Probabilistic Design of a Rubble Mound Breakwater

P.S. Everts



Challenge the future

Probabilistic design of a rubble mound breakwater

MASTER OF SCIENCE THESIS

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

P.S. Everts

June 16, 2016

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Photo cover image:

Wave breaking on rock. Online image. Source: https://www.pexels.com June 2016





DELFT UNIVERSITY OF TECHNOLOGY DEPARTMENT OF HYDRAULIC ENGINEERING

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

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by

P. S. EVERTS

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Graduation committee:

prof. dr.ir. S.N. Jonkman Delft University of Technology Hydraulic Engineering Hydraulic structures and flood risk

ir. H.J. Verhagen Delft University of Technology Hydraulic Engineering Coastal engineering

ing. C. Kuiper

Delft University of Technology Hydraulic Engineering Coastal engineering

ir. R. Bouw

Witteveen+Bos Raadgevende ingenieurs Kust- en rivierwaterbouw Kusten, rivieren en landaanwinning

Abstract

All over the world rubble mound breakwaters are built to protect harbours, shorelines, and other vulnerable coastal areas against wave action and currents. Most of the designs for these structures use so called deterministic or semi-probabilistic design methods (level I). With these methods insight in the uncertainties, and consequently the actual failure probability and behaviour of the structure, is lacking. Moreover, less is known about the physical and mathematical relation between the variables and design formulas. The uncertainties in the variables and design formulae in a semi-probabilistic method are taken into account by partial safety factors. This could lead to an overly conservative design.

By applying a probabilistic calculation (level II and III) insight is obtained in the relations between variables, the failure behaviour and the probability of failure of the structure. This information can explain why certain structures, which are designed with an semi-probabilistic method, fail even though the design conditions are not reached or in most cases survive above the design conditions. Information on the actual failure behaviour and probability is desired to make a more reliability design and economic optimization.

Despite these benefits probabilistic design methods offer, it is not often applied in daily engineering practice. Multiple studies show the feasibility of designing a rubble mound breakwaters with a probabilistic design method in theory. However in practice only few rubble mound breakwaters are designed with a probabilistic design method.

This research investigates how a probabilistic design of a rubble mound breakwater can be made in practice and provides some guidelines when a probabilistic design can be considered. The project Taman is used as a case and from this project a rubble mound breakwater is selected for the fully probabilistic calculation. Simplifications are made regarding the applied mathematical models¹ and only four failure mechanisms related to the Ultimate Limit State (ULS) are examined. The four main failure mechanisms are: seaside and rear-side armour stability, toe stability and macro stability. With these simplifications a clear and thoroughly insight is gained in the probabilistic design process of a rubble mound breakwater without loosing track of the actual objective of this study.

The fully probabilistic calculation is made with a level III probabilistic design method by applying a Monte Carlo simulation. The results show that this method is a good way to take into account the occurring statistical and physical correlation. Furthermore the Monte Carlo analysis gives a good insight in the most dominant failure mechanisms and in the governing failure situations for each mechanism.

The results of the fully probabilistic calculation show that making a semi-probabilistic design based on the design rules in The Rock Manual [2007] results in a conservative design ($P_{f,sys,t_L} = 0.5\%$). One optimization step is made for the simplified case in this research by applying lower stone classes for the four considered failure mechanisms. This results in failure probability ($P_{f,sys,t_L} = 11.25\%$) which is still lower than the in general allowable probability of failure ($P_{f,sys,t_L} = 15\%$).

Although not all failure mechanisms for the ULS are taken into account, this study proves that a fully probabilistic calculation results in a more optimized design compared to a semi-probabilistic design method for the examined case. In conclusion a fully probabilistic calculation for a rubble mound breakwater is possible in practise. The results show that it is certainly beneficial to apply a fully probabilistic calculation (Level III) compared to a semi-probabilistic (level I) calculation.

¹Models which describe the hydrodynamic processes (SWAN and Delft3D) and geotechnical stability (D-Geo Stability)

However, not in all cases it is possible to apply a fully probabilistic calculation and a couple of aspects should be checked before starting the calculation:

- Statistical and physical correlation of the main variables have to be known
- Sufficient reliable data for boundary conditions should be available
- Applied mathematical models should be incorporated in the fully probabilistic calculation
- Design requirements don't follow directly from the standards and therefore must be agreed with the client

This research shows that neglecting the statistical and part of the physical correlation results in an over dimensioned design for a rubble mound breakwater. A fully probabilistic (level III) design method with a Monte Carlo simulation proves to be a good way to included these correlations in the fully probabilistic design process.

Sufficient reliable data for the boundary conditions should be available at the project location to make a fully probabilistic design method feasible. Large uncertainties in the boundary conditions result in a high failure probability of the rubble mound breakwater. To determine for which boundary conditions sufficient reliable data has to be known, a FORM analysis (level II) could be applied. This analysis gives α -values which indicates the influence of each input variable on the failure probability of the rubble mound breakwater. For example in the examined case the uncertainties in the significant wave height have a large contribution to the variation in the probability of failure. The results show that a fully probabilistic calculation is not feasible in this case when the uncertainties in H_s have a standard deviation (σ) of 25% or more.

In the semi-probabilistic design the hydraulic boundary conditions are determined via the models SWAN and Delft3D. Additionally the model D-Geo Stability is used for the semi-probabilistic design to check the geotechnical failure mechanisms. In this research is concluded that all hydraulic, geotechnical and geometric conditions need to be carried out in fully probabilistic way. For the fully probabilistic calculation of the simplified case simplifications are made for the mathematical models. These simplifications give a good approximation of the models in this study. However, the mathematical models have to be (in some way) integrated in the statistical analysis to make a fully probabilistic design.

Preface

This report is the graduation thesis of Siemen Everts and performed as conclusion of my master Hydraulic Engineering at the Faculty of Civil Engineering and Geosciences (CEG) at Delft University of Technology. With this thesis the degree of Master of Science at Delft University of Technology is obtained.

My interest in Hydraulic Engineering already started at a young age when I made a visit with my dad to the Oosterscheldekering. Little did I know that this was one of the first large structures in the Netherlands which was designed with a probabilistic design method. Now, several years, later Witteveen+Bos gave me the opportunity to make a probabilistic design myself. A lot of people have supported me during this process, for which I am grateful.

First of all I would like to thank my graduation committee members, Prof. dr. ir. Bas Jonkman, ir. Henk Jan Verhagen, ing. Coen Kuiper and ir. Ruud Bouw for their time to review the submitted content and constructive feedback. Especially Ruud Bouw and Coen Kuiper, my direct and daily supervisors, for taking the initiative for this graduation topic and their support until the very last moment of this thesis.

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Besides the people who helped me during my thesis, I would also like to thank my friends and fellow students for the great years at Delft University of Technology.

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Delft, June 16, 2016 P.S. Everts

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Introduction

The topic of this research is the probabilistic design method of rubble mound breakwaters. First a theoretical analysis is made and afterwards a case is examined to investigated the feasibility of this method in practice. In this chapter the problem description, problem definition and research objectives are described. At last a reading guide is given which explains the structure of the report.

1.1. Problem description

All over the world breakwaters are built to protect harbours, shorelines, and other vulnerable coastal areas against wave action and currents. Most of the designs for these structures are based on a semi-probabilistic design method. A design storm is chosen and partial safety factors are applied for the load and the strength variables to cover the unknown uncertainties in these variables. These partial safety factors are provided via standards and codes, for example in PIANC [1992].

The semi-probabilistic design method is characterized as a level I method. With this method insight in the uncertainties and consequently the actual failure probability and behaviour of the structure is lacking. Moreover, less is known about the physical and mathematical relation between the variables and design formulas. Since the applied semi-probabilistic method uses general partial safety factors the designed structure, could be too conservative [Verhagen, 2003].

Other approaches can be applied to calculate the reliability of a structure by taking in account the uncertainties instead of applying safety factors. Level II methods assess uncertainties through the mean and the standard deviations of the basic variables; the probability of failure is approximated via this method. Level III methods quantifies uncertainties by the joint probability function and determines the exact probability of failure. The last method is the risk based (Level IV) method which also takes in account the consequences of failure, and uses the risk to quantify the reliability. [Jonkman et al., 2015]

• Level 0 = Deterministic.

• Level I = Semi-probabilistic

- Level III = Fully probabilistic
- Level IV = Risk based
- Level II = Probabilistic with approximations

These methods can be used to determine the total reliability of the structure. In order to do so, each failure mechanism of the breakwater should be investigated separately. As shown by several studies [Van der Meer, 1988; PIANC, 1992; Plate, 1995] calculating the failure probability of a single failure mechanisms by means of a probabilistic (level II or III) design method is not the issue. The master thesis of Plate [1995] shows the feasibility of calculating the failure probability for different failure mechanisms of a rubble mound breakwater in practice. The reliability of the structure cannot be determined exactly due to the lack of insight in the correlation between different aspects in the probabilistic design process.

The Probabilistic Design Tools for Vertical Breakwaters (PROVERBS) [Allsop et al., 1999] describes the probabilistic design process thoroughly. The study provides a framework on how to deal with all relevant physical processes and uncertainties associated with coastal structures. Two methods are presented which explains how to include the probabilistic analysis into this design process. The first method is to find the optimal level of safety including a cost optimisation procedure. As second method a partial safety factor system (PSFS) is develop for vertical breakwaters and this is verified for several existing structures. In this research also valuable information is provided on how to determine the target failure probability of a breakwater. Castillo et al. [2004] presented an alternative method based on the research of Allsop et al. [1999] that allows controlling safety factors and failure probabilities with respect to different failure mechanisms. The applicability of this method is illustrated by application to the design of a rubble mound breakwater and shows the feasibility of this method.

The incentive to design a breakwater with a probabilistic design method is to gain insight in the relations between variables, the failure behaviour and the probability of failure of the structure. This information can explain why certain structures, which are designed with an semi-probabilistic method, fail even though the design conditions are not reached or in other cases survive above the design conditions [Burcharth, 1987; Maddrell, 2005]. Information on the actual failure behaviour and probability is desired to make a more reliability design and economic optimization. This can possibly be achieved via a probabilistic design method.

In the recent years more studies showed the possibility of using a probabilistic design method for optimizing the design of a rubble mound breakwaters design. Castillo et al. [2006] proposed an optimal engineering design method for composite breakwaters to minimize the initial and construction costs. Hoby et al. [2015] improved this optimization and applied the method to a rubble mound breakwater. Even though there are still issues that have to be solved in the probabilistic design process, such as the physical correlation [Campos et al., 2011], different studies have shown the benefits and feasibility of a probabilistic design method for rubble mound breakwaters in practice[Dai Viet et al., 2008a,b].

In the The Netherlands the conditions of primary flood defences will be designed with probabilistic calculation methods from January 2017 due to adjustments of the law (OI2014 [Rijkswaterstaat, 2014b]). The problem of correlation is partly overcome by assigning failure spaces for each failure mechanisms. In the coming years new standards and guidelines will be developed and present in the instrumentation WTI2017 [Rijkswaterstaat, 2014a].

In conclusion multiple studies have shown the feasibility of designing a rubble mound breakwaters with a probabilistic design method in theory. As seen in The Netherlands a shift is going to be made in the design process of levees from a semi-probabilistic method to a probabilistic method. However in practice only few breakwaters are designed with a probabilistic design method. Recently in Spain the secondary breakwater in the new harbour basin of the outer port of La Coruña [Maciñeira, 2016] is designed with a fully probabilistic design method. The main problem encountered during this design process was the unknown physical correlation between different failure mechanisms.

A probabilistic instead of a semi-probabilistic design method gives more insight in the actual failure behaviour and probability of failure of a rubble mound breakwater. This insight can be used to consider the reliability for each failure mechanism separately, efficiently invest time and money to reduce the uncertainties in the design and optimize the life cycle cost of the breakwater. Despite these benefits a probabilistic design methods offer it is not often applied in daily engineering practice. The reason is the lack of information regarding the following aspects of the full probabilistic method when applied in practice, which are elaborated in the next section:

- Design requirements (see chapter 7)
- Boundary conditions (see chapter 8)
- Statistical and physical correlation (see chapter 10)

1.2. Problem definition

The following problem definition is formulated for this research:

Insight in the physical failure behaviour and probability of failure of a rubble mound breakwater is lacking and can be obtained by applying a probabilistic design method (Level II and III). These methods are not often applied due to the lack of knowledge on how to determine the design requirements and boundary conditions for the probabilistic design process, in addition it is unknown how to deal with the statistical and physical correlation in this process.

Clarification of mentioned aspects:

- **Design requirements** To carry out a thorough probabilistic analysis the design requirements are one of the key elements. The design requirements for the probabilistic design method (e.g. allowable failure probability of the structure (P_{f,sys,t_L}) differ from the semi-probabilistic method (e.g. design storm) which is usually applied for the design of a rubble mound breakwater. Due to the fact that a probabilistic method is not often applied realistic design requirements should be examined and determined.
- **Boundary conditions** For a probabilistic design method the hydraulic, geotechnical and geometric boundary conditions have to be determined accurately. Especially determining the reliability and quantification of the uncertainties is crucial.
- **Statitical correlation** Describing reality with a model requires gathering, processing and interpreting a lot of data. Statistical correlation describes the dependency between variables and datasets, which is the result of common usage or derivation of the same basic principles.
- **Physical correlation** A rubble mound breakwater consists of multiple elements. All these components are subject to changes and influence each other. For example as the roughness of the armour decreases, wave overtopping could increase. These processes are defined as physical correlation.

1.3. Research objectives

The objective of this research is:

Investigate how a probabilistic design of a rubble mound breakwater can be made.

To achieve this objective several research questions are defined:

- · How to describe the probabilistic design method in a process diagram?
- How to determine the required boundary conditions for the probabilistic design of a rubble mound breakwater?
- How to define the design requirements for the fully probabilistic design method?
- Which variables have the most influence and are the most important for the fully probabilistic design?
- What is the influence of uncertainties in the basic variables on the failure probability?
- In which aspects in the design process do statistical and physical correlation play a role?
- How to determine the total probability of failure of the structure?

Ι

Project background

 \sum

Project phase of interest

The research which is described in this masters thesis focusses on the design requirements, final design and makes use of information form the preliminary design. In this chapter the position of this research in the entire design process is described.

