484	4.404	4.421	4.843
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	40.852	48.559	53.221	51.482	47.701	42.648	38.052	34.929
m (Clay Dike)	0.023	0.014	0.009	0.017	0.039	0.072	0.122	0.23
m (Clay,U; Clay,N)	0.466	0.507	0.574	0.667	0.72	0.842	0.815	0.972
m_d	11.586	12.43	12.414	12.286	11.822	10.895	10.1	9.76

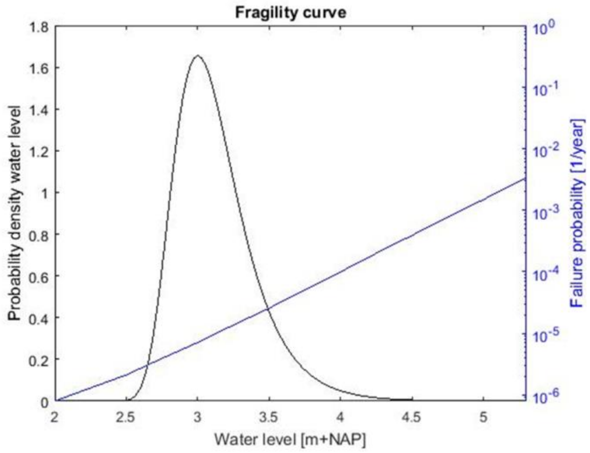




Test Case 6

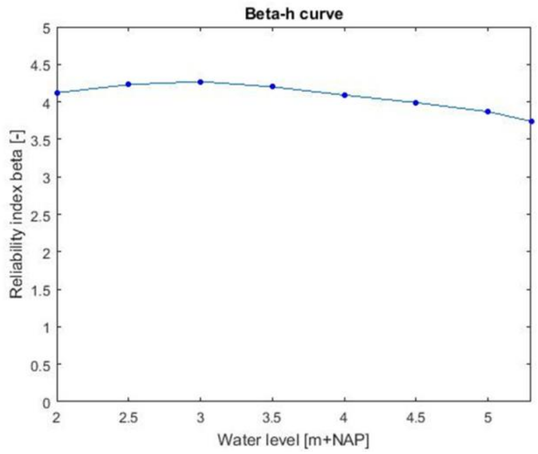
	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.3**
Failure analysis								
Reliability β [-]	4.80	4.60	4.34	4.05	3.72	3.36	2.97	2.71
Failure probability [-]	7.93 E-07	2.11 E-06	7.12 E-06	2.56 E-05	9.96 E-05	3.90 E-04	1.49 E-03	3.36 E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.045	0.193	0.472	0.951	1.543	2.337	3.329
λ_{out}	0	0.032	0.134	0.33	0.659	1.064	1.59	2.163
Intrusion length	0.143	0.392	0.033	0.672	3.289	7.741	13.445	17.738
Yield (38.4)	14.508	14.771	15.607	16.47	17.734	18.311	18.5	18.228
Yield(70)	3.891	4.021	4.155	4.239	4.713	4.593	4.345	4.034
φ (Sand dike)	0.518	0.475	0.454	0.426	0.378	0.329	0.275	0.24
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	67.804	67.309	66.559	64.777	60.253	55.24	49.336	44.838
m (Clay,B; Clay,N)	1.228	1.277	1.374	1.493	1.747	1.83	1.898	1.951
m_d	11.908	11.677	11.491	11.122	10.277	9.35	8.275	7.478

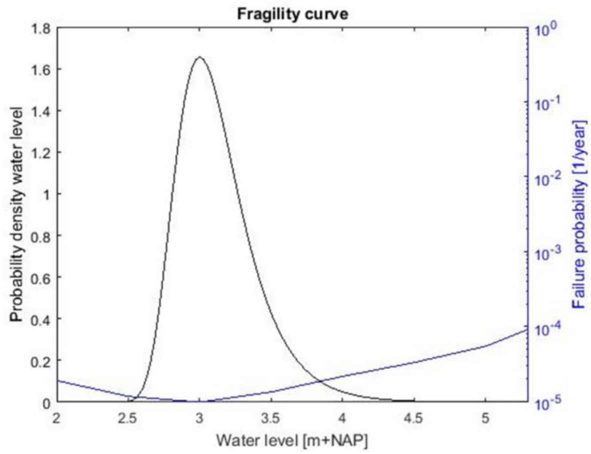




Test Case 7

	Water level [m+NAP]							
	2	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.3**
Failure analysis								
Reliability β [-]	4.12	4.23	4.27	4.2	4.09	3.99	3.87	3.74
Failure probability	1.89	1.17	9.77	1.33	2.16	3.3	5.44	9.2
[-]	E-05	E-05	E-06	E-05	E-05	E-05	E-05	E-05
Influence factor (α^2) [%]								
λ_{in}	0	0.002	0.036	0.266	0.566	1.223	2.08	0.043
λ_{out}	0	0.003	0.02	0.169	0.373	0.824	1.4	0.314
Intrusion length	0.543	0.077	0.162	1.408	4.798	10.281	16.064	19.028
Yield Dike (38.4)	3.218	1.989	1.414	1.556	1.906	2.167	2.463	2.862
Yield Aquitard (38.4)	32.253	26.522	22.861	21.833	21.728	20.214	19.172	19.556
Yield(175)	6.444	5.732	6.128	6.233	6.124	6.83	5.77	5.553
φcs (Clay dike)	6.562	6.041	4.513	4.156	4.289	4.247	4.256	4.657
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	39.363	46.813	51.777	51.358	47.632	42.447	37.75	36.607
m (Clay Dike)	0.531	0.573	0.659	0.751	0.816	0.947	0.934	1.158
m (Clay,U; Clay,N)	0.027	0.017	0.012	0.02	0.042	0.079	0.142	0.221
m_d	11.059	12.231	12.418	12.25	11.727	10.741	9.969	10.002





Appendix E. Semi-probabilistic assessment vs. Safety Standard

As mentioned before, the semi-probabilistic method uses a design water level. The design water level is chosen such that its exceedance frequency is equal to the safety standard of the dike.

Looking more closely at this procedure, a remarkable fact can be noticed. If other safety standards would be assigned to a certain dike, the design water level would change. Using a different design water level in the semi-probabilistic calculation will result in a different FoS. Using a different FoS in the calibration formula (2.20) results in a different approximated failure probability. This is illustrated in the flowchart of Figure E.1.

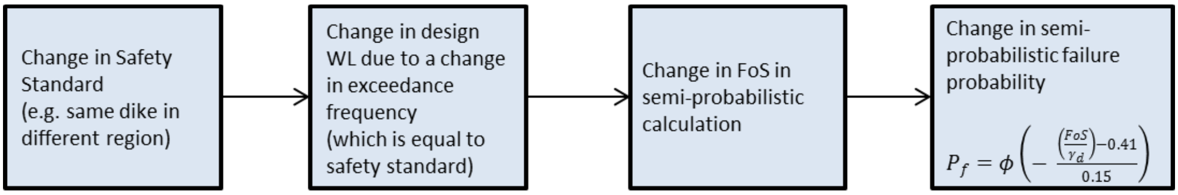


Figure E.1: Flowchart dependence semi-probabilistic result

In short, the above means that the failure probability of a dike, calculated using the semi-probabilistic method, depends on the safety standard of the dike. So changing the safety standard would mean that one gets a different failure probability as a result of the semi-probabilistic calculation.

It may seem odd that the (semi-probabilistic) failure probability of a dike depends partly on the required failure probability. However, remember that the semi-probabilistic method leads to an approximated failure probability, based on a calibration study. In the calibration study, it was chosen to use a design water level with an exceedance frequency equal to the safety standard. Based on the semi-probabilistic results of different cases using this design water level and the fully probabilistic results, a relation was calibrated to transform the semi-probabilistic FoS into an approximated failure probability. The design water level therefore does not really have a physical meaning in the semi-probabilistic method, but is just based on a choice which was also used in the calibration. Besides, there is an analogy with the Eurocodes, which describe the safety requirements for houses and utility buildings. In here, different reliability classes are determined, which describe the amount of reliability different types of buildings require. For every reliability class different values for partial safety factors are used in the semi-probabilistic calculations. So in the Eurocodes it also holds that the result of the semi-probabilistic calculation depends on the reliability requirement.

As mentioned, the semi-probabilistic failure probability changes if one changes the safety standard. The result of the fully probabilistic calculation stays the same, because it is independent of the safety standard. After all, the fully probabilistic failure probability uses the full water level distribution and not just a design point in this distribution that is governed by the safety standard. In the fully probabilistic calculation the water levels do have a physical meaning, and are not just based on a choice. It makes sense that the fully probabilistic failure probability (which is not an approximation, but more closely resembles the reality) does not change, because nothing has changed about the dike itself. Only the safety standards, the safety requirements the dike has to fulfil, have changed.

Concluding the above, if one changes the safety standard, the semi-probabilistic probability will change, but the fully probabilistic failure probability will stay the same. This means that the conservatism (or underestimation) caused by the semi-probabilistic rule also depends on the safety standard.

To illustrate this, Table E.1 and Table E.2 show, for two different water level distributions (Gumbel, u = 3, α = 4.5 and α = 7.5 respectively), the fully probabilistic failure probability (2nd column) and the semi-probabilistic failure probabilities for different safety standards, and thus different design water levels. Semi-probabilistic failure probabilities that are higher than the fully probabilistic probabilities (conservative) are depicted in green, while semi-probabilistic results that underestimate the failure probability are depicted in red. From these results it can be concluded that for stricter safety standards (lower exceedance frequency), the semi-probabilistic method becomes more conservative, because the design water level increases and the approximated failure probability increases, while the fully probabilistic failure probability stays the same.

Test Case	Pf prob.	Pf semi-prob (1/30,000)	Pf semi-prob (1/10,000)	Pf semi-prob (1/3000)	Pf semi-prob (1/1000)	Pf semi-prob (1/300)	Pf semi-prob (1/100)
1	4,07E-02	9,27E-03	6,92E-03	5,59E-03	4,33E-03	3,33E-03	2,60E-03
3	8,68E-04	2,39E-04	2,22E-04	2,07E-04	1,88E-04	1,67E-04	1,52E-04
4	6,40E-03	2,13E-03	1,61E-03	1,31E-03	1,04E-03	8,24E-04	6,49E-04
5	1,52E-05	7,57E-05	4,41E-05	2,97E-05	1,83E-05	1,14E-05	7,26E-06
6	1,47E-05	1,93E-05	1,43E-05	1,28E-05	1,14E-05	1,02E-05	9,12E-06
7	1,50E-05	8,38E-05	4,53E-05	2,97E-05	1,83E-05	1,11E-05	7,06E-06
8	1,14E-05	1,39E-05	1,28E-05	1,14E-05	9,92E-06	8,14E-06	7,06E-06

Table E.1: Results for different safety standards (alpha = 4.5), showing for all test cases the probabilistic failure probability and the semi-probabilistic failure probability for different safety standards. ***Green = conservative, Red = underestimation***

Test Case	Pf prob.	Pf semi-prob (1/100,000)	Pf semi-prob (1/30,000)	Pf semi-prob (1/10,000)	Pf semi-prob (1/3000)	Pf semi-prob (1/1000)	Pf semi-prob (1/300)	Pf semi-prob (1/100)
1	3.84E-02	4.33E-03	3.66E-03	3.15E-03	2.70E-03	2.31E-03	1.97E-03	1.71E-03
3	8.39E-04	1.88E-04	4.55E-05	1.63E-04	1.52E-04	1.41E-04	1.31E-04	1.22E-04
4	5.00E-03	1.13E-03	9.78E-04	7.73E-04	7.56E-04	6.64E-04	5.82E-04	5.20E-04
5	1.02E-05	1.88E-05	1.35E-05	1.05E-05	7.91E-06	5.94E-06	4.44E-06	3.31E-06
6	1.38E-05	1.17E-05	1.08E-05	9.92E-06	9.38E-06	8.61E-06	7.91E-06	7.26E-06
7	9.86E-06	1.88E-05	1.35E-05	1.02E-05	7.69E-06	5.77E-06	4.19E-06	3.12E-06
8	1.07E-05	1.21E-05	1.08E-05	9.92E-06	9.38E-06	8.37E-06	7.47E-06	6.86E-06

Table E.2: Results for different safety standards (alpha = 7.5), showing for all test cases the probabilistic failure probability and the semi-probabilistic failure probability for different safety standards. ***Green = conservative, Red = underestimation***

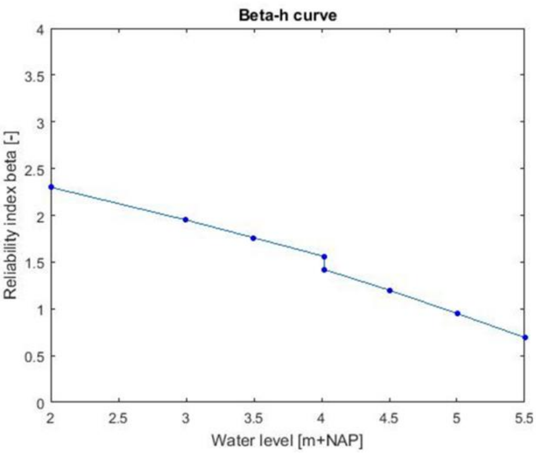
Appendix F. Results fully probabilistic calculations WBI failure definition