2.1. General project cycle

The entire lifetime of a rubble mound breakwater from the initiative and feasibility study till demolition is captured in the construction cycle and described with five different phases [Hertogh and Bosch-Rekveldt, 2015].

- Initial phase
- Design phase
- Realisation phase
- Use phase
- Demolition and recycling phase

In the initial phase the feasibility study is executed and the design requirements are prepared. In the next phase, the design phase, the complete design of the structure is made. When the design is finalized the structure is build, which is the realisation phase. Once the structure is build it can be used and often maintenance is required. At last the structure is demolished and if possible recycled.

2.2. Phase of interest

As described in chapter 1 this study focusses on the probabilist design method for a rubble mound breakwater. During this research only the initial and the design phases are considered. From the initial phase the design requirements are examined for the semi-probabilistic design method and the probabilistic design method. The design requirements differ for both methods and have a large impact on the design since the proposed design should meet these requirements.

Additionally the design process for a probabilistic design is examined. The design process, which is part of the design phase, is subdivided in four different design phases. The design starts with a *sketch of the design* subsequently the *preliminary design* is made from there on the *final design* is created and finally the *detailed design* is made [Hertogh and Bosch-Rekveldt, 2015].

To make a probabilistic design a preliminary design is required as first estimate and guidance during the design process. This preliminary design is made on the basis of a semi-probabilistic design method. The final design is made with the probabilistic design method. So from the design phase the preliminary and the final design are investigated for the probabilistic design process of a rubble mound breakwater.

3

General description breakwaters

In this chapter a system analysis is made for a breakwater and the different types of breakwaters are described. In this research only a rubble mound breakwater is examined in further detail and the corresponding fault tree with all the failure mechanisms is given.

3.1. System analysis

An breakwater can have several functions. A very common objective is protection of a certain area against wave attack. For example detached breakwaters to protect a sandy coast from eroding or protection of a harbour basin against wave attack. This research focus on the latter.

The objective of the considered breakwater is to keep the downtime of the port due to wave attack to a requested minimum. To fulfil this goal the main function of the breakwater is defined as [Allsop et al., 1999]:

"Protection of the harbour basin against unacceptable wave action".

The minimum requested downtime and the definition of unacceptable wave action depends on the design requirements of the harbour.

Two different situations can be distinguished which both lead to unacceptable wave action in the port basin:

- The situation where the breakwater is not damaged however the wave action is too severe due to for example overtopping of waves. The breakwater is not damaged and remains stable however the design requirement is not met since there is too much wave energy entering the harbour basin. This state is distinguished as the *Serviceable Limit State* (SLS).
- Additionally there is the state where the breakwater is in fact damaged and looses its structural integrity. Often this damage leads to insufficient protection of the harbour basin against waves action. This state is defined as the *Ultimate Limit State* (ULS).

The definition of 'damage' makes the distinction between the two different states. Therefore the damage level of the breakwater needs to be defined accurately. The acceptable level of damage depends on the desires of a client and the capabilities to repair the structure in a acceptable timespan. Since the acceptable level of damage depends strongly on the examined breakwater, no description can be given in general. In chapter 7 the design requirements are examined and defined for the semi-probabilistic and probabilistic design method.

One should keep in mind that these limit states are related to the port downtime which is in this case the top event. Besides the ULS and SLS for the breakwater, other limit states for different structures which contributed to the total port down are present. This is not taken into account in this research. However it is important to realize that with a minimum chance of reaching the ULS and SLS of the breakwater the port downtime could still be significant due to other events (e.g. obstruction of the entrance channel).

On overview of these limit states of the breakwater in relation to the port downtime is given in figure 3.1



Figure 3.1: Fault tree entire harbour based on Dai Viet et al. [2008b]. (Fault tree A see appendix A and figure 3.7. Fault tree B see figure 3.8).

3.2. Breakwater types

Different types of breakwaters are used to protect harbour basins against unacceptable wave action. Breakwaters can be subdivide into categories based on their structural features [d' Angremond et al., 2012]:

- Mound types: Large heaps of loose elements
- Monolithic types: Cross-section acts as a solid block
- Composite types: Combines a monolithic element with a low-crest mound type
- Unconventional types: E.g. floating, pile and pneumatic breakwaters

In this research only the mound type breakwater is examined, for more information regarding the other types is referred to d' Angremond et al. [2012]. The mound type breakwater can occur in different forms, see figure 3.2. A crucial difference is the stability of the breakwater. On the hand are for example beaches of which the profile is continuously changing and on the other hand a statistically stable breakwater. Somewhere in between there is the berm breakwater which has a dynamically stable profile. Additionally selection is made based on the crest above still water level. Mound breakwater with a crest below still water levels are referred to as submerged breakwaters, a special type is the reef breakwater which has a reshaping mound. During this research only the conventional rubble mound breakwater is examined.



Figure 3.2: Mound breakwater types[d' Angremond et al., 2012]

3.3. Components

The assumed rubble-mound breakwater in this research consists of six different a components, see figure 3.3



Figure 3.3: Typical cross-section of conventional rubble-mound breakwater

The *core* of the rubble-mound breakwater is placed on the *subsoil*. On top of the core most of the time a *filter layer* is placed, this depends on the grading of the layer on top of it and the governing filter rules [The Rock Manual, 2007]. The top layer is the *armour layer* which is exposed severely to wave action. Furthermore a *toe* is placed to increase the stability of the breakwater and the armour layer. At last often a *crest element* is placed on top of the breakwater to reduce overtopping and increase the wave attack on the rear-side of the breakwater.

3.4. Failure mechanisms

To make a proper design which meets all the design requirements all the possible failure mechanism should be known. In figure 3.4 an overview is given the failure mechanisms which can occur for a rubble-mound breakwater according to Burcharth and Liu [1995].



Figure 3.4: Failure mechanisms of a rubble-mound breakwater according to Burcharth and Liu [1995]

3.4.1. Series or Parallel system

Failure mechanisms are either related in series or in parallel [PIANC, 1992], depending on the top event.



Figure 3.5: Series system

In a series system if any of the failure mechanisms, M = 1, 2, ..., n, fails it results in failure of the top event, see figure 3.5. In a fault tree a series system is pictured with an or-gate.

In a parallel system failure of the top event only occurs when all the failure mechanisms (M = 1, 2, ..., n) fail, see figure 3.6. A parallel system is pictured with an and-gate in a fault tree.



Figure 3.6: Parallel system

3.4.2. Fault tree

In chapter 3.1 a distinction is made between the Ultimate and the Serviceable Limit State to describe the top event of the fault tree (see figure 3.1).

The failure mechanisms pictured in figure 3.4 can be subdivided into the ULS or the SLS based on their impact on the breakwater. For example overtopping contributes to the SLS since it causes unacceptable wave action in the harbour basins without damaging the breakwater. On the other hand fracture of the armour results in damage to the breakwater and it looses its structural integrity. This is referred to as the ULS.

For each limit state a separate failure tree is made. The fault tree for the ULS is constructed as follows. The top is the ULS which corresponds with the first level, the second level are the type of failure mechanism (e.g. hydraulic or geotechnical). The third level is the failure mechanism as pictured in 3.4 and the last level corresponds with the different components as described in section 3.3. Besides the types hydraulic and geotechnical some special types of failure mechanisms could be present such as ice loads, earthquakes and ship collisions.

Furthermore the failure mechanisms and the components are related in series or parallel as described in subsection 3.4.1. In the ULS all the components are related in series. If one of the components fails the ULS is reached and there is port downtime due to repair and unacceptable wave action in the harbour basin. The special cases are also in series. Besides the components the failure mechanisms concerning a component are also all related in series. In conclusion the whole system is in series. Further explanation on the relation and interaction between failure mechanisms is given in chapter 10. The total fault tree for the ULS is too large to display here so only part of the fault tree is given in figure 3.7. The total fault tree is shown in appendix A. The fault trees for the ULS is part of the fault tree for the entire harbour given in figure 3.1 as indicated by *Fault tree A*. [d' Angremond et al., 2012; Plate, 1995; Burcharth and Liu, 1995].



Figure 3.7: Part of the fault tree for the Ultimate Limit State of a rubble-mound breakwater

The fault tree for the SLS is smaller than the fault tree of the ULS. Only few failure mechanisms cause unacceptable wave action in the port basins without affecting the structural integrity of the breakwater. The fault tree is displayed in figure 3.8 and indicated by *Fault tree B* in figure 3.1.



Figure 3.8: Fault tree of the Serviceable Limit State of a rubble mound breakwater

4

Reliability of an element

The reliability of the designed structure has to be determined to check if the structure meets the design requirements. To check the reliability of the total structure the reliability of all the different components should be examined. In this section the reliability of a single element is treated to discuss the different calculation methods. The combination of multiple elements and the resulting correlation is discussed in chapter 10.

In this research a level I method is used for the preliminary design as a start for the probabilistic design which is examined by a level II and III method. The level II method is used to examine the sensitivity and influence of different variables on the probability of failure of the structure. The exact failure probability is determined by level III method with a Monte Carlo simulation. In the following sections these reliability methods are explained in detail and an example is given to illustrate the possibilities.

4.1. Reliability function

The reliability of an element depends on the margin between the resistance and the solicitation. The reliability function describes this relation between the strength (i.e. resistance [R]) and the load (i.e. solicitation [S]) of an element.

The strength and load effects are often functions of several stochastic variables:

$$R = R(X_1, X_2, \dots X_m)$$
(4.1)

$$S = S(X_{m+1}, X_{m+2}, \dots X_n)$$
(4.2)

The reliability function (Z) expresses the margin between the resistance (R) and the solicitation (S). The value of the reliability functions (Z) describes the state of the element.

$$Z = R - S = Z(X_1, X_2, ... X_n)$$

$$Z < 0 \rightarrow failure$$

$$Z = 0 \rightarrow limit state$$

$$Z > 0 \rightarrow no failure$$
(4.3)

Now the different states are known, the reliability can be quantified. The reliability is often expressed as the probability of proper function i.e. the probability P(Z > 0). This can be expressed in terms of the probability of failure (P_f).

$$P(Z \ge 0) = 1 - P_f \tag{4.4}$$

The probability of failure is defined as follows:

$$P_f = P(Z < 0) = P(S > R)$$
(4.5)

Given the equations 4.3 and 4.5 and the fact that the strength and load effects are independent functions of several stochastic variables, the failure probability becomes:

$$P_f = \iint \int_{z<0} \dots \int f_R(x_1, x_2, \dots, x_m) f_S(x_{m+1}, x_{m+2}, \dots, x_n) dx_1 dx_2 \dots dx_n$$
(4.6)

In figure 4.1 the reliability function in a RS-plane is shown and the joint probability density function is displayed. The failure probability is the volume of the joint probability density function in the unsafe region (Z < 0), indicated by the hatched area.



Figure 4.1: Reliability function in R-S plane with probability mountain [Schiereck, 2001]

To determine the failure probability as indicated in figure 4.1, several reliability methods can be used. These methods are described in the following sections.

4.2. Reliability methods

In this study only the level 0 till III reliability methods are examined. Level IV methods also examines the consequences of failure and the resulting risk. This research focusses on determining the failure probability of a rubble mound breakwater and not quantifying the consequences, so level IV methods are not part of this research.

- Level 0 = deterministic
- Level I = semi-probabilistic
- Level II = probabilistic with approximations
- Level III = fully probabilistic

These four different calculation methods are described and at the end a selection is made which calculation method is applied for this research. The paper of Verhagen [2003] gives a very clear description of the different calculation methods, when designing a coastal protection. This paper is used as guidance to describe the different calculations methods. The description starts with a general explanation and is clarified by an example.

The design of the armour layer of a rubble mound breakwater is used as example with the formula of Van der Meer [1988] for rock and plunging waves.

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$
(4.7)

Where:

Parameters	Symbol	Unit
Nominal stone diameter	<i>D</i> _{<i>n</i>50}	[m]
Damage factor	S_d	[-]
Significant wave height	H_s	[m]
Surf similarity parameter	Ţ	[-]
using mean wave period	ςm	
Number of waves	Ν	[-]
Slope	α	[°]
Relative buoyant density	Δ	[-]
Notional permeability of the structure	Р	[-]
Constant for plunging waves	c_{pl}	[-]

Table 4.1: Input parameters Van der Meer formula for deep water with plunging waves

In the formula of Van der Meer the nominal stone diameter (D_{n50}) is seen as the resistance (R) since this indicates the strength of the structure. The solicitation (S) is defined as the significant wave height (H_s) divided by the right hand side of the formula, which describes the load on the structure.

$$R = D_{n50}$$

$$S = \frac{H_s}{c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \Delta}$$
(4.8)

This results in the following reliability function:

$$Z = D_{n50} - \frac{H_s}{c_{pl} P^{0.18} \left(\frac{s}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \Delta}$$
(4.9)

4.2.1. Deterministic

A deterministic calculation method uses deterministic or nominal values of the basic variables. This gives a value for the strength and a value for the load. Resulting in a single state for the reliability function, i.e. failure, limit state or no failure. Often a global safety factor (γ) is applied to deal with the unknown uncertainties in the basic variables. [Jonkman et al., 2015]

$$R_{nom} \ge \gamma \cdot S_{nom} \tag{4.10}$$

The formula of Van de Meer given in equation 4.7 is used as an example. The values for all parameters can be determined on the basis of boundary conditions, research and design requirements. For example the value of H_s is set by determining the governing significant wave height during the chosen design storm. When all values for the requested parameters are known the required stone size (D_{n50}) can be calculated.

The required stone class is chosen according to the calculated D_{n50} . This stone class should results in a positive value of *Z*, so the design won't fail for the *calculated* situation. However if the value of *Z* is close to zero, a small margin between the resistance to failure and the load, only a small difference in the calculated situation could lead to failure. To ensure the reliability of the structure a global safety factor is introduced, this is often a 95% confidence interval (γ).