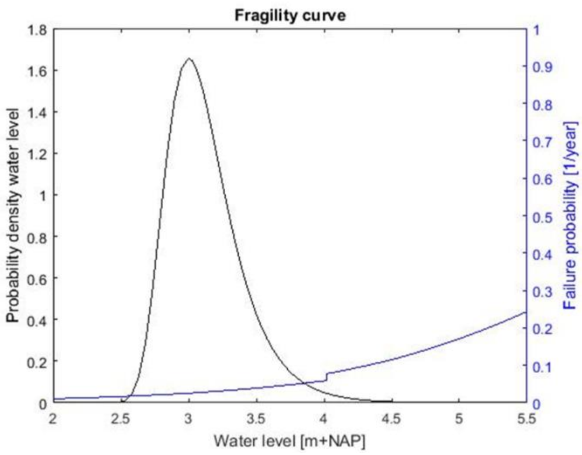
This appendix summarizes the results of the fully probabilistic calculations of the test cases (with foreshores), using the WBI failure definition for all water levels. The table below shows for every test case the conditional reliability and the location of the entrance point of the critical slip surface for all water levels. The next pages show for every test case the results of the FORM calculations for all water levels, the (β-h)-curve and the fragility curve.

Case 1	water level [m+NAP]	β [-]	entrance point	Case 5	water level [m+NAP]	β [-]	entrance point
	2	2.3	right part crest		2	4.13	upper part inner slope
	3	1.95	right part crest		2.5	4.31	right edge crest
	3.5	1.76	right part crest		3	4.27	right part crest
	4.02	1.56	right part crest		3.5	4.13	right part crest
	4.021	1.42	right part crest		4.02	3.97	right part crest
	4.5	1.2	right part crest		4.021	4.04	right part crest
	5	0.95	right part crest		4.5	3.95	mid crest
	5.5	0.694	right part crest		5	3.81	mid crest
Case 2	**water level [m+NAP]**	**β [-]**	**entrance point**		5.5	3.74	mid crest
	2	3.18	right edge crest	**Case 6**	**water level [m+NAP]**	**β [-]**	**entrance point**
	3	3.19	right-part crest		2	4.94	right part crest
	3.5	3.07	right-part crest		3	4.49	right part crest
	4.02	2.95	mid crest		3.5	4.25	right part crest
	4.021	3.01	mid crest		4	3.98	right part crest
	4.5	2.92	mid crest		4.53	3.67	right part crest
	5	2.85	mid crest		4.531	3.37	right edge crest
	5.5	2.81	mid crest		5	3.01	right edge crest
Case 3	**water level [m+NAP]**	**β [-]**	**entrance point**		5.5	2.77	right edge crest
	2	5.32	halfway inner slope	**Case 7**	**water level [m+NAP]**	**β [-]**	**entrance point**
	2.5	4.85	halfway inner slope		2	4.08	right edge crest
	3	3.82	halfway inner slope		2.5	4.31	right edge crest
	3.5	2.25	halfway inner slope		3	4.35	right part crest
	4.02	2.09	halfway inner slope		3.5	4.2	right part crest
	4.021	1.82	halfway inner slope		4	4.04	right part crest
	4.5	1.69	halfway inner slope		4.53	3.88	mid crest
	5	1.49	halfway inner slope		4.531	3.97	mid crest
	5.5	1.28	halfway inner slope		5	3.86	mid crest
Case 4	**water level [m+NAP]**	**β [-]**	**entrance point**		5.3	3.81	mid crest
	2	4.79	right part crest				
	3	4.4	right part crest				
	3.5	4.2	right part crest				
	4.02	3.93	right part crest				
	4.021	3.72	right part crest				
	4.5	3.43	right edge crest				
	5	3.04	right edge crest				
	5.5	2.7	right edge crest				

Test Case 1

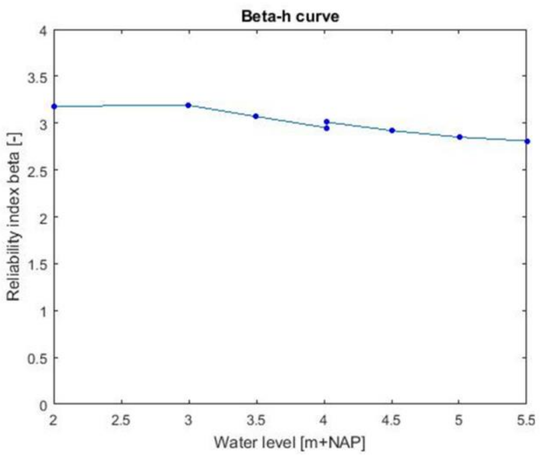
	Water level [m+NAP]							
	2	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	2.30	1.95	1.76	1.56	1.42	1.2	0.95	0.694
Failure probability	1.07	2.56	3.92	5.94	7.78	1.15	1.71	2.44
[-]	E-02	E-02	E-02	E-02	E-02	E-01	E-01	E-01
Influence factor (α^2) [%]								
λ_{in}	0	0.109	0.254	0.469	0.474	0.736	1.328	1.797
λ_{out}	0	0.075	0.175	0.322	0.326	0.505	0.838	1.295
Intrusion length	0	0.006	0.046	0.278	1.361	2.537	2.601	4.149
Yield (82.06)	0	0	0	0	0	0	0	0
Yield (139.12)	14.469	15.84	16.598	17.38	17.697	18.427	18.423	19.266
Yield(175)	2.929	3.088	3.163	3.233	3.227	3.251	4.04	4.082
φ (Sand dike)	0.374	0.351	0.338	0.323	0.294	0.271	0.242	0.21
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	73.355	71.774	70.745	69.42	68.26	66.126	64.48	61.407
m (Clay,B; Clay,N)	0.892	1	1.062	1.127	1.131	1.193	1.356	1.541
m_d	7.981	7.757	7.62	7.446	7.231	6.953	6.692	6.253

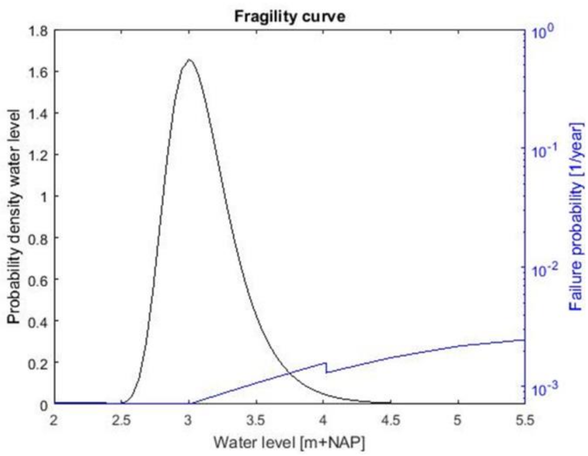




Test Case 2

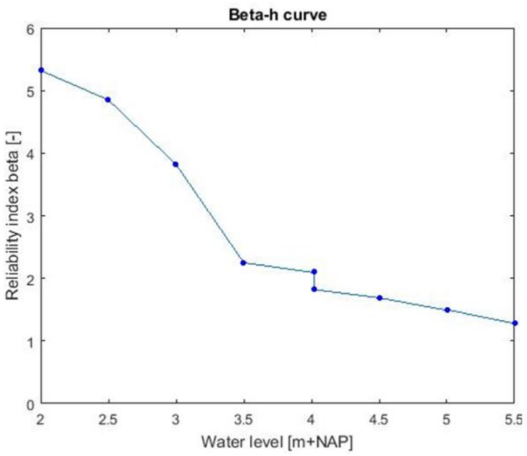
	Water level [m+NAP]							
	2	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	3.18	3.19	2	2.95	3.01	2.92	2.85	2.81
Failure probability	7.36	7.11	1.07	1.59	1.31	1.75	2.19	2.48
[-]	E-04	E-04	E-03	E-03	E-03	E-03	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.08	0.192	0.381	0.44	0.643	1.196	1.958
λ_{out}	0	0.049	0.119	0.241	0.265	0.415	0.776	1.255
Intrusion length	0.783	0.922	1.836	2.908	3.232	5.98	8.424	11.969
Yield Dike (82.06)	0	0	0	0	0	0	0	0
Yield Aquitard (82.06)	0	0	0	0	0	0	0	0
Yield Dike(139.12)	2.964	1.516	1.507	1.466	1.544	1.82	2.132	2.572
Yield Aquitard (139.12)	29.648	22.394	22.814	22.736	22.472	22.279	20.971	20.322
Yield(175)	4.04	5.749	5.774	5.205	5.154	5.045	5.787	5.437
φcs (Clay dike)	7.777	4.151	4.088	4.025	4.201	4.376	4.539	4.958
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	43.632	53.81	52.514	51.962	51.547	48.554	45.425	41.06
m (Clay Dike)	0.055	0.026	0.026	0.028	0.036	0.061	0.11	0.206
m (Clay,U; Clay,N)	0.413	0.552	0.618	0.635	0.653	0.706	0.9	1.005
m_d	10.69	10.749	10.512	10.413	10.457	10.123	9.738	9.257

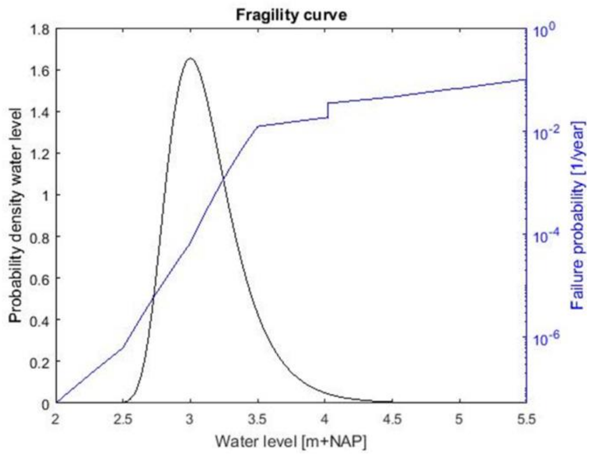




Test Case 3

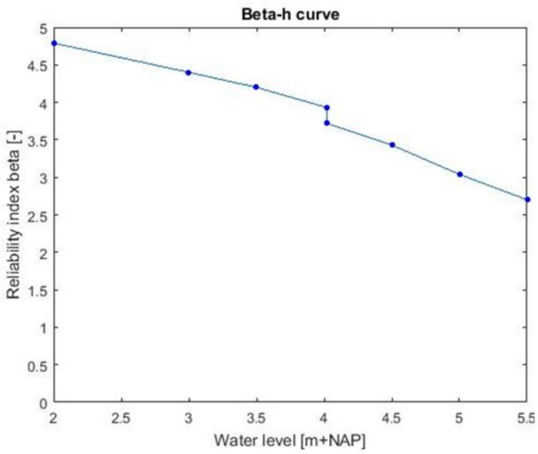
	Water level [m+NAP]								
	2	**2.5**	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis									
Reliability β [-]	5.32	5.06	3.82	2.25	2.09	1.82	1.69	1.49	1.28
Failure probability	5.19	2.10	6.67	1.22	1.83	3.43	4.55	6.81	1.00
[-]	E-08	E-07	E-05	E-02	E-02	E-02	E-02	E-02	E-01
Influence factor (α^2) [%]									
λ_{in}	0	0.191	10.182	0.016	0.027	0.002	0.002	0.003	0.004
λ_{out}	0	0.105	5.148	0.016	0.022	0.006	0.007	0.012	0.014
Intrusion length	0.786	5.553	1.316	3.668	7.449	7.082	6.681	7.378	8.164
Yield (82.06)	0	0	0	0	0	0	0	0	0
Yield (139.12)	23.765	22.902	27.712	37.28	47.139	48.484	51.433	54.482	55.632
Yield(175)	2.015	2.805	1.2	0.343	0	0	0	0	0
φ (Sand dike)	0.744	0.639	0.689	0.696	0.509	0.465	0.407	0.432	0.365
φ (Sand Pleistocene)	0	0	0.433	0.447	0	0	0	0	0
S (Clay,B; Clay,N)	58.564	54.07	36.882	39.788	35.976	35.332	33.428	30.073	28.644
m (Clay,B; Clay,N)	2.886	3.498	5.779	6.461	1.819	1.798	1.673	1.7	1.672
m_d	11.24	10.237	10.658	11.286	7.059	6.833	6.369	5.92	5.505