This 95% confidence interval is included in the Van der Meer formula via the parameter c_{pl} . To determine the value of c_{pl} , Van de Meer carried out a lot of experiments. To obtain the average expected value for $\frac{H_s}{\Delta d_n}$ a value of 6.2 for c_{pl} was found. However in practice it is recommended to use a value of 5.5 for c_{pl} , this value is the 95% non-exceedance value or referred to as the global safety factor (γ). [d' Angremond et al., 2012]

4.2.2. Semi-probabilistic

The semi-probabilistic method is a level I calculation. This calculation is based on standards (e.g. PIANC [1992]) and no failure probabilities are calculated. Partial safety factors are implemented for the strength (γ_r) as well as for the load (γ_s).

$$\frac{R}{\gamma_r} > \gamma_s S \tag{4.11}$$

These partial safety factors create a certain margin between the representative value of the strength and the load. This margin ensures that the designed element is sufficiently reliable. [Jonkman et al., 2015]

A well known study on the use of partial safety factors is given in PIANC [1992]. The Van der Meer formula for rock with plunging waves including the safety factors is according to PIANC as follows.

$$\frac{1}{\gamma_z} c_{pl} P^{0.18} \Delta d_{n50} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \ge \gamma_{H_s} H_s^T$$
(4.12)

One partial safety coefficient is applied for the significant wave height (γ_{H_s})which is the common parameter in all the formulae for the failure mechanisms in the research of PIANC [1992]. The other partial safety coefficient is applied on the rest of the formula (γ_z).

These partial safety factors are determined in the study of PIANC [1992] by three different models. For all models a target reliability and structural life time are set. The research of PIANC shows that model 1 is the most accurate. The partial safety coefficient are given as:

$$\gamma_{H_s} = \frac{H_s^{t_{P_f}}}{H_s^{t_L}} + \sigma' \frac{\left(1 + \left(\frac{H_s^{3t_L}}{H_s^{t_L}} - 1\right) k_\beta P_f\right)}{F_{H_s}} + \frac{k_s}{\sqrt{P_f \cdot N}}$$
(4.13)

$$\gamma_z = 1 - \left(k_\alpha \ln k_\beta P_f\right) \tag{4.14}$$

The partial safety coefficient given in equation 4.13 consists of three parts. The first term gives the correct partial safety coefficient provided no statistical uncertainty and measurements errors are present in H_s . The term signifies the encounter probability, which is the exceedance probability of the design wave height during the lifetime of the structure, caused by the vagaries of the nature. This is expressed in the central estimate of H_s for a certain return period given an extreme value distribution for the storms, for example a Weibull distribution [Verhagen, 2003]. $H_s^{t_L}$ is the central estimate for H_s with a return period equal to the design life time (t_L) of the structure. For example an allowable failure probability ($P_{f,sys,tl}$) of 20% during a design life time (t_L) of 50 years results in a return interval for the storms (t_{P_f}) of 225 years (i.e. a yearly probability of failure $P_{f,y} = 1/225$), see equation 4.15. [PIANC, 1992; Verhagen, 2003]

$$P_{f,sys,t_L} = 1 - (1 - P_{f,y})^{t_l}$$
(4.15)

The middle term is included in the the partial safety factor to take into account the measurement errors and the short-term variability related to the wave data. This is done by implying a factor (F_{H_s}) on H_s . The factor (F_{H_s}) has a mean value of 1.0 and a variational coefficient $\sigma'_{F_{H_s}}$.
Typical values for this variational coefficient depends on the method of determination and are given in the PIANC [1992] report. The value of $H_s^{3t_L}$ is determined as the central estimate of H_s with a return period equal to three times the design life time (t_L) of the structure.

The last part stands for the statistical uncertainty of the estimated extreme distribution of H_s . The statistical uncertainty depends on the number of wave data (*N*) if the extreme wave statistics is also based on *N* wave data. If this is not the case, for example, H_s is determined by water level variations in shallow water, then the last term is removed and $\sigma'_{F_{H_s}}$ accounts for the inherent uncertainty. The coefficient k_s depends on the type of failure mechanism and is found by optimization. [PIANC, 1992]

The partial safety coefficient given in equation 4.14 is applied on the rest of the Van der Meer formula for rock with plunging waves, see equation 4.12. This coefficient depends on the allowable failure probability during lifetime (P_f) and the coefficients k_{α} and k_{β} . The value of these coefficients are obtained by carrying out optimization for each failure mechanism and given in PIANC [1992].

The research of PIANC [1992] showed that the model, of which the partial safety factors are described above, is the most accurate and the performance is acceptable. However it is concluded that the bulk of results generated with this model will be between 9% underestimation and 13% overestimation of the block volume. Even a underestimation of 15% could occur. Furthermore the proposed method is only applicable for deep water conditions.

4.2.3. Probabilistic with approximations

A third possibility is to use a level II reliability method which is a probabilistic calculation with approximations. This method determines the probability of failure by linearisation around the design point. Only the concept is explained and no example is given since the calculations are based on a iterative process and rather extensive. For examples and further explanation is referred to Jonkman et al. [2015] and Schiereck [2001].

In this method the reliability function is described with a normal distribution. However some parameters can have an deviating distribution. This distribution needs to be replaced by a normal distribution with the same value and slope in the design point. The design point is the point with the highest probability density on the line Z = 0, see figure 4.1, often referred to as the most probable failure point [Jonkman et al., 2015]. In figure 4.2 an example of a linearisation in a design point is shown.



Figure 4.2: Linearisation in design point [Jonkman et al., 2015]

In Jonkman et al. [2015] explained is how the design point can be found with several methods. This is an iterative process and requires extensive calculation, which is not further elaborated here.

Once the properties, μ_z and σ_z , of the normally distributed Z-function are known the probability of failure can be determined. The probability of failure (P_f) is directly related to the reliability index (β) :

$$\beta = \frac{\mu_z}{\sigma_z} \tag{4.16}$$

Once the reliability index is known the probability of failure can be calculated as showed in figure 4.3.



Figure 4.3: Failure probability in level II method [Schiereck, 2001]

Another very important and useful characteristic of the level II method is the influence coefficient (α) that is calculated.

This coefficient shows the influence of the uncertainty a certain parameter (σ_{x_i}) in the reliability function to the total uncertainty (σ_Z) .

(

$$x_i = \frac{\partial Z}{\partial x_i} \frac{\sigma_{x_i}}{\sigma_Z} \tag{4.17}$$

If (α_i) is large it indicates that the variable related to this (α_i) has a large contribution to the total uncertainty of the reliability function and consequently to the failure probability of the structure. This information can be used to reduce the uncertainty in this variable and as a result the failure probability.

4.2.4. Fully probabilistic

When a fully probabilistic method is applied, known as a level III method, the probability of failure is calculated exactly and directly linked to the reliability of the element. This is done by solving the following integral [Jonkman et al., 2015]:

$$P_f = \int\limits_{g(\underline{X})<0} f_{\underline{X}}(\underline{x}) \ d\underline{x}$$
(4.18)

The probability density functions of all strength and load variables are considered. To calculate the probability of failure in equation 4.18 an explicit calculation or a Monte Carlo simulation can be used. Which method to use depends on the number (*n*) of variables and integrals. When n > 2 the only practical method to solve equation 4.18 is by Monte Carlo simulations. Further explanation on the possible level III methods is given in appendix B. [Jonkman et al., 2015]

To determine the suitable level III, method the design equation has to be rewritten to a reliability function. The formula of Van der Meer for a rock armour layer of a rubble mound breakwater with plunging waves, see equation 4.7, is used as example. Often design functions are defined on the basis of experiments and fit parameters are used which is also the case for the Van der Meer formula. The value of c_{pl} in the Van der Meer equation (4.7) is a fit parameter based on experiments carried out by Van der Meer [Van der Meer, 1988]. In fit parameters or other variables in design formulas extra safety can be hidden. Important is that fit parameters are examined and hidden safety is expelled from the reliability function. If not, this will result in a overestimated value for the probability of failure.

As defined in the beginning of this chapter the reliability function is as follows:

$$Z = D_{n50} - \frac{H_s}{c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \Delta}$$
(4.19)

Subsequently for each variable the probability density function is determined. For example the variable c_{pl} has a normal distribution with a given mean value and standard deviation [The Rock Manual, 2007].

As seen in equation **??** the reliability function consists of a more than two variables (n > 2) so it's rather complex to solve with analytical formulations or numerical integration. For this reliability function and the other reliability functions concerning the failure mechanisms given in section 3.4 a Monte-Carlo simulation is preferred.

In a Monte Carlo simulation a random value is selected for a variable (X). The idea is that performing this simulation thousands of times all the possible scenarios are predicted. If all possible scenarios are known, the probability of failure can be calculated.

The Monte Carlo simulation draws a random number from a uniform probability density function between zero and one. With this number (X_u) a value of the variable (X) can be determined via the CDF. The CDF of a continuous random variable X is calculated as follow from its probability density function $f_X(t)$.

$$F_X(x) = \int_{-\infty}^{x} f_X(t) dt$$
for $-\infty < x < \infty$
(4.20)

The non-exceedance probability of the random variable X for a value of x can be calculated via the CDF given in equation 4.20. The non-exceedance probability is uniformly distributed between zero and one.

Using this characteristic a random value for the variable X can be generated by drawing a random number X_u with the Monte Carlo simulation, see equation 4.21, and it's illustrated in figure 4.4a.

$$X = F_X^{-1}(X_u) (4.21)$$





(a) Generate random sample [Jonkman et al., 2015]



Figure 4.4: Steps in Monte Carlo analysis

This approach can be applied to multiple variables if the base variables are statistically independent. For each basic variable X_i (i = 1, ..., n) the Monte Carlo procedure simulates N realizations $x_{i1}, x_{i2}, ..., x_{iN}$.

To determine the failure probability for each set j (j = 1, ..., N) the reliability function (Z) is checked. If $Z(x_{i,j}) < 0$ a counter N_f is increased by one. The probability of failure after N simulations is calculated as follows [Jonkman et al., 2015]:

$$P_f = \frac{N_f}{N} \tag{4.22}$$

The Monte Carlo procedure is easily performed by different program languages as pictured in figure 4.4b.

5

Design process

In the previous chapters is explained that the research focusses on the preliminary and final design of a rubble mound breakwater. The preliminary design is made with a semi-probabilistic approach. The final design is created with a full probabilistic design method. As stated in the introduction, chapter 1, one of the research objectives is to describe the probabilistic design process.

In this chapter an overview is given of the probabilistic design method for a rubble mound breakwater. This is the first step in determining the problems an engineer encounters in practice when using such a method. First a simplified flow chart is given with the most important aspects of the design process. Subsequently the total probabilistic design process is given in a detailed process diagram.

Once the detailed overview is made the different aspects of the probabilistic design process are discussed in Part II of this report. Once the system is complete understood and the problems are pointed out, solutions are provided on the basis of the case of Taman.

5.1. Simplified process diagram

In figure 5.1 a simplified overview of the process diagram for the probabilistic design process of a rubble mound breakwater is given. First the boundary conditions are determined and the design requirements are defined. In the first stage of the project a preliminary deterministic design is made. This design is used as a first estimated of the final design. Making a probabilistic design without the basis of a preliminary design is like a ship without a rudder. The total reliability, expressed in a probability of failure, is calculated for the probabilistic design. The reliability of the system is checked with the design requirements and the design can be adjust if necessary.



Figure 5.1: Simple process diagram probabilistic design

5.2. Detailed process diagram

In appendix C the detailed process diagram is given. In figure 5.2 a flowchart is given with all the steps that has to be taken to go through the detailed process diagram. The steps in this flowchart corresponds with the steps in the detailed process diagram in appendix C. Each step is discussed shortly below and explained in more detail in the next chapters

The boundary conditions are subdivided in hydraulic, geotechnical and geometric (step 1). Some boundary conditions can't be subdivided and are classified as other boundary conditions. The boundary conditions that needs to be determined and the encountered problems are further discussed in chapter 8. The design requirements (step 2) need to be specified for the Ultimate Limit State (ULS) and the Serviceable Limit State (SLS) and expressed in a probability of failure for the design life time, see chapter 7.

As first estimate for the final design is based on the preliminary deterministic design (step 3a), see chapter 9. This design is made with deterministic input variables and a semi-probabilistic design method as described in 9. The next step is to make a probabilistic design based (step 3b) on the preliminary design. First the preliminary design is subject to a probabilistic reliability method. The probabilistic method uses the same input variables, however now a probability distribution is determined for each variable. Subsequently the reliability of each failure mechanisms is calculated, see chapter 11 (steps 4 to 8).

Before the total reliability of the system can be determined (step 9) the aspect correlation needs to be examined. Correlation arises multiple times in the probabilistic design process as seen on the detailed process diagram. Correlation occurs between boundary conditions, failure mechanisms and different sections of the breakwater. Further explanation is given in chapter 10.

After the reliability of the complete system is known the design requirements are checked for the ULS and the SLS (step 10). If the design requirements are not fulfilled, a fail loop (right solid line in flowchart) is executed and the design is adjusted. In case the design requirements are met, further optimization can be made (step 11). All these steps to perform a fully probabilist calculation are pictured in a flowchart in figure 5.2.



Figure 5.2: Flowchart of probabilistic design process based on Allsop et al. [1999]

II

Fully probabilistic design (level III) of a rubble mound breakwater

6

Simplified case

The objective of this research is to investigate if a probabilistic calculation for a rubble mound breakwater can be made. The flowchart of the design process (figure 5.2) shows that a preliminary design (level I) is required to make a fully probabilistic design (level III). Since making a preliminary design takes a lot of time, a case is used for which the preliminary design is already made. This also means that the boundary conditions in the fully probabilistic design process are determined for the project location of the chosen case, see chapter 8.

The project Taman is used as case. From this project a rubble mound breakwater is selected and one section of the breakwater is examined. Furthermore the decision is made to investigate only a couple of failure mechanisms present in this section. These decisions leads to the simplified case which is completely elaborated in the next sections.

6.1. Project Taman

As mentioned the simplified case is derived from the project Taman. The Taman project comprises the design of a new deep sea port. The port is protected by breakwaters from severe waves from the Black Sea and the Sea of Azov. The breakwaters of Taman consist of several different sections with for each section different boundary conditions and failure mechanisms. In the simplified case section B is examined, see figure 6.1.



Figure 6.1: Overview project Taman, rubble mound breakwater of section B is highlighted in red

For section B a couple of failure mechanisms are taken into account. In this simplified case four failure mechanisms which are related to the Ultimate Limit State (ULS) of the breakwater are examined, see figure 6.3.