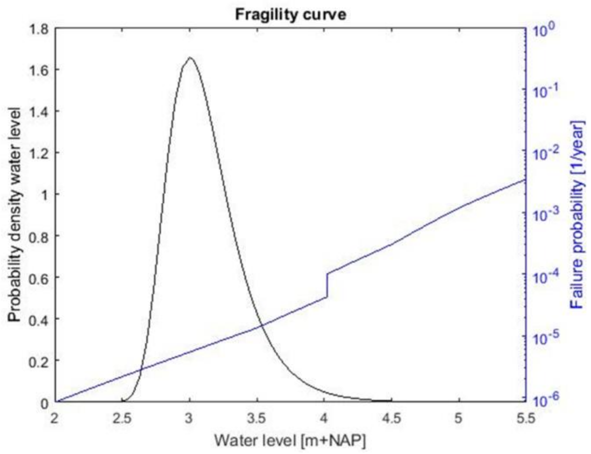




Test Case 4

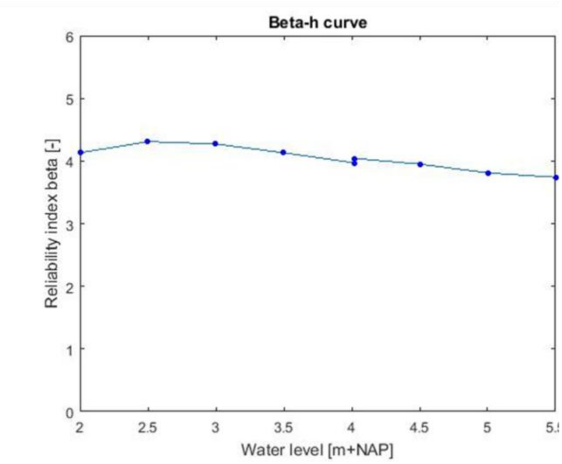
	Water level [m+NAP]							
	2	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis								
Reliability β [-]	4.79	4.4	4.2	3.93	3.72	3.43	3.04	2.7
Failure probability	8.34	5.41	1.33	4.25	9.96	3.02	1.18	3.47
[-]	E-07	E-06	E-05	E-05	E-05	E-04	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.185	0.437	0.849	0.89	1.078	2.621	2.235
λ_{out}	0	0.128	0.302	0.586	0.614	0.737	1.766	1.502
Intrusion length	0.352	0.008	0.106	0.634	4.559	3.038	3.26	7.112
Yield (82.06)	0	0	0	0	0	0	0	0
Yield (139.12)	14.292	15.574	16.252	17.088	17.322	19.525	19.818	21.726
Yield(175)	3.654	3.879	3.975	4.058	3.963	4.77	5.625	4.742
φ (Sand dike)	0.526	0.511	0.5	0.475	0.418	0.36	0.33	0.272
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	68.197	66.872	65.719	63.867	60.522	58.993	55.488	52.005
m (Clay,B; Clay,N)	0.982	1.098	1.172	1.269	1.243	1.481	1.701	1.761
m_d	11.996	11.746	11.537	11.174	10.469	10.018	9.391	8.645

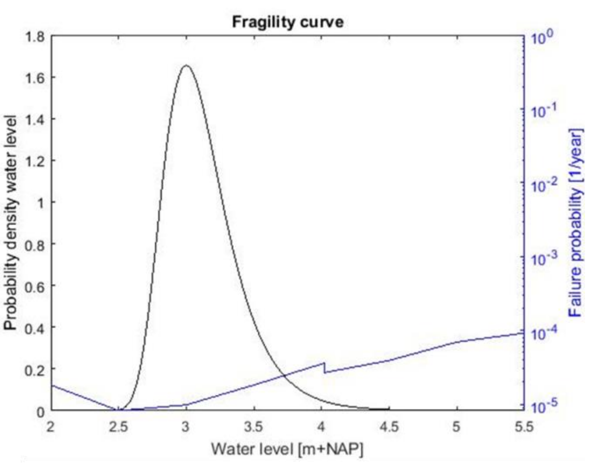




Test Case 5

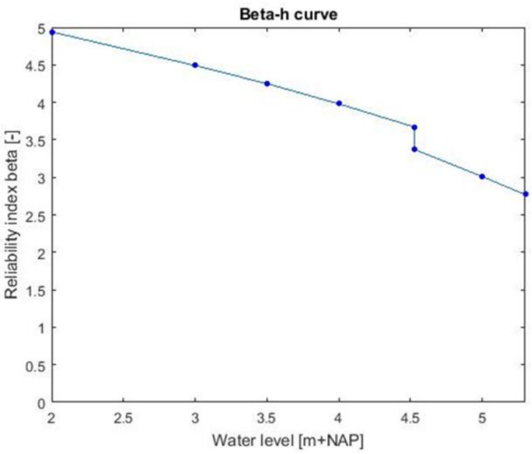
	Water level [m+NAP]								
	2	**2.5**	**3**	**3.5**	**4.02**	**4.021**	**4.5**	**5**	**5.5**
Failure analysis									
Reliability β [-]	4.13	4.31	4.27	4.13	3.97	4.04	3.95	3.81	3.74
Failure probability [-]	1.81 E-05	7.11 E-06	9.77 E-06	1.81 E-05	3.59 E-05	2.67 E-05	3.91 E-05	6.95 E-05	9.2 E-05
Influence factor (α^2) [%]									
λ_{in}	0.005	0	0.105	0.281	0.578	0.585	1.031	1.755	2.784
λ_{out}	0.005	0	0.065	0.18	0.379	0.384	0.704	1.194	1.807
Intrusion length	1.102	0.154	0.939	3.054	6.073	6.198	11.744	17.489	22.809
Yield Dike (82.06)	0	0	0	0	0	0	0	0	0
Yield Aquitard (82.06)	0	0	0	0	0	0	0	0	0
Yield Dike(139.12)	2.618	2.535	1.748	1.722	1.767	1.841	2.051	2.237	2.635
Yield Aquitard (139.12)	40.472	24.285	21.374	21.78	22.048	21.72	20.832	19.132	17.732
Yield(175)	2.772	5.773	6.074	6.103	6.072	6.008	5.26	5.724	5.152
φcs (Clay dike)	6.996	6.131	4.517	4.386	4.266	4.446	4.487	4.326	4.595
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	34.954	47.807	52.026	49.808	46.729	46.617	42.439	37.467	32.429
m (Clay Dike)	0.05	0.048	0.03	0.029	0.034	0.04	0.07	0.119	0.222
m (Clay,U; Clay,N)	0.539	0.488	0.491	0.524	0.555	0.554	0.505	0.63	0.715
m_d	10.488	12.778	12.63	12.133	11.499	11.606	10.876	9.926	9.12

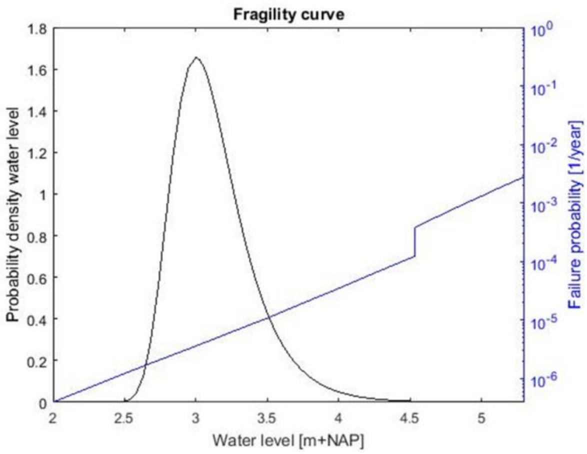




Test Case 6

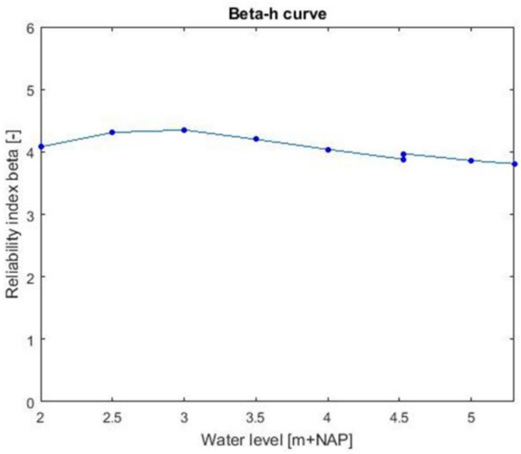
	Water level [m+NAP]							
	2	**3**	**3.5**	**4**	**4.53**	**4.531**	**5**	**5.3**
Failure analysis								
Reliability β [-]	4.94	4.49	4.25	3.98	3.67	3.37	3.01	2.77
Failure probability	3.91	3.56	1.07	3.45	1.21	3.76	1.31	2.80
[-]	E-07	E-06	E-05	E-05	E-04	E-04	E-03	E-03
Influence factor (α^2) [%]								
λ_{in}	0	0.186	0.442	0.864	1.499	1.457	2.385	2.505
λ_{out}	0	0.13	0.308	0.599	1.038	1.009	1.642	1.709
Intrusion length	0.004	0.572	0.071	0.585	1.945	9.249	2.288	17.003
Yield (83.59)	0	0	0	0	0	0	0	0
Yield (138.4)	14.098	15.203	16.059	17.301	18.184	18.09	21.049	18.371
Yield(170)	3.93	4.202	4.348	4.945	4.99	4.756	5.227	4.457
φ (Sand dike)	0.534	0.475	0.459	0.428	0.397	0.322	0.312	0.247
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0
S (Clay,B; Clay,N)	68.356	66.536	65.713	62.992	60.073	54.466	56.013	46.45
m (Clay,B; Clay,N)	1.031	1.13	1.215	1.418	1.551	1.442	1.691	1.516
m_d	12.046	11.566	11.384	10.868	10.322	9.209	9.393	7.741

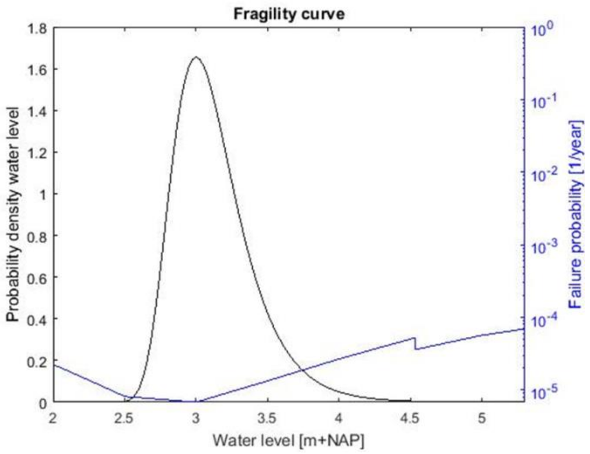




Test Case 7

	Water level [m+NAP]								
	2	**2.5**	**3**	**3.5**	**4**	**4.53**	**4.531**	**5**	**5.3**
Failure analysis									
Reliability β [-]	4.08	4.31	4.35	4.2	4.04	3.88	3.97	3.86	3.81
Failure probability	2.25	8.16	6.81	1.33	2.67	5.22	3.59	5.67	6.95
[-]	E-05	E-06	E-06	E-05	E-05	E-05	E-05	E-05	E-05
Influence factor (α^2) [%]									
λ_{in}	0	0	0.107	0.287	0.582	1.074	1.323	1.869	2.272
λ_{out}	0	0.001	0.068	0.188	0.389	0.743	0.915	1.291	1.558
Intrusion length	0.764	0.148	1.071	3.24	6.401	12.283	13.888	19.31	21.787
Yield Dike (82.06)	0	0	0	0	0	0	0	0	0
Yield Aquitard (82.06)	0	0	0	0	0	0	0	0	0
Yield Dike(139.12)	6.161	2.7	2.169	2.138	2.08	2.143	2.057	2.347	2.619
Yield Aquitard (139.12)	2.88	31.34	21.303	21.714	22.016	21.344	19.88	18.906	18.467
Yield(175)	6.292	4.766	6.271	6.304	6.281	5.517	6.465	6.043	5.598
φcs (Clay dike)	11.948	5.517	4.507	4.378	4.195	3.994	3.864	4.007	4.261
φ (Sand Pleistocene)	0	0	0	0	0	0	0	0	0
S (Clay,U; Clay,N)	54.282	43.323	51.335	49.061	46.039	41.775	40.688	35.983	33.429
m (Clay Dike)	0.657	0.575	0.521	0.553	0.569	0.522	0.585	0.636	0.669
m (Clay,U; Clay,N)	0.144	0.062	0.048	0.047	0.046	0.067	0.076	0.135	0.202
m_d	16.874	11.569	12.601	12.09	11.402	10.537	10.257	9.473	9.138





Appendix G. Failure probability water levels below foreshore taking into account residual width

The table below shows for all test cases the results of the analysis of scenarios 2 and 3. It shows the probabilities of all individual slides and the amount of foreshore that is lost due to the slide. Furthermore, it shows the reliabilities for both progressive failure (scenario 3) and a long sliding plane (scenario 2) leading to that amount of foreshore loss. Note that the lowest value of these reliabilities is used as resulting reliability for the corresponding step.

The foreshore loss uses the start of the foreshore (land side) as reference. Negative values in this column mean that slip surface does not cross the foreshore, but has an entrance point in the crest or the inner slope.

Note that all cases have a negligible failure probability for water levels below foreshore level; for all cases an individual slide with a probability lower than 6.0E-11 is found before reaching failure (less than 3m of remaining width).

Case 1	ID slide	β [-]	Pf [-]	foreshore loss [m]	β progressive failure [-]	β long slip surface [-]
	1	1.57	5.82E-02	-2	1.57	2.35
	2	2.87	2.05E-03	6	2.87	4.5
	3	4.33	7.46E-06	9	4.33	5.17
	4	4.86	5.87E-07	13.5	4.86	6.03
	5	4.53	2.95E-06	17.5	4.53	7.38
	6	4.99	3.02E-07	22.5	4.99	-
	7	5.11	1.61E-07	26.5	5.11	-
	8	5.29	6.12E-08	30.5	5.29	-
	9	5.37	3.94E-08	38	5.37	-
	10	5.84	2.61E-09	44.5	5.84	-
	11	5.68	6.73E-09	52	5.68	-
	12	6.15	3.87E-10	58	6.15	-
	13	5.85	2.46E-09	62.5	5.85	-
	14	6.29	1.59E-10	68.5	6.29	-
	15	6.32	1.31E-10	73.5	6.32	-
	16	6.2	2.82E-10	77	6.2	-
	17	7.47	4.01E-14	84	7.47	-
Case 2	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	2.95	1.59E-03	-4	2.95	3.22
	2	4.33	7.46E-06	0	4.4	4.33
	3	5.21	9.44E-08	3	5.52	5.21
	4	6.03	8.20E-10	7	6.44	6.03
	5	6.5	4.02E-11	9	-	6.5
Case 3	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	2.1	1.79E-02	-11.5	2.1	3.61
	2	5.45	2.52E-08	2.5	5.45	9.5
	3	6.62	1.80E-11	6	6.62	-
Case 4	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	3.93	4.25E-05	-1.5	3.93	5.01
	2	5.29	6.12E-08	3.5	5.29	6.68
	3	6.48	4.59E-11	7	6.48	-
Case 5	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	3.97	3.59E-05	-2.5	3.97	4.64
	2	4.53	2.95E-06	1.5	4.53	5.92
	3	6.48	4.59E-11	4.5	6.48	6.55
Case 6	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	3.67	1.21E-04	3	3.67	5.34
	2	4.93	4.11E-07	7	4.93	6.46
	3	5.14	1.37E-07	11.5	5.14	7.85
	4	6.61	1.92E-11	15	6.61	-
Case 7	**ID slide**	**β [-]**	**Pf [-]**	**foreshore loss [m]**	**β progressive failure [-]**	**β long slip surface [-]**
	1	3.88	5.22E-05	4.5	3.88	5.44
	2	5.98	1.12E-09	8	5.98	6.03
	3	6.48	4.59E-11	13.5	6.48	7.49