With these simplifications a clear and thoroughly insight is gained in the probabilistic design process of a rubble mound breakwater without loosing track of the actual objective of this study. In the section 6.2 a typical cross-section for the simplified case is given and in section 6.3 the four different failure mechanisms are described. Subsequently the design formulae for the selected failure mechanisms are given in section 6.4.

6.2. Structure properties

The structural properties are based on the preliminary design which is already made for the project Taman. Section B is selected because this section has an armour layer which consists of rock. A rock section is selected, because many research is available about the uncertainties of the design methods for rock slopes. Furthermore it fits the criteria for this research (see section 3.3).

The existing design of the armour layer of the rubble mound breakwater at the seaside consists of rock with a grading of 60-300 kg, which also holds for the toe at the seaside. The armour layer is directly placed on top of the quarry run. The rear-side armour layer is constructed with a grading of 5-40 kg. On top of the breakwater a crest element is placed.

Below the cross-section for section B is given and the components, described in section 3.3, which are investigated by means of the failure mechanisms are highlighted in red. The investigated failure mechanisms and related design formulae are further discussed in the next sections. The characteristics of the design, given in figure 6.2, are completely explained in chapter 9.



Figure 6.2: Cross-section B with highlighted investigated components

6.3. Fault tree and failure mechanisms

As seen in Appendix A the total fault tree for the ULS is subdivided in the different components of the rubble mound breakwater. In the simplified case only the armour, toe and subsoil are examined, see figure 6.3. The failure mechanisms for the armour are stability of the layer at seaside (1) and the rear-side (2). Furthermore the stability of the toe (3) is taken into account. These three failure mechanisms are all strongly related to the hydraulic boundary conditions (water level and waves). At last the slip of the subsoil (4), which is a geotechnical failure mechanism, is investigated. The distinction between hydraulic and geotechnical failure is made for simplicity. Although this distinction is made in the fault tree, see figure 6.3, there is only one integrated design. Assumed is that the described system of this integrated design is in series, which means that if one of the components fails the total system fails.



Figure 6.3: Fault tree Ultimate Limit State with selected failure mechanisms for simplified case

The choice of the failure mechanisms is based on the fact that these four failure mechanism are also examined in the semi-probabilistic design of the project Taman. Furthermore it is estimated that the selected failure mechanisms have the largest contribution to the total failure probability of the rubble mound breakwater in section B. After the probabilistic calculation this statement is discussed and analysed in section 12.7. Besides these aspects it is most likely that between these failure mechanisms correlation occurs. This is desired so the influence of the correlation on the probability of failure can be investigated with this simplified case.

6.4. Design formulas

In this section all the design formulas are described for the chosen failure mechanisms. In the fully probabilistic design method (level III) the reliability function are required so for the convenience the used formulae are expressed in reliability functions. Furthermore all the input variables are listed. The values and the probability distribution for the variables which are specific for a design formula are also given.

6.4.1. Seaside armour stability

The armour of the breakwater in section B consists of rock. Failure of the armour layer occurs due to instability of the seaside or the rear-side layer. In this subsection the failure due to instability of the seaside armour layer is examined (1). To check if the stability is sufficient the Van der Meer formulae are used [Van der Meer, 1988].

Two aspects determines which type of Van der Meer formula to use. First of all if the breakwater with the investigated armour layer is located in shallow or deep water. This is determined based on the relation between the significant wave height and the water depth at the toe of the breakwater. Furthermore a distinction is made between plunging and surging types of waves.

For simplicity first the distinction between deep and shallow water is made and for each situation the difference between plunging and surging waves is given.

Deep water

Deep or shallow water influences the type of formula. To examine the deep water conditions the formula developed by Van der Meer [1988] is used, which is given below rewritten to a reliability function. The mean wave period T_m is defined as the spectral wave period $T_{m0,2}$.

For plunging $(\xi_m < \xi_{cr})$:

$$Z = \frac{H_s}{\Delta d_{n50}} - c_{pl,d} P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$
(6.1)

For surging $(\xi_m \ge \xi_{cr})$:

$$Z = \frac{H_s}{\Delta d_{n50}} - c_{s,d} \ P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{\cot \alpha} \ \xi_m^P$$
(6.2)

Where

$$\xi_{cr} = \left[\frac{c_{pl,d}}{c_{s,d}}P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$
(6.3)

$$\xi_m = \tan \alpha / \sqrt{2\pi H_s / (gT_{m0,2}^2)}$$
(6.4)

$$\Delta = \frac{\rho_s}{\rho_w} - 1 \tag{6.5}$$

Shallow water

To make the formulae of Van der Meer better applicable in very shallow waters, Van Gent et al. [2004] modified it by using $T_{m-1,0}$ instead of T_m and recalibrating the $c_{pl,s}$ and $c_{s,s}$ values by model tests. This formula is rewritten to a reliability function and as follows:

For plunging ($\xi_m < \xi_{cr}$):

$$Z = \frac{H_s}{\Delta d_{n50}} - c_{pl,s} P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) (\xi_{s-1,0})^{-0.5}$$
(6.6)

For surging $(\xi_m \ge \xi_{cr})$:

$$Z = \frac{H_s}{\Delta d_{n50}} - c_{s,s} P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \left(\frac{H_s}{H_{2\%}}\right) \sqrt{\cot \alpha} \left(\xi_{s-1,0}\right)^P$$
(6.7)

Where ξ_m

$$\xi_{cr} = \left[\frac{c_{pl,s}}{c_{s,s}}P^{0.31}\sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$
(6.8)

$$\xi_{s-1,0} = \tan \alpha / \sqrt{2\pi H_s / (g T_{m-1,0}^2)}$$
(6.9)

Input variables

In table 6.1 the variables are given which are specific for the Van der Meer formula for armour stability. In table 6.2 the input variables are given which differ from case to case. The Van der Meer formula is derived empirical and a fit parameters (c_s , c_{pl}) are applied. Assumed is that all the uncertainties of the variables which are specific for the Van der Meer formula (e.g. P) are included in the probability distribution of the fit parameters.

Variable	Distribution	Mean (µ)	Deviation (σ)	5% limit value
Surging deep water coefficient $(c_{s,d})$	Normal	1.0	0.08	0.87
Plunging deep water coefficient $(c_{pl,d})$	Normal	6.2	0.4	5.5
Surging shallow water coefficient $(c_{s,s})$	Normal	1.3	0.15	1.05
Plunging shallow water coefficient ($c_{pl,s}$)	Normal	8.4	0.7	7.25
Notional permeability of the structure (P)	Deterministic	0.1 - 0.6	-	-
Damage factor (S_d)	Deterministic	2-17	-	-

Table 6.1: Formula specific input variables Van der Meer formula for Armour stability

Variable	Symbol	Unit
Nominal stone diameter	<i>D</i> _{<i>n</i>50}	[m]
Damage factor	S _d	[-]
Significant wave height	Hs	[m]
Wave height exceeded by 2%	H _{2%}	[m]
Mean wave period	<i>T_{m0,2}</i>	[s]
Spectral mean energy wave period	$T_{m-1,0}$	[S]
Number of waves	N	[-]
Slope	α	[°]
Density of stone	ρ_s	$[kg \ m^{-3}]$
Density of water	ρ_w	$[kg \ m^{-3}]$
Standard gravity	g	$[m s^{-2}]$

Table 6.2: Input variables Van der Meer Armour stability formula, equations 11.3 - 6.9

An important parameter in the Van der Meer formula is the damage factor (S_d) . The The Rock Manual [2007] provides some guidance for several stages of damage dependent on the slope of the structure, see table 6.3.

Slope $(\cot(\alpha))$	Stage of damage		
	Start of damage	Intermediate damage	Failure
1.5	2	3-5	8
2	2	4-6	8
3	2	6-9	12
4	3	8-12	17

Table 6.3: Guidelines for the damage factor, S_d , for armour stone in a double layer [The Rock Manual, 2007]

6.4.2. Rear-side slope stability

The stability of the rear-side slope of the breakwater is examined with the formula which is derived by Van Gent and Pozueta [The Rock Manual, 2007]. The formula is rewritten to a reliability formula.

$$Z = D_{n50} - 0.008 \left(\frac{S_d}{\sqrt{N}}\right)^{-1/6} \left(\frac{u_{1\%} T_{m-1,0}}{\sqrt{\Delta}}\right) (\cot \alpha_{rear})^{-2.5/6} \left(1 + 10 \exp\left(\frac{-R_{c,rear}}{H_s}\right)\right)^{1/6}$$
(6.10)

Where:

$$u_{1\%} = 1.7 \left(g\gamma_{f-c}\right)^{0.5} \left(\frac{R_{u1\%} - R_c}{\gamma_f}\right)^{0.5} / \left(1 + 0.1\frac{B}{H_s}\right)$$
(6.11)

Where:

$$R_{u1\%}/(\gamma H_s) = c_0 \xi_{s-1,0} \qquad for \ \xi_{s-1,0} \le p \tag{6.12}$$

$$R_{u1\%}/(\gamma H_s) = c_1 - c_2/\xi_{s-1,0} \qquad for \ \xi_{s-1,0} > p \tag{6.13}$$

And $p = 0.5 \frac{c_1}{c_0}$, $c_2 = \frac{c_1^2}{c_0}$. Furthermore $\gamma = \gamma_f \gamma_\beta$ with $\gamma_\beta = 1 - 0.0022\beta$. The damage factor (*S*_d) is determined via table 6.3.

Input variables

In table 6.4 all the input variables are listed for the rear-side slope stability formula which are case specific. Variables which are already included in table 6.2 are left out. This formula is also derived empirical and the factor of 0.008 should be seen as a fit parameter. However no standard deviation is given for this value. Van Gent and Pozueta expressed the uncertainty of this formula in the factor $\frac{S_d}{\sqrt{N}}$. For values of $S_d < 10$ a standard deviation of $\sigma = 0.1$ is used. For larger values of S_d a deviation of $\sigma = 0.3$ should be used. The coefficient c_0 and c_1 are assumed to be deterministic since the uncertainty is included in the factor $\frac{S_d}{\sqrt{N}}$. In table 6.5 the formula specific variables are given with their probability distribution.

Variable	Symbol	Unit
Angle of rear side slope	α_{rear}	[°]
Crest free board relative to water level at rear side	$R_{c,rear}$	[m]
Maximum velocity at rear side of the crest during overtopping exceeded by 1% of the waves	$u_{1\%}$	$[m \ s^{-1}]$
Crest level relative to still water at seaward side	R_c	[m]
Crest width	В	[m]
Roughness of seaward slope	γ_f	[-]
Roughness at the crest	γ_{f-c}	[-]
Fictitious run-up level exceed by 1% of the waves	$R_{u1\%}$	[m]
Angle of incoming waves	β	[°]
Coefficient	c_0	[-]
Coefficient	c_1	[-]

Table 6.4: Case specific input variables for Van Gent and Pozueta rear-side stability formula

Coefficients	Distribution	Mean (μ)	Deviation (σ)
Coefficient c_0	Deterministic	1.45	-
Coefficient c_1	Deterministic	5.1	-
Factor $\frac{S_d}{\sqrt{N}}$	Normal	-	0.1 or 0.3

Table 6.5: Formula specific input variables Van Gent and Pozueta rear-side stability formula

6.4.3. Stability of the Toe

The stability of the toe, indicated with number 3 in the fault tree, is calculated with the formula developed by Van de Meer [The Rock Manual, 2007]. The design formula is rewritten to a reliability formula and given below.

$$Z = D_{n50} - \frac{H_s}{\left(2 + 6.2\left(\frac{h_t}{h}\right)^{2.7}\right) \cdot N_{od}^{0.15} \Delta}$$
(6.14)

Input variables

The input variables are given in table 6.6. To prevent repetition, the variable is not included in table 6.6 if this one is already listed in table 6.2 or 6.4. In the formula for the toe stability the specific variables are the damage factor N_{od} and the factor h_t/h to describe the uncertainty in this formula.

An important parameter in the Van der Meer formula for the toe stability is the damage factor N_{od} . In table 6.7 the possible values for this damage factor are given based on the stages of damage.

Variable	Symbol	Unit
Water depth in front of toe	h	[m]
Depth of toe below water level	h_t	[m]
Damage factor for toe	Nod	[-]

Table 6.6: Input variables Van der Meer Toe stability formula 11.5

Stage of damage	Nod
Almost no damage	0.5
Acceptable damage	2.0
Failure	5

Table 6.7: Guidelines for the damage factor, N_{od} , for toe stability in a double layer [The Rock Manual, 2007]

The formula for the toe stability is derived empirical. The uncertainty in this formula is determined by examining the measured data, see figure 6.4 [The Rock Manual, 2007]. The uncertainty is expressed in the factor h_t/h , the estimated standard deviation is $3\sigma = 0.15$ so $\sigma = 0.05$. This is implemented in the probabilistic design process.



Figure 6.4: Toe stability as function of h_t/h [The Rock Manual, 2007]

6.4.4. Macro stability

The macro stability of the breakwater is checked with the formula of Bishop [Schiereck, 2001].

$$F = \frac{\sum \frac{c + (\rho_s g h - p) \tan \phi}{\cos \alpha_s (1 + tan(\alpha_s) tan(\phi/F))}}{\sum \rho_s g h \sin \alpha_s}$$
(6.15)

Input variables

The input variables are given in table 6.8. To avoid redundancy the variable is not included in table 6.8 if this one is already listed in table 6.2, 6.4 or 6.6. In the Bishop formula no formula specific variables are included, the probability distribution of the variables given in table 6.8 are given in chapter 9.

Variable	Symbol	Unit
Stability number	F	[-]
Cohesion of soil	С	[Pa]
Water pressure	р	[Pa]
Density of soil	ρ_s	$[kg \ m^{-3}]$
Internal friction angle	ϕ	[°]
Angle of slip circle	α_s	[°]

Table 6.8: Input variables Bishop formula for Macro stability in equation 11.6

Design requirements

In the first stages of the project the design requirements are defined which depend on the desires of the client, the purpose of the breakwater and the present environment. The final design should meet the design requirements.

The design requirements are specified depending on which kind of reliability method is chosen. The semiprobabilistic design methods (level II) are used most of the time and is also applied in this research to the preliminary design. Furthermore this research focusses is on the fully probabilistic design method (level III) for the final design, see figure 7.1. In the following paragraph the different design requirements are explained and a guideline is given how to determine the target reliability (i.e. allowable failure probability) of the structure.



Figure 7.1: Indication of design requirements in probabilistic design process

7.1. Required design requirements

In section 3.1 a system analysis is made for the investigated port and a fault tree is given in figure 3.1. The top event of this fault tree is unacceptable port downtime during design lifetime. Multiple events can lead to unacceptable down time which requires several design requirements. In this research a distinction is made between limit states of the breakwater and other events which result to port downtime (i.e. different objects of the port). This is also pictured in the fault tree in figure 3.1, a few examples are given below:

- The breakwater should be accessible for pedestrians 350 days a year. (SLS Breakwater)
- Only 1 day a year obstruction of the entrance channel is allowed. (SLS other events)
- The quay wall has a maximal allowable failure probability of 0.1% per year. (ULS other events)
- Failure probability of breakwater may not exceed 15% for a lifetime of 50 years. (ULS Breakwater)

As described in chapter 6 the simplified case only focuses on the ultimate limit state for the rubble mound breakwater. In figure 7.2 the method to select the correct design requirements for the selected object and limit state is pictured. Important to notice is that the preliminary and probabilistic design requirements are derived from the same design requirements. This design requirement for the ULS of the breakwater is the allowable failure probability (P_{f,sys,t_L}) during the design life time (t_L). The definition of failure should therefore be defined accurately. In the following sections is explained how the design requirements for the preliminary and probabilistic design should be specified.



Figure 7.2: Determining design requirements for ULS rubble mound breakwater

7.2. Preliminary design method

The preliminary design for the simplified case, given in chapter 6, is based on a semi-probabilistic design method, see subsection 4.2.1. It is also possible to use the PIANC method as explained in subsection 4.2.2, however this method is not used for the preliminary design in this case.

The preliminary design is based on the preliminary design requirements which are derived form the requirements for the ULS of the breakwater, see figure 7.2. Each failure mechanisms for the ULS requires a different specification of the design requirements. As seen in section 6.3 the failure mechanisms are subdivided in hydraulic and geotechnical mechanisms. For the preliminary design it's convenient to describe the design requirements in the way the failure mechanisms are divided. Since in the semi-probabilistic reliability method each failure mechanism is considered separately. At last the geometric design requirements are given, these often imply boundaries as seen in C.

7.2.1. Hydraulic design requirements

The main difference between the design requirements for the preliminary and probabilistic design are the specification of the hydraulic design requirements. For both reliability methods the allowable failure probability (P_f) during the design lifetime (t_L) are used. However in the preliminary design a design storm is determined via the Poisson distribution, see equation 4.15, based on the failure probability and design lifetime. This results in a design storm with a certain return interval (t_{P_f}) from which the required hydraulic input parameters for the design formulae can be derived, such as the significant wave height (H_s).

Aside from this approach, sometimes a design storm with a return interval is chosen by expert judgement based on the purpose of the breakwater and experience with other similar types of structures. This is also done for the rubble mound breakwater in the simplified case, a design storm with a return period of 1/100 year is set as design requirement. Furthermore for each failure mechanisms different design requirements are specified. For example the Van der Meer formula, see equation 4.7, requires the definition of the damage factor (S_d). The value for the damage factor is in the preliminary design specified as the start of damage, given in The Rock Manual [2007].

The design storm and the damage factors can be seen as disguised safety factors because they result in safety margins. Often extra safety margins are implied in the semi-probabilistic design method. In some formulae this is very clear, for example the coefficients c_{pl} and c_s in the Van der Meer formula which implies a 95% non-exceedance value. On the other in some formulae it is not always clear if implicit safety limits are implied. Especially for complex empirical formulae such as the formula of Van Gent and Pozueta for the rearside slope stability, see equation 11.4. This formula is used in the simplified case.

So the design storm can be seen as the overall hydraulic design requirement for the preliminary design and depending on the different failure mechanisms additional requirements are needed. Below the design requirements for the ULS of the preliminary design in the simplified case are given. The values for the design requirements depend on the used design formulae. In the simplified case the design formulas of Van der Meer are used for the seaside armour and toe stability [The Rock Manual, 2007].

Overall hydraulic design requirement:

• Design storm with probability of exceedance of 1/100 year;

Specific design requirements:

- Maximum damage level for rock slopes is 'start of damage' ($S_d = 2$); (Sea and rear-side armour stability)
- 95% non-exceedance value for $c_{pl,d}$ = 5.5 and $c_{s,d}$ = 0.87; (*Sea side armour stability*)
- 95% non-exceedance value for $c_{pl,s}$ = 7.25 and $c_{s,s}$ = 1.05; (sea side armour stability)
- The maximum damage factor for the toe berm is 'start of damage' ($N_{od} = 0.5$). (Toe stability)

7.2.2. Geotechnical design requirements

The geotechnical design requirements for the preliminary design are not directly based on the probability of failure during the design lifetime of the breakwater.

Most of the geotechnical failure mechanisms for rubble mound breakwaters are related to slip stability. A distinction is made between the static situation and the dynamic situation (earthquakes). For both these scenarios the stability factor (F) needs to be defined. Usually partial safety factors are applied for soil parameters to include safety limits for the unknown uncertainties (level I).

Other failure mechanisms are sliding, tilting and settlement. In the simplified case these are not explained in further detail. However the concept is the same, partial safety factors are applied to cover the unknown uncertainties.

In the simplified case only static loading is considered and only the macro stability. For this failure mechanisms the Bishop formula applies, see 6.4.4. The stability number in this formula has to be larger than one to ensure stability (F > 1). In addition partial safety factors are applied to take in account the uncertainties. These partial safety factors are applied on the soil characteristics for static loading and given in the Eurocode 7 [British Standards Institution, 2004].

- Cohesion *y_c*: 1.25
- Friction angle y_{ϕ} : 1.25
- Undrained shear strength y_{cu} : 1.40

7.2.3. Geometric design requirements

Geometric design requirements are often imposed by functional requirements and availability of equipment and materials. Examples of functional requirements are the maximal height for ships to ensure enough sight, the area where the structure can be placed or the available stone classes. These requirements are translated to geometric boundary conditions and indicates the range for possible solutions.

In the simplified case only one geometric design requirement, which set boundaries to the solution space, is present. Namely, only standard gradings as presented in The Rock Manual [2007] are available for the project. At the project location the maximum available standard grading for stone is 300 - 1,000 kg. For large project is can be more economic to design fit for purpose non-standard gradings.

7.3. Probabilistic design method

For the probabilistic design method no distinction is made between the different design requirements for individual failure components. The key concept of the probabilistic approach is that the minimal required reliability of the structure is set and examined by design formulae for the failure mechanism. The target reliability is usually specified via the allowable failure probability (P_f) during the design lifetime (t_L), often in the order of 10 - 20%. From here on the design storm is determined, see section 7.2 [Verhagen, 2003]. This approach is generally accepted when using a semi-probabilistic design method. However for a probabilistic approach this probability of failure can overestimate because hidden safeties are present in the design formulae which are used for a semi-probabilistic design method. For example in the value for c_{pl} as explained in section 4.2.1. When this approach is also applied in a probabilistic design method these values add extra (i.e. hidden) safeties and result in an underestimation of the reliability.

The target reliability is the basis for the preliminary and probabilistic design requirements and referred to as the design requirement for the ULS of the rubble mound breakwater, see figure 7.2. How to determine this target reliability is explained in the following sections.

7.3.1. Target reliability

In the simplified case (level I) a storm with a return period of 1/100 year is taken as overall design requirements for the hydraulic failure mechanisms. If this is applied in the fully probabilistic design method (level III) as the maximal allowable failure probability, it means that during a lifetime of 50 years the structure has a change of 20% to fail. However for the fully probabilistic design method no design storm is chosen, so a different approach is used.

In PROVERBS (see Allsop et al. [1999]) three points of view are given which should be considered when determining the target reliability for the fully probabilistic design method.

- A personal acceptable level of risk
- · A socially acceptable level of risk
- · Acceptable level of risk based on economic optimization

In this case the personal risk is low, since the rubble mound breakwater does not protect a living area. So the target probability should be based on the socially acceptable level of risk and the optimal risk with respect to economic optimization.

The target reliability of the structure can be determined in several ways. In PROVERBS four different methods are given:

Expert judgement: based on experience and knowledge from other types of similar structures.

- **References case:** Calculating the reliability level of similar structures which have proven to be designed well from an economic and safety point of view.
- **Codes and standards:** The target reliability depends on the safety class and consequences of failure. These codes and standards are based on structural systems and marine structures where probabilistic methods have been applied.

Economic optimization: The total expected costs in the lifetime are minimized with respect to typical design parameters. The costs in this approach consists of construction costs, maintenance costs and expected failure costs. This is the level IV method as shortly described in section 1.1.

The economic optimization method is illustrated in figure 7.3, the construction cost are plotted against the failure cost. The most favourable solution from an economical point of view is the design with the lowest cost. This point is indicated in the figure with *Optimal Pf*. However it is very difficult to estimate the cost due to failure and this method does not take in account and human and gross errors. Therefore this approach can be subject to large uncertainties. For these reasons it is preferred to use expert judgement and reference cases instead of the level IV method when it is hard to estimate the costs. (PROVERBS [Allsop et al., 1999])



Figure 7.3: Economic optimization and target reliability

Determining the target reliability based on expert judgement, reference cases and codes will often not result in the most economical optimal design, as pictured in figure 7.3 with the blue lines. Although the level IV method is not part of this study, see section 4.2, the consequences of failure should be examined to make a accurate estimate of the target reliability. In Eurocode 1 [1994] and ISO [1996] values are given for the target reliability taking in account the cost and the consequences of failure. A more specific code for marine structures is develop by Det Norske Veritas in 1992, see table 7.1. The class of failure depends on the possibility for timely warning and development of failure and the possibility to repair. To determine the consequences of failure the use, location and surroundings of the structure should be examined. The consequences of failure depends on for example personnel injuries, physical damage and economical losses as a result of failure.

These codes and standards should be used as a first estimate and guidance. A more precise target reliability should be determined by evaluating existing structures of the same type and function that are known to have adequate safety.

	Consequence of failure		
Class of failure	Less serious	Serious	
I - Redundant structure.	$P_F^t = 10^{-3}$	$P_F^t = 10^{-4}$	
	$(\beta^{t}=3.09)$	$(\beta^{t}=3.71)$	
II - Significant warning before the occurrence	$P_F^t = 10^{-4}$	$P_F^t = 10^{-5}$	
of failure in a non-redundant structure.	$(\beta^{t}=3.71)$	$(\beta^{t} = 4.26)$	
III - No warning before the occurrence of fail-	$P_F^t = 10^{-5}$	$P_F^t = 10^{-6}$	
ure in a non-redundant structure.	$(\beta^{t} = 4.26)$	$(\beta^{t} = 4.75)$	

Table 7.1: Acceptable annual probabilities of failure for marine structures [Det Norske Veritas, 1992]

The target reliability differs from case to case depending on the consequences of failure and class of failure. Below a guideline is given to determine the target reliability for a rubble mound breakwater:

- 1. Determine target reliability based on codes and standards (e.g. table 7.1)
- 2. Examining target reliability of similar structures which are known to have adequate safety. (e.g. PIANC [1992]; Burcharth [1994]; Benassai et al. [2000] and PROVERBS [Allsop et al., 1999])
- 3. More case specific estimate based on expert judgement
- 4. Economic optimization via a Level IV reliability method. (figure 7.3)

The sequence of these steps is important and should be followed precisely. Each step taken results in a target reliability which is more case specific. Step 1, 2 and 3 give the target reliability for a fully probabilistic design method (level III). Step 4 is optional and consists of a risk based design method. (level IV)

Advised is to not skip any steps to prevent incorrect estimates of the target probability. For example only performing a economic optimization (step 4) could result in a very high probability of failure which is maybe not social acceptable.

7.4. Conclusion simplified case

The design requirements for the preliminary design (i.e. semi-probabilistic) are already know for the simplified case. The design requirements for the fully probabilistic calculation should be determined from scratch. This is done with the information given in the previous sections.

The most important design requirements for the probabilistic design is the target reliability (i.e. allowable failure probability). This is determined via the four steps given in the previous sections:

1. Determine target reliability based on codes and standards (e.g. table 7.1)

The rubble mound breakwater is seen as a redundant structure, if one of the structures components is damaged it won't necessarily fail, for example breakage of one stone. Furthermore there is significant warning before failure, a rubble mound breakwater has a dynamically stable profile (section 3.2). The consequence of failure are quantified as less serious since there is only economic damage and nearly no risk of personal injuries. Using table 7.1 this results in a failure probability of 10^{-3} per year (P_F^t). Which is a failure probability (P_{f,sys,t_L}) of 5% for a structure with a lifetime of 50 years.

2. Examining target reliability of similar structures which are known to have adequate safety. (e.g. PIANC [1992]; Burcharth [1994]; Benassai et al. [2000] and PROVERBS [Allsop et al., 1999])

The second step is to examine the reliability of similar structures. PROVERBS did the most extensive research and states that a typical value for the probability of failure for recently (around 2000) designed structures is 15% for the total lifetime (P_{f,sys,t_L}).

3. More case specific estimate based on expert judgement

In 2000 the breakwater of Punta Langosteira was designed with a fully probabilistic calculation method (level III). One of the engineers, Enrique Maciñeira, who worked on this project was consulted to get more insight in the latest perspectives on the target reliability. For the secondary breakwater with dangerous goods in the rear part of the breakwater a target failure probability of 1% for the ULS with a lifetime of 50 years is used. The main breakwater, where the consequences of failure are lower, a target reality of 3% for initiation of damage and 15% for total destruction for a lifetime of 50 years is applied [Maciñeira, 2016]. This shows that the consequences of failure have a large influence on the target reliability which is also clear from the codes and standards.

4. Economic optimization via a Level IV reliability method.

This step is not part of this research and not further investigated.

In table 7.2 an overview is given of the resulting target failure probability per step. In conclusion the target failure probability for the simplified case is set to 15% for serious damage for a lifetime of 50 years (P_{fil}). This is based on the codes and standards (1), target reliability of similar structures (2) and an expert opinion (3). The value of step 3 is taken as governing value since this one is the most case specific.

Step	Based on	Target failure probability $(P_{f_{tl}})$ [%]
1	Codes and standards	5
2	Similar structures	15
3	Expert judgement	15
4	Level IV reliability method	-

Table 7.2: Determining the target failure probability for the rubble mound breakwater in simplified case

The chosen target failure probability of 15% ($P_{f_{tl}}$) for a design lifetime of 50 years results in a yearly failure probability (P_F^t) of 3.2 · 10⁻³ (β = 2.72). So in the guidelines set by Det Norske Veritas [1992] given in table 7.1 this is qualified as a redundant structure with less serious consequence of failure.

Furthermore specific design requirements for each failure mechanisms should be derived. These are the damage factors and the applied safety factors for the geotechnical design. The damage factors are set on the basis of tables 6.3 and 6.7 for the damage level 'failure'. To prevent interpolation a slope of 1:2 is assumed for the damage factor S_d . In table 7.3 all the relevant design requirements in the fully probabilistic design method for the ultimate limit state (ULS) are given for the simplified case. A comparison is made with the semi probabilistic design requirements.

Design requirements		Calculation method		
		Semi-probabilistic	Fully probabilistic	
Overall requirements	Target reliability	1/100 year storm	$15 \% P_{f_{tl}}$	
•	Design lifetime	50 years	50 years	
Specific requirements	Damage number, S_d	2	8	
•	Damage number, N _{od}	0.5	5	
	Stability number, F	1	1	
•	Factor cohesion, y_c	1.25	1	
•	Factor friction angle, y_{ϕ}	1.25	1	

Table 7.3: Design requirements for the Ultimate Limit State (ULS) for the simplified case

8

Boundary conditions

In this chapter all the boundary conditions which are required to construct a rubble mound breakwater are discussed. In the following sections the boundary conditions are categorized based on the probabilistic design process given in appendix C. For these boundary conditions also the uncertainties are included via probabilistic distributions. Two types of uncertainties can appear in the probabilistic design process [Allsop et al., 1999]:

- · inherent uncertainties of the input parameters
- · model uncertainty

These are explained in further detail for each type of boundary condition.

8.1. Environmental boundary conditions

According to Van der Meer [1992] part of the boundary conditions can be distinguished as environmental boundary conditions. These conditions cannot be influenced by the structure or the designer of the structure. In the following subsection the environmental boundary conditions are divided in the hydraulic and geotechnical boundary conditions. For some boundary conditions it is not possible to appoint a certain category so they are placed in the group of other boundary conditions.

8.1.1. Hydraulic boundary conditions

The hydraulic boundary conditions define the load for the hydraulic failure mechanisms and partly for the geotechnical failure mechanisms, see section 6.4. In the simplified case of Taman the following hydraulic boundary conditions near the structure are required:

Parameters	Symbol	Unit
Design high water level	D.H.W.L.	[m+BS]
Significant wave height	H_s	[m]
2% Wave height	$H_{2\%}$	[s]
Spectral wave period	$T_{m-1,0}$	[s]
Mean wave period	<i>T</i> _{m0,2}	[s]
Wave direction relative to North	β	[°N]

Table 8.1:	Hydraulic	boundary	conditions
10010 0.11	riyuruune	boundary	contantions

Usually the structure is located nearshore and in shallow water ($d \le \frac{1}{20}L$). Often only the offshore hydraulic conditions in deep water ($d \ge \frac{1}{2}L$) are known. A conversion is needed from the known offshore conditions to the nearshore hydraulic boundary conditions. Numerical wave models (e.g. SWAN), which take into account

all the shallow water effects, are used to calculated the hydraulic boundary conditions near the structure.

These kind of models require input variables such as the wind direction, wind speed, bathymetry, currents and water level (see appendix C). These input variables can be obtained via measurements, satellite data and models (e.g. ARGOSS [2016]; NOAA [2016]). In the following paragraphs is discussed how to determine these variables in the probabilistic design process. Once the water level, currents, bathymetry and wind statistics are determined the hydraulic boundary conditions can be calculated. As mentioned different models are required to make the calculation. Due to time restrictions of this research some simplifications for the models are applied, which are clearly explained in this chapter.

These simplifications are based on an important characteristic of the boundary conditions for the project Taman. The port of Taman is located in the Black Sea and is attacked by waves from the Black Sea and the Sea of Azov. Both these seas are a *closed system* due to the land boundaries around the seas which results in a limited fetch length. In this case the wave statistics (H_s , $H_{2\%}$, $T_{m-1,0}$, $T_{m0,2}$) and the storm surge (wind driven water level set-up) are directly related to the wind speed and direction. If the port was located in an *open system* (e.g. at a shoreline of an ocean), the wave statistics are not directly related to the wind speed. Because swell waves are present and there is no clearly defined fetch length.

Offshore wind direction and speed

The wind speed and direction can be processed in different ways into the wave models. Often the wind data is provided as pictured in figure 8.1. A wind rose is given where the directions are divided in equal bins of certain degrees. For each directional bin a chart with the occurrence of the wind speed in time is given.



Figure 8.1: Wind rose and distribution of wind speed

In figure 8.2 three different ways of dealing with wind data are pictured. In the semi-probabilistic approach, see subsection 4.2.2, a design storm based on the acceptable probability of failure during the design life time is chosen. This design storm has a certain return interval which also applies to the wind speed. The dominant wind direction is determined based on the orientation of the structure and the maximum wind speeds. The wind speed is determined given the dominant wind direction and the calculated return interval, this is pictured in graph C in figure 8.2.



Figure 8.2: Three different options to process wind speed data

The method picture in graph C is only applicable for a semi-probabilistic approach. When applying a probabilistic reliability method multiple scenarios are simulated by for example a Monte Carlo simulation. To make sure all the different scenarios are examined each wind direction and all the corresponding wind speeds should be part of the calculation. This can be achieved by selecting option B in figure 8.2. Depending on the type, number and length of observations, a certain uncertainty is added to the wind speed. Especially for the ones with a small return interval (e.g. 1/1000 year), since it's based on extrapolation of measured values. To take this uncertainty into account option A should be chosen which gives all the possible wind speeds including an uncertainty band.

The simplified case explained in chapter 6 is used to illustrate which method to apply for the fully probabilistic design method. First the occurrence of the offshore wind should be determined. In the case of Taman 16 different wind directions are distinguished for a dataset with a time span of 8 years.

Important is to select the correct wind occurrence per direction for the desired output of the probabilistic calculation. The required reliability of the structure should be calculated per year. Therefore the wind speed which is processed in the model should also be the yearly extreme distributions to obtain the yearly failure rate of the structure. The data of the wind occurrence is measured every three hours. This occurrence can be completely different for the daily winds compared to the yearly extremes. To make it suitable for the occurrence of the yearly extreme distributions a threshold is applied. The applied threshold is a wind speed of $10.97 m s^{-1}$ which is the lowest value of a yearly extreme in the probability distributions, see appendix D. The exact Matlab procedure is given chapter 11. In the case of Taman the occurrence for the daily wind speeds is completely different compared to the wind speeds above the threshold, see figure 8.3.



Figure 8.3: Wind occurrence without and with threshold

Once the occurrence per bin is known the probability distribution for the offshore wind speed is determined for each bin. For the case of Taman several measurements are carried out for the wind speed and a Weibull distribution is fitted trough these parameters. This data was already available however for simplicity a exponential distribution is used to approach the Weibull fit. The wind speeds start with a exceedance probability of once a year since only the yearly extremes are taken into account in this research.

Two bins are used as an example which are present offshore at the project location of the case Taman. As seen in figure 8.4 the exponential distribution approaches the Weibull fit quite good. For each bin this approximation is carried out, see appendix D. To run a Monte Carlo simulation the quantile function i.e. the inverse cumulative distribution function (F^{-1}) should be determined, see section 4.2.4. Since only the yearly extremes are used a threshold (ϵ) is applied for the wind speed. The derivation is as follows:

$$f(W_s) = \lambda \exp^{-\lambda \cdot (W_s - \epsilon)}$$
(8.1)

$$F(W_s) = \int_{-\infty}^{W_s} f(W_s) dW_s$$
(8.2)

The quantile function is derived by finding the value of W_s for a certain value of X. Where X is randomly picked between 0 and 1 in the Monte Carlo simulation.

$$W_s = F_{W_s}^{-1}(X)$$
(8.3)

$$W_{s}(X) = -\frac{1}{\lambda} ln(1-X) + \epsilon$$
(8.4)

As seen in figure 8.4a and 8.4c the values for the rate parameter (λ) and the starting point [i.e. threshold (ϵ)] are derived by fitting through the points of the Weibull distribution.



(a) Exponential distribution, Direction: N







(b) Weibull distribution, Direction: N



(d) Weibull distribution, Direction: S

Figure 8.4: Probability distributions for wind speed

Once the occurrence of the extreme wind speeds (figure 8.3) and the probability distributions (figure 8.4) for each bin are known, random values for the offshore wind can be derived via the Monte Carlo simulation. In figure 8.5 the procedure is given to determine the offshore wind speeds.

Simulating offshore wind speeds [m-file: Windspeed]	for i = 1:N
% Load winddata from 1992-2000 and apply threshold of 10.967 m/s Widata = ncep_time_ws_wr_1992_2000(ncep_time_ws_wr_1992_2000(:,2) < 10.967, .) = [];	x = rand(1,1);
M-file: Wind occurance INPUT "Winddata" OUTPUT "WindOcr"	% Datermine which bin is randomly selected via x for ii = 1:16 If WindOcr(ii:2) <x &&="" x<="WindOcr(ii+1,2)</td"></x>
Derive cdf occurance from pdf occurance	True
For i = 2:17	
WindOcr(1,2) = WindOcr(1,1); WindOcr(0,2) = WindOcr(0,1)+WindOcr(0-1,2);	X = rand(1,1);
	% Windspeed = -1/A ^ log(1-X) + € Windspeed(i,1) = -Windbins(2,0*log(1-X) + Windbins(1,0);
% Load exponential probability distribution each bin Windbins = textread(Windbins.bt)	
% Number of simulations (N) N = 30e4	

Figure 8.5: Matlab procedure to determine offshore wind speeds

At least 10^5 simulations are required to ensure an uniform distribution for a random selected variable by Matlab, see appendix E. In this procedure $3 \cdot 10^5$ simulations are carried out. In figure 8.6 the simulated wind speeds are given for the northern direction and omnidirectional. The wind speeds from the northern direction are clearly exponential distributed, a sharp edges is seen at the point of the value for the threshold (ϵ). The distribution for the omnidirectional wind speeds consist of 16 different exponential distributions, for each wind bin an unique distribution. For this reason the probability distribution is not an exact exponential distribution. However in the tail of the distribution the exponential character can be seen. In total $3 \cdot 10^5$ offshore wind speeds are generated which can be used to determine the nearshore conditions.



Figure 8.6: Probability distribution wind speeds

Bathymetry

In the semi-probabilistic approach which is used for the preliminary design a fixed value for the bottom level is chosen. However uncertainties are present in this value due to measurement errors and minor changes in the bottom profile. In the probabilistic design process these uncertainties are included via a probability distribution.

In the simplified case of Taman the bathymetry is based on a local hydrographical chart and a bathymetrical survey that has been carried out in 2011. The probability distribution of this parameter should be determined via the reliability of the applied methods and the difference in the survey and hydrographical chart. Determining the exact inherent uncertainty is too time consuming and not the scope of this research. Therefore it is assumed that the measurements errors result in a normal distribution for the bottom level with a standard deviation (σ) of 5 centimetre and a mean (μ) of -8 meter with respect to Baltic Datum (BD). As seen in figure 8.7 the normal distribution is approximate via the Monte Carlo simulation (N = $3 \cdot 10^5$).



Figure 8.7: Approximation of normal distribution via MC for bottom level

Currents

The currents offshore are determined by models and measurements, the values can for instance be obtained via NOAA [2016]. When a semi-probabilistic method is applied a single value is used in the wave model. Same as for the bathymetry uncertainties in the value of the currents occur due to measurement errors and changes in time. These uncertainties are dealt with in the probabilistic design process by describing the variable with a probabilistic distribution.

In the case of Taman the currents near the project location are very small and not taken into account in the wave model. Therefore no further investigation is carried out on how to determine the most suitable approach to process currents into the fully probabilistic calculation.

Water level

The existing water level consists usually of different components. The components which are present in the simplified case of Taman are listed below. In general these are also the most common components, see appendix C:

- · Sea level rise
- Tide
- Seasonal variations
- · Wave set-up
- · Storm surges

In the semi-probabilistic design method for all these components the most unfavourable value is taken depending on the type of failure mechanism. When the highest possible water level is the most unfavourable situation for a failure mechanism, all the upper limits are taken for the above listed components. For example for the sea level rise the most conservative predication is taken to be on the safe side. This results in combinations where the highest waves always occur during the highest water level. This is not an realistic situation and could lead to an overestimation of the failure probability.

Sea level rise, tidal and seasonal variations differ in time and do not depend on the direction and magnitude of the wind speed in the simplified case. For the tidal and seasonal variations an uniform distribution is assumed. This assumption is correct for the Black Sea, however for other project locations this could be incorrect. For instance locations where the tidal variations are asymmetric. The sea level rise depends on two uniform distributions, first of all the age of structure (between 0 and design lifetime) and the yearly sea level rise. By picking a random age of the structure and a constant yearly sea level rise, the total sea level rise at that moment in time is calculated by multiplying these two. In table 8.2 these distribution are given and the range of the possible values for these components.

Component	Value	Distribution	
Tide	+/- 0.05 m	Uniform	
Seasonal variations	+/- 0.1 m	Uniform	
Sea level rise:	0 - 0.25 m	[-]	
- Age of structure	0 - 50 years	Uniform	
- Yearly sea level rise	0 - 0.005 m	Uniform	

Table 8.2: Distributions of water level components

The wave set-up is determined with Goda [2000] and depends on the deep water wave steepness and the local water depth. However since the used design formulas are all empirical and wave set-up is a very local phenomenon, this is already included in the formulas. For this reason the wave set-up is not taken account or else it would be included twice.

Storm surges are usually calculated with a model, in the case of Taman this is Delft3D, and depends on the bathymetry, local geometry, wind direction and wind speed. Applying a model in this research would be too time consuming so a easier approach is applied. The relation between the wind speed and the storm surges is described with the basic equation for set-up, see equation 8.5 [The Rock Manual, 2007]. Where the set-up (*dh*) due to storm surges depends on the friction factor (c_w), density of the air and water ($\rho_{air,water}$), wind speed (u_w), water depth (*h*) and wave angle (ϕ).

$$dh = c_w \frac{\rho_{air}}{\rho_{water}} \frac{u_w^2}{gh} F \cdot \cos\phi$$
(8.5)

For simplicity this formula is reduced and a factor (f_{ss}) is implemented to describe the relation between the wind speed (u_w) and the set-up (dh), see equation 8.6. This factor is determined for each wind direction via the known wind speeds and set-up from the semi-probabilistic design process. Assumed is that this factor holds for all possible wind speeds.

$$dh = u_w^2 \cdot f_{ss} \tag{8.6}$$

The factor is defined for each direction. For the directions WNN till ESE this factor is negative (i.e. down surge) and for the directions ES till WN this factor is positive (i.e. up surge). Combining all the components (except wave set-up) this results in the following probability distribution for the water level, see figure 8.8.



Figure 8.8: Probability distribution of water level by Monte Carlo simulation

Two clear normal distributions can be distinguished due to the down surge for the northern wind directions and the upsurge for the southern wind directions. These normal distributions are not gradually merging into each other which would be a more realistic situation. This can be explained by the fact that this distribution is derived from the semi-probabilist data. No transitions is present between the northern (i.e. negative factor) and the southern (i.e. positive factor) directions. Figure 8.8 shows clearly that there is a correlation between the wind direction and water level.

Nearshore wave conditions

Once the offshore wind conditions, bathymetry and the water levels are known the nearshore wave conditions can be calculated. As mentioned using a model which converts these offshore conditions to nearshore conditions for N simulations takes to much time for this study. The rubble mound breakwater is located in the Black Sea which has a clearly defined fetch length and no swell waves are present, so the waves are wind driven. For this situation the formula of Brettschneider is used to calculated the nearshore significant wave height (H_s) [Schiereck, 2001], see equation:

$$\frac{gH_s}{u_w^2} = 0.283 \tanh\left[0.578 \left(\frac{gh}{u_w^2}\right)^{0.75}\right] \tanh\left[\frac{0.0125 \left(\frac{gF}{u_w^2}\right)^{0.42}}{\tanh\left[0.578 \left(\frac{gh}{u_w^2}\right)^{0.75}\right]}\right]$$
(8.7)

For each direction the fetch (F) and the water depth (h) is determined. The water depth for the fetch is determined via the depth chart of the Black Sea, see appendix F.1. The chosen water depth is the lowest point at the total fetch, for the northerly wind directions this is 7.9 meters and for the southerly wind directions 20 meters.

So for each randomly drawn wind direction and corresponding wind speed a wave height is calculated via Brettschneider. Nowhere in the process to determine the required hydraulic boundary conditions uncertainties are included for the wind speed, fetch length and water depth. Only $3 \cdot 10^5$ scenarios are examined. As mentioned in the introduction two types of uncertainties can appear in the probabilistic design process:

- · inherent uncertainties of the input parameters
- · model uncertainty

Inherent uncertainties of the input parameters is definitely present in the hydraulic boundary conditions and caused by measurements of the wind speed, water level and bathymetry. Furthermore model uncertainties are implemented due to the used simplification of reality by the Brettschneider formula. To take into account these uncertainties a factor (R) is applied over H_s as proposed by Van der Meer [1988]. This factor has a normal distribution with a mean (μ) and an assumed standard deviation (σ) of 0.025 (i.e. 2.5 %). To see if this assumed deviation is correct a comparison is made with a similar formula, namely the Young and Verhagen formula for wind waves in water of finite depth. Bart [2013] showed that the inaccuracy in this formula is in the order of 1%. So the estimated deviation of 2.5% for H_s seems realistic. Especially since the uncertainties in the wind speed (u_w) and fetch length (F) are also included in this deviation of 2.5%.

In figure 8.9a the wind speed is plotted against the significant wave height. The red line pictures the Monte Carlo simulations without this factor (R) and the blue illustrates the simulations including this factor. In figure 8.9b the Matlab procedure is given to determine the significant wave height with the Brettschneider formula including the model and inherent uncertainties of the input parameters.



(a) Wind speed plotted versus significant wave height. Direction: N.

(b) Procedure to determine significant wave height with Brettschneider

Figure 8.9: Example of failure probabilities and correlation for failure mechanisms M1-M4

As listed in table 8.1 a few more hydraulic boundary conditions should be determined. Given the fact that these are all correlated to the significant wave height, the procedure so far is sufficient to calculated all the required hydraulic boundary conditions. The procedure to calculated $H_{2\%}$, T_{m02} and $T_{m-1,0}$ is given in section 10.1.1.

In figure 8.10a the probability distribution for the wave heights in the northern direction are given. This probability distribution can be distinguished as an exponential distribution, which is expected since it depends on the exponential distribution for the offshore wind. In the whole Monte Carlo simulation $3 \cdot 10^5$ wave heights are calculated and picture in figure 8.10b. Each of this wave heights is examined in the probabilist calculation for the simplified case.



(a) Probability distribution significant wave height. Direction: N.

(b) Probability distribution significant wave height. Omnidirectional

Figure 8.10: Probability distribution of significant wave height

8.1.2. Geotechnical boundary conditions

Geotechnical boundary conditions are required to make a complete design. The structure and the designer of the structure are also not able to change these geotechnical boundary conditions, so it's referred to as environmental boundary conditions.

The boundary conditions which are needed depend on the used formulae and geotechnical models (e.g Plaxis and D-Geo Stability) to examine the failure mechanisms. The boundary conditions are determined on the bases of field tests (e.g. cone penetration test), laboratory tests and the Eurocode. Important is that the mean values are taken in the probabilistic design instead of the characteristic 95% values which are used in the semi-probabilist calculation (level I). If not, this result in a overestimation of the probability of failure.

Typical geotechnical boundary conditions are the characteristics of the subsoil such as the internal friction angle (ϕ), specific weight (γ) and the cohesion (c). A very specific but sometimes governing environmental geotechnical boundary condition is an earthquake. For several cases this boundary condition is the governing load and is seen as dynamic loading. In case of a semi-probabilistic design method partial factors for the soil parameters are applied for static loading and dynamic loading. When applying a fully probabilistic design method (level III) these partial factors are eliminated and the representative values of the soil are used.

In the simplified case only the slip stability is examined, see section 6.3. This slip circle is examined via the method of Bishop. In table 8.3 the input variables for this formula are given an the assumed distributions with their characteristics. The densities are given taking into account the porosity. The standard deviation for the geotechnical boundary conditions is estimated to be 2.5% of the mean, to take into account the inherent uncertainties of the input parameters.

Parameters	Symbol	Distribution	Mean (μ)	Deviation (σ)	Unit
Unsaturated density armour layer	$\gamma_{a_{un}}$	Normal	$17 \cdot 10^{3}$	425	$[N/m^2]$
Saturated density armour layer	γ_{a_s}	Normal	$20.5 \cdot 10^3$	512.5	$[N/m^2]$
Saturated density quarry run	γ_{q_s}	Normal	$16 \cdot 10^{3}$	400	$[N/m^2]$
Angle of internal friction armour layer	ϕ_a	Normal	40	1	[°]
Angle of internal friction quarry run	ϕ_q	Normal	40	1	[°]

Table 8.3: Geotechnical boundary conditions

8.1.3. Other boundary conditions

Besides the hydraulic and geotechnical boundary conditions, other boundary conditions could be required depending on the failure mechanisms for the designed structure. Examples are loads on the structure due to ice, ship collision and the influence of the environment on the strength on the structure (e.g. corrosion). It is not possible to appoint these conditions to one category so they are referred to as other boundary conditions, see the process scheme in appendix C.

These boundary conditions are not present in all cases however sometimes they exert large forces on the structure and can influence the design of the breakwater (e.g. ice loads Mennessier [2012]). In the simplified case of Taman no other boundary conditions than the hydraulic and geotechnical boundary conditions are present.

8.1.4. Independent variables

Besides the described boundary conditions in the above chapters independent variables are also part of the environmental boundary conditions. These variables can't be changed by the structure or the designer of the structure and are always present. For example the standard gravity and the density of the water. Independent variables often have a more or less fixed value, so in the probabilistic design method these variables or either fixed (i.e. deterministic) or have a very narrow probability distribution (i.e small deviation). In table 8.4 the independent variables for the simplified case are given. The assigned distributions and their characteristic are an estimation and are not further investigated in this research.

Parameters	Symbol	Distribution	Mean (μ)	Deviation (σ)	Unit
Density of the water	ρ_w	Normal	1014	5	$[kg/m^3]$
Gravitational acceleration	g	Deterministic	9.81	-	$[m/s^2]$

Table 8.4: Independent variables

8.2. Geometric boundary conditions

Additionally to the environmental boundary conditions, the geometric boundary conditions should be determined. Environmental boundary conditions are not set by the client or designer and can't be changed. The opposite holds for geometric boundary conditions, most of the time these are imposed by the design and functional requirements.

First of all the project area is defined where the structure can be realized. This gives limitations to the width and length of the structure from an area perspective. Furthermore design requirements impose boundaries, such as the maximum height to ensure enough sight for incoming and outgoing ships. At last the functional requirements can lay down boundaries, for instance when an inner slope of grass is applied the slope can't be to steep to make sure maintenance equipment can still ride over the inner slope.

Uncertainties in these conditions occur due to errors during realisation of the structure. The slope for instance could be slightly different constructed then designed. These small variations are taken into account in the probabilistic design method by a probability density function. The characteristics of these probability density functions are given in chapter 9 for variables such as the slope (α) and crest width (B).

In the simplified case the project location is already defined by the semi-probabilistic design. Assuming the probabilistic designed rubble mound breakwater is located at the same place no further geometric boundary conditions are present for the probabilistic design of the simplified case.

9

Preliminary design

In this chapter the preliminary design is determined which is used in the probabilistic design process, see figure 9.1. The preliminary design describes the broad lines of the system and determines the required components (e.g. armour layer, crest element). In this study the design conditions for the preliminary design are also used in the probabilistic design process. Since the focus of this research is on the probabilistic design process the preliminary design is derived from a existing case to save time. In this study the preliminary design is derived from the project Taman as explained in chapter 6. This design is made with a semi-probabilistic design method (level I) and is discussed briefly in this chapter. For a detailed description when to apply a rubble mound breakwater and how to design such a structure with a semi-probabilistic design method is referred to the book *Breakwaters and Closure Dams* by d' Angremond et al. [2012].



Figure 9.1: Indication of preliminary design in probabilistic design process

In conclusion the preliminary design is based on the semi-probabilistic design derived from the project Taman. This design is evaluated with a fully probabilistic design method (level III) in this study, the procedure is explained in chapter 11. In the next section an explanation is given of the different input variables for the design formulae. Subsequently the simplified case of Taman, which serves as preliminary design in this study, is checked on the basis of the guidelines given in the The Rock Manual [2007].

9.1. Background information

The preliminary design is made with a semi-probabilistic design method and is the basis and the first guideline for the probabilistic design (Level II and III). With this in mind it's wise to set up the preliminary design in such a way that it only has to be altered slightly to use in the probabilistic design method. In the probabilistic design various scenarios are examined and each variable is included in the design process with an own probabilistic distribution. The key concept here is to parametrize the rubble-mound breakwater and specify every variable. Castillo et al. [2004] defined five types of variables which are used in the probabilistic design process of a rubble mound breakwater:

- · Design or geometric variables
- · Parameters used in preliminary design
- · Random variables used only in probabilistic design
- · Parameters used in probabilistic design that define the random variability
- Non basic variables whose values can be obtained from those of the basic variables using formulae

Nearly all these variables can be changed and optimized within the limitations imposed by the boundary conditions. This is an important characteristic of these variables and is used during the iterative design process. First to make a correct design according to the given parameters and in a later stage during the optimization of the breakwater [Castillo et al., 2006].

The most important distinction between those different variables is the difference between basic variables and variables which can be obtained via the basic variables. This difference and the influence of these variables in the probabilistic design process is further explained in chapter 10.

The variables and parameters for the preliminary design are determined via the boundary conditions and design requirements. A semi-probabilistic calculation does not require probability distributions for each input variable. However in this chapter the probability distributions for some input variables are already given. This is only done for the variables were the probability distribution remains the same for each simulation (e.g. density of the water ρ_w). The significant wave height for example is derived separately for each Monte Carlo simulations and can not be described by a standard probability distribution, see chapter 8. The mean (μ) given in the tables is the value used for the semi-probabilistic design. And the type of distribution, mean (μ) and sigma (σ) are used in the fully probabilistic design. The uncertainties which are present in the complete design formula (so not per variable) due to the empirical derivation is explained in chapter 6.

9.2. Simplified case

Making use of a semi-probabilistic approach the most unfavourable situation is governing for the design formulae. The hydraulic boundary conditions are based on the design requirement that the structure should withstand a 1/100 year storm, see section 7.2. The storm with a 1/100 year return interval are processed in the model SWAN to determine the nearshore wave conditions, see chapter 8. SWAN generates nearshore wave data for directional bins of 22.5° of the wind, see figure 8.1. The wave conditions (i.e. hydraulic boundary conditions) for all wind directions are examined for section B of the rubble mound breakwater. The wind directional bin of 247.5° gives the maximum waves conditions, given in table 9.1. These wave conditions are the maximum conditions however it does not mean that these are governing for each failure mechanism. For instance combinations of water level and wave height from other directions could be governing. The governing situation for each design formula in the semi-probabilistic design is given in the next sections.

Variable	Symbol	Unit	Value
Design high water level	D.H.W.L.	[m+ BS]	1.2
Significant wave height	Hs	[m]	3.5
Peak wave period	Tp	[S]	14.4
Spectral wave period	$T_{m-1,0}$	[S]	8.4
Mean wave period	<i>T_{m0,2}</i>	[S]	5.1
Wave direction relative to North	β	[°N]	225

Table 9.1: Hydraulic boundary conditions, wind direction of 247.5
The semi-probabilistic design is already made for the project Taman, so in this chapter the design is only checked if it meets the guidelines as given in the The Rock Manual [2007]. In the following subsections the four different failure mechanisms, as described in section 6.3, are evaluated. For some variables the values remain the same for different scenarios and failure mechanisms, such as material properties and independent variables as described in chapter 8.1.4. These variables and their distribution (only used in fully probabilistic design [Level III]) are listed in table 9.2.

Variable	Symbol	Distribution	Mean (µ)	Deviation (σ)	Unit
Density of stone	ρ_s	Normal	1014	5	$[kg/m^3]$
Density of water	ρ_w	Normal	2600	15	$[kg/m^3]$
Standard gravity	g	Deterministic	9.81	-	$[m/s^2]$

Table 9.2: Fixed variables for all design formulae

Furthermore fixed standard deviations are applied (only in the fully probabilistic design) for variables which are used in multiple design formulae, see table 9.3. For the geometric variables (e.g. crest width (B), crest level (*cr*)) a standard deviation of 5% is applied. Because this design is examined before realisation, this standard deviation should be included to take into account the construction uncertainties. The distribution for the D_{n50} of the stone classes is based on the standard classes of rock grading (EN 13383) [CIRIA/CUR, 1991]. The standard deviation is estimated to be in the order of 5%. The standard deviation of the soil parameters is assumed to be in the order of 2.5%.

Variable	Symbol	Distribution	Mean (μ)	Deviation (σ)	Unit
Geometric parameters	B, α, cr, h_{toe}	Normal	-	5%	[-]
Nominal stone diameter	D_{n50}	Normal	-	5%	[m]
Soil parameters	γ, ϕ	Normal	-	2.5%	[-]

Table 9.3: Fixed standard deviations for variables applied in the design formulae

Because the design made in the project Taman is used for the preliminary design, two different solutions occur. First of all the design applied in the case of Taman (1) which is based on the 50% values for the design parameters (e.g. c_{pl}, c_s). These 50% are used because the design is tested with 3D physical model tests. Secondly the calculated design in this chapter (2) where the 95% values are used for the design parameters. For the calculated D_{n50} in this chapter two different situations can be distinguished. Namely, the calculated required D_{n50} (2a) and the required standard stone classes with the corresponding D_{n50} (2b). So a total of three possible designs for the given boundary conditions. For each failure mechanism, with the corresponding design formula, the solutions for these three conditions are given.

- 1. Design as applied in project Taman
- 2. Calculated design simplified case via the The Rock Manual [2007]
 - (a) Simplified case D_{n50}
 - (b) Stone class simplified case

9.2.1. Seaside armour layer

To see if the stability of the seaside armour layer is sufficient the formula of Van der Meer is used as described in section 6.4. The load on the armour layer are the waves with the characteristics as described in table 9.1. Sufficient strength of the seaside armour layer in the Van der Meer formula can be achieved by altering the nominal stone diameter D_{n50} and the slope of the armour layer (α_{sea}). In this case the seaside slope has a fixed values, so the variable which is calculated for the seaside armour layer is:

• Minimal required nominal stone diameter D_{n50}

The wave conditions given in table 9.1 are the extreme conditions for the 1/100 year storm. In figure 6.1 can be seen that section B is not perpendicular to the north. For waves which are approaching the breakwater from an angle (relative to the normal of the breakwater (θ)) the stability of the armour layer increases.

The influence of oblique wave attack is included in the design by a factor (f) on the stability number (N_s) by Wolters and van Gent [2010].

$$N_{s,\theta} = \frac{N_{s,\perp}}{f} \tag{9.1}$$

$$f = \cos(\theta)^X \tag{9.2}$$

As recommend by Wolters and van Gent [2010] for rock slope a value of X = 1.05 is taken in equation 9.2. The range of the method suggested by Wolters and van Gent [2010] is only applicable till 70°, for larger angles no further reduction is take into account. The standard deviation of the formula of Wolters is assumed to be $\sigma = 0.05$.

The damage factor (S_d) is set to 2 as defined in section 7.2. The armour layer is directly placed on the quarry run, so the structure has a notional permeability (P) of 0.5 [The Rock Manual, 2007]. All the different wind directions result in different hydraulic boundary conditions. These scenario's are all examined and for some the deep water formula applies and for other situations the shallow water formula has to be applied, as explained in section 6.4. The required coefficient for these formulae are given in table 6.1, for the preliminary design the 95% non-exceedance value (i.e. 5% limit) is used, see also subsection 4.2.1.

In the Van der Meer formula the wave height is part of the stability number $(N_s = \frac{H_s}{\Delta d_{n50}})$. To see the influence of the correction for oblique waves proposed by Wolters, the factor is applied to the wave height (H_{θ}) . In table 9.4 the most important input variables and the calculated D_{n50} are given. The values given for the wave and water level statistics are only applicable for the semi-probabilistic approach. In the fully probabilistic calculation these differ for each simulation.

Variable	Symbol	Distribution	Mean (μ)	Deviation (σ)	Unit
Significant wave height	H_{s}	-	3.42	-	[m]
Angle of incoming waves	θ	-	220	-	[°]
Correct H_s for oblique waves	H_{θ}	-	1.11	-	[m]
Angle of seaside slope	α_{sea}	Normal	22	1	[°]
Nominal stone diameter	D_{n50}	Normal	0.46	0.023	[°]

Table 9.4: Required stone size for seaside armour layer calculated with the formula of Van der Meer

Given this calculation the following solutions for the seaside armour layer are given for the three conditions.

	Condition	Stone class	D_{n50} (Mean μ) [m]	Deviation (σ) [m]
1	Project Taman	60-300 kg	0.38	0.019
2a	Simplified case D_{n50}	-	0.46	0.023
2b	Stone class simplified case	300-1000 kg	0.59	0.030

Table 9.5: Nominal stone diameters (D_{n50}) seaside armour layer for three different conditions

9.2.2. Rear-side slope stability

The required armour layer at the rear-side of the breakwater is calculated with the formula of Van Gent and Pozueta [The Rock Manual, 2007]. The total formula and all it's input variables are given in section 6.4. In this case the required strength of the rear-side slope is derived from the nominal stone diameter and the slope angle of the rear-side. Since the slope angle has a set value of 27° ($cot\alpha = 2$), the variable which is calculated for the rear-side armour layer stability with the formula of Van Gent and Pozueta is:

• Minimal required nominal stone diameter D_{n50}

For the rear-side slope stability is assumed that waves with an angle of more than 90° to the normal of the breakwater do not research the breakwater. The governing wave height with corresponding wave statistics within this 90 ° has an angle of attack of 70°, see table 9.6.

A reduction factor is applied for the waves which are approaching the breakwater not perpendicular. This correction is included in the formula of Van Gent and Pozueta in the reduction factor gamma (γ).

The maximum damage factor for the rear-side slope is the same as for the seaside slope, namely $S_d = 2$. The probability distributions are also given which are used in the fully probabilistic calculation. Variation in the roughness is caused by unreliable estimated of the roughness during the design calculation, a normal distribution is assumed with a standard deviation of 2.5%.

Variable	Symbol	Distribution	Mean (μ)	Deviation (σ)	Unit
Significant wave height	H_{s}	-	1.87	-	[m]
Angle of incoming waves	β	-	70	-	[°]
Rear-side slope	α_{rear}	Normal	27	1	[°]
Roughness of seaward slope	γ_f	Normal	0.55	0.014	[-]
Roughness at the crest	γ_{f-c}	Normal	1.0	0.025	[-]
Crest level seaside	crl	Normal	3.7	0.185	[m]
Crest level rear-side	cr _{lr}	Normal	2	0.1	[m]
Crest width	В	Normal	2.4	0.12	[m]
Required stone size	D_{n50}	Normal	0.149	0.0075	[m]

Table 9.6: Required stone size for rear-side slope calculated with the formula of Van Gent and Pozueta

In this formula no 50% or 95% non-exceedance values are present. For this reason the stone class for Taman (1) is the same as for the calculated situation in this section (2b). This results in the following solutions for the rear-side armour layer:

	Condition	Stone class	D_{n50} (Mean μ) [m]	Deviation (σ) [m]
1	Project Taman	5-40 kg	0.17	0.0085
2a	Simplified case D_{n50}	-	0.149	0.0075
2b	Stone class simplified case	5-40 kg	0.17	0.0085

Table 9.7: Nominal stone diameters (D_{n50}) rear-side armour layer for three different conditions

9.2.3. Stability of Toe

At both sides of the breakwater a rocky toe is placed to ensure the stability of the breakwater. The toe at the rear-side is not exposed to severe wave attack and the applied rock grading is the same as for the rear-side armour layer (5-40 kg). This toe is not further examined and assumed is that it has no contribution to the failure probability of the breakwater in the simplified case. For the toe at the seaside of the breakwater the required armour grading is calculated with the Van de Meer formula for toe stability [The Rock Manual, 2007], described in further detail in section 6.4. The resulting armour grading is derived from the minimal required D_{n50} , so the variable which is calculated for the toe is:

• Minimal required nominal stone diameter D_{n50}

The reduction factor of Galland [1994], given in equation 9.3, is applied for waves that approach the breakwater under an angle. This factor leads to a reduction of the wave height (H_{s_c}) which is used in the formula of Van der Meer for the toe stability. The range of the method is till 45°, assumed is that larger wave angles cause no further reduction of the wave height. The value for x in equation 9.3 is 0.6 for toe stability with quarry stone, as given by Galland. The standard deviation for this formula is assumed to be $\sigma = 0.03$.

$$H_{s_c} = H_s \cos(\beta)^x \tag{9.3}$$

The design requirement for the toe is that the damage factor (N_{od}) has a value of 0.5, see section 7.2.

Variable	Symbol	Distribution	Mean (µ)	Deviation (σ)	Unit
Significant wave height	H_s	-	3.45	-	[m]
Angle of incident waves	β	-	112	-	[°]
Corrected wave height	(H_{s_c})	-	2.81	-	[m]
Toe berm level	[-]	Normal	-5.7	0.285	[m +BS]
Required stone size	<i>D</i> _{<i>n</i>50}	Normal	0.41	0.02	[m]

Table 9.8: Required stone size for seaside toe calculated with the Van der Meer formula

Given this calculation the following solutions for the toe are given for the three conditions.

	Condition	Stone class	D_{n50} (Mean μ) [m]	Deviation (σ) [m]
1	Project Taman	60-300 kg	0.38	0.019
2a	Simplified case D_{n50}	-	0.41	0.02
2b	Stone class simplified case	300-1000 kg	0.59	0.030

Table 9.9: Nominal stone diameters (D_{n50}) toe for three different conditions

9.2.4. Macro stability

In the original design of the case Taman the model D-Geo Stability is used to examine the macro stability. The model D-Geo Stability gives for the most unfavourable slip circle for section B in the case Taman a stability number of F = 1.69. Applying this model would be too time consuming to apply in this study, so an approximation for the simplified case is made by only examining the governing slip circle. The seaside slope (α_{sea}) of the breakwater should ensure the minimal required stability (F>1). For the seaside armour layer the slope is set to 22°. So in this calculation is checked if the stability factor (F) for macro stability is larger than 1 with the formula of Bishop, for the following condition:

• Seaside slope α_{sea} of 22 degrees

In table 9.10 the input variables are displayed and the corresponding probability distribution used in the full probabilistic calculation are also given. At the bottom the calculated stability factor (F) is displayed for the situation where only the governing slip circle is examined without the model D-Geo Stability. The examined slip circle for the macro stability in the simplified case is given in figure 9.2.



Figure 9.2: Examined slip circle for macro stability with the formula of Bishop

Variable	Symbol	Distribution	Mean (µ)	Deviation (σ)	Unit
Unsaturated density armour layer	γunsat−a	Normal	11500	285	$[N/m^{3}]$
Saturated density armour layer	γsat−a	Normal	17000	425	$[N/m^{3}]$
Saturated density quarry run	γsat−q	Normal	20500	515	$[N/m^{3}]$
Angle of internal friction armour layer	ϕ_a	Normal	40	1	[°]
Angle of internal friction quarry run	ϕ_q	Normal	40	1	[°]
Cohesion armour layer	С	Deterministic	0	-	[kPa]
Cohesion quarry run	С	Deterministic	0	-	[kPa]
Stability factor	F	-	1.67		[-]

Table 9.10: Input variables for Bishop formula to examine macro stability

The calculated stability number is F = 1.67, which is approximately the same as the stability number calculated by the model D-Geo Stability (F = 1.69). It shows that the macro stability is sufficient for this design.

The results for Taman (1) and the simplified case calculated this section (2) are the same. For both conditions a seaside slope of 22° is applied to ensure macro stability, see table 9.11.

	Condition	Seaside slope α_{sea} (Mean μ) [°]	Deviation (σ) [°]	Stability factor (F)
1	Project Taman	22	1	1.69
2	Simplified case	22	1	1.67

Table 9.11: Angle of seaside slope (α_{sea} to ensure macro stability

9.3. Final preliminary design

In this chapter four different failure mechanisms (i.e. components of the breakwater) are investigated with design formulae from the The Rock Manual [2007]:

- Seaside armour stability Van der Meer formula (very shallow water by Van Gent et al. [2004])
- Rear-side armour stability Van Gent and Pozueta formula
- Toe stability Van der Meer formula
- Macro stability Bishop formula

Three different situations are distinguished:

- 1. Design as applied in project Taman
- 2. Calculated design simplified case via the The Rock Manual [2007]
 - (a) Simplified case D_{n50}
 - (b) Stone class simplified case

An overview of the results are given in table 9.12.

	Condition	Seaside armour	Rear-side armour	Toe	Seaside slope	Rear-side slope
Condition		$D_{n50} [{ m m}]$	<i>D_{n50}</i> [m]	<i>D</i> _{<i>n</i>50} [m]	$[\cot(\alpha_{sea})]$	$[\cot(\alpha_{rear})]$
1	Project Taman	0.38	0.17	0.38	2.5	2
2a	Simplified case	0.46	0.149	0.41	2.5	2
2b	Stone class	0.59	0.17	0.59	2.5	2

Table 9.12: Nominal stone diameters (D_{n50}) toe for three different conditions

In figure 9.3 the preliminary design (level I) of the rubble mound breakwater in the simplified case is given with the standard stone classes [The Rock Manual, 2007], which is condition 2b (table 9.12).



Figure