Appendix H. Critical state friction angle

This appendix gives insight in the way the critical state friction angle φ_{cs} of the clay above the phreatic line is determined. Like explained in Section 2.4, until recently, the strength of clay layers was calculated using the Mohr-Coulomb model, based on the cohesion and friction angle. However, following the WBI 2017 and recent developments, the strength of soil is now determined using the Critical State Soil Model (CSSM). In undrained conditions the SHANSEP model is used, based on the parameters S, m and OCR. For these parameters, (Rijkswaterstaat, 2016b) gives standard values. The part of a clay dike that is above the phreatic line, however, has to be calculated using drained analysis. In the CSSM, this drained analysis is not anymore based on cohesion c and friction angle φ, but only on a critical state friction angle φ_{cs}, taking no cohesion into account (c = 0) (see Section 2.4.6). For this new parameter φ_{cs} there are no standard values available. To determine values for $\varphi_{cs,}$ data from the national database are used (Tigchelaar & Daggenvoorde, 2017).

In the national database, the strength of different kinds of clay was examined. These kinds of clay were divided into independent classes, based on the volumetric weight and the parameter M. M is the slope of the critical state line in (q,p')-plane, following from triaxial tests. Figure H.1 shows a scatter plot of the values of M that were found, plotted against the volumetric weight of the clay. The black vertical dashed lines show the class boundaries.

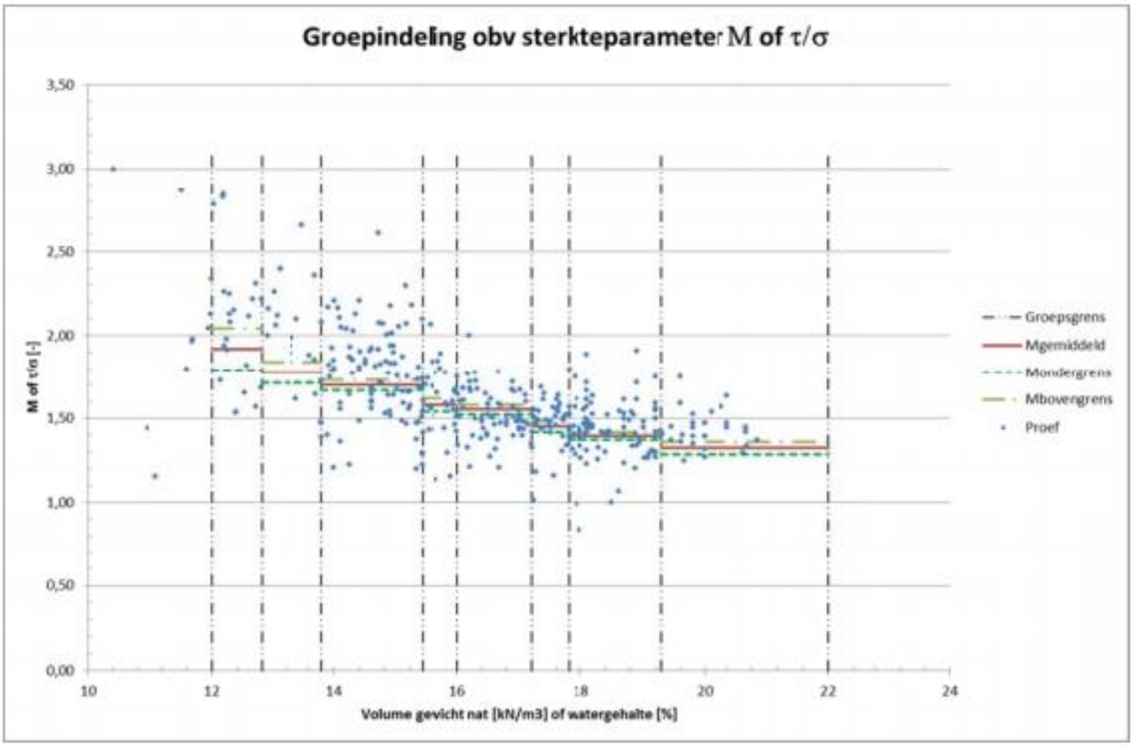


Figure H.1: Values of M for different volumetric weights of clay (Tigchelaar & Daggenvoorde, 2017)

(Wroth, 1984) shows there is a clear relation between M and φ_{cs}:

$$M = \frac{6 * \sin(\phi_{cs})}{3 - \sin(\phi_{cs})}$$

Using this relation, all found values of M in the class that corresponds to the clay in the dike (based on volumetric weight) are transferred into values of $\varphi_{cs.}$ On these values, a lognormal distribution is fitted, which can be used in the stability calculations.

Appendix I. Macro-stability in combination with wave overtopping

This appendix describes the assessment for the combination of macro-stability and wave overtopping. For this assessment, an extra assessment rule is proposed next to the normal macro-stability assessment, with a less strict required safety. This provides a nice analogy for a possible improvement of the macro-stability assessment for dikes with high foreshores.

In the new safety standards, it is stated that the critical overtopping discharge for dikes is higher than was previously expected: around 5 to 10l/s/m. However, one can imagine that an amount of water that goes over the dike, has an influence on macro-stability. After all, water going over the dike will infiltrate, at least partly, into the dike body, which will cause the phreatic line to rise. This has a negative influence on the dike in two ways:

- Due to the rise of the phreatic line, there is a higher probability of macro-instability with a deep slip surface, causing a large part of the dike to slide away.
- Due to the rise of the phreatic line, there is a higher probability of a macro-stability with a shallow slip surface, washing away the cover, after which the dike core is not protected anymore and will further erode due to the overtopping/overflowing water.

To account for this negative influence, one has to perform another macro-stability safety assessment, besides the standard assessment. After all, in the standard assessment overtopping discharge was not taken into account. In the extra assessment overtopping of water should be taken into account. However, for this assessment the requirements are less strict, as described in (de Visser, 2016)

This extra assessment is performed using a water level corresponding to an overtopping discharge of 1l/s/m. This is higher than the design water level discussed before, because the design water level is below the crest of the dike and this will for a river dike not lead to a substantial amount of overtopping.

The target probability for the extra assessment is determined by taking the original target probability and dividing it by the probability of occurrence of an overtopping discharge of 1l/s/m. For this target probability given overtopping, a target reliability given overtopping can be determined. The required damage factor to reach this target reliability can be determined as follows (de Visser, 2016):

$$\begin{aligned} \gamma_n &= 0{,}042(\beta_{T,Q} - 2{,}3) + 1, \quad \text{for } \beta_{T,Q} < 2.3 \\ \gamma_n &= 0{,}219(\beta_{T,Q} - 2{,}3) + 1, \quad \text{for } \beta_{T,Q} \geq 2.3 \end{aligned} \tag{I.1}$$

In short, the approach is such, that an extra assessment rule is created, for which a heavier loading is taken into account, but which has to fulfil less strict requirements, as the heavier loading has a lower probability of occurrence.

Appendix J. Example Calibration

This Appendix gives an example of the calibration for the conditional failure probability given the water level is above foreshore ($P_{f|h>h^*}$), which is used in the semi-probabilistic method using two separate assessment rules. This calibration is performed for the test cases used in this Thesis. Note that for all test cases in this Thesis the contribution of the water levels below foreshore is negligible. As described in Section 8.3, the failure probability can then be described as:

$$P_f = P_{f|h>h^*} * P_{h>h^*}$$

The safety requirement can then be written as:

$$P_{f|h>h^*} \leq \frac{P_{f,target}}{P_{h>h^*}}$$

The calibration is needed to transform the result of the semi-probabilistic method, γ_n (= FoS/γ_d), into $P_{f|h>h^*}$.

Of all the test cases of this Thesis, the failure probabilities (P_f) and the probabilities of flooding of the foreshore are known ($P_{h>h^*}$), which means that the conditional probability $P_{f|h>h^*}$ can be determined. This probability is transformed into a conditional reliability ($\beta_{|h>h^*}$). The relation between $\beta_{|h>h^*}$ and γ_n will be calibrated.

Note: In this example, for the values of γ_n the standard semi-probabilistic approach is followed, using a design water level with an exceedance frequency equal to the safety standard. However, it is possible to make other choices for the design water level (for example a fixed distance of x meter above the foreshore). The calibration then has to be performed using this choice for the design water level.

Test Case	P_f	β	$P_{h>h^*}$	$P_{f\|h>h^*}$	$\beta_{\|h>h^*}$	γ_n
1	9.55E-04	3.104	1.00E-02	9.55E-02	1.308	0.782
2	1.51E-05	4.172	1.00E-02	1.51E-03	2.966	0.941
3	4.00E-04	3.353	1.00E-02	4.00E-02	1.751	0.836
4	2.08E-06	4.603	1.00E-02	2.08E-04	3.530	1.005
5	3.30E-07	4.973	1.00E-02	3.30E-05	3.990	1.042
6	7.11E-07	4.822	1.00E-03	7.11E-04	3.190	1.019
7	4.34E-08	5.352	1.00E-03	4.34E-05	3.925	1.049

Table J.1: Results of the test cases needed for the calibration (α=4.5)

Test Case	P_f	β	$P_{h>h*}$	$P_{f\|h>h*}$	$β_{\|h>h*}$	$γ_n$
1	4.15E-05	3.936	4.76E-04	8.72E-02	1.358	0.823
2	6.73E-07	4.833	4.76E-04	1.41E-03	2.986	0.952
3	1.77E-05	4.136	4.76E-04	3.72E-02	1.784	0.870
4	6.90E-08	5.268	4.76E-04	1.45E-04	3.624	1.055
5	1.42E-08	5.551	4.76E-04	2.98E-05	4.014	1.053
6	5.91E-09	5.702	1.04E-05	5.68E-04	3.254	1.084
7	4.24E-10	6.136	1.04E-05	4.08E-05	3.940	1.057

Table J.2: Results of the test cases needed for the calibration (α=7.5)

Figure J.1 shows the scatter plot of the safety factor ($γ_n$) against the conditional reliability given the water level is above foreshore ($β_{|h>h*}$). The calibrated relation is fitted at the 20%-quantile of the reliabilities, like in the original calibration study (Section 2.4.5). The relation is as follows:

$$\gamma_n = 0.107 * \beta_{|h>h^*} + 0.678$$

Based on this, the following relation between the FoS and $P_{f|h>h^*}$ is found:

$$P_{f|h>h^*} = \phi\left(-\frac{\left(\frac{FoS}{\gamma_d}\right)-0.678}{0.107}\right)$$

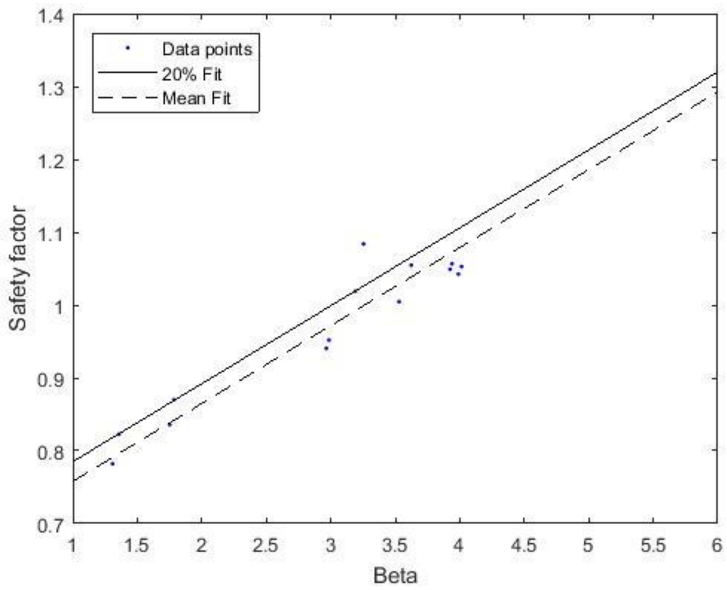


Figure J.1: Scatter plot calibration

Note: In the example above the contribution of the water levels below foreshore to the total failure probability is negligible for all test cases. However, in reality this may not be the case. One then has to perform two calibrations, one for the conditional reliability given the water level is below foreshore and one given the water level is above foreshore. For the first calibration, one needs to set up an alternative semi-probabilistic approach to determine $γ_n$, using a (design) water level that is below foreshore level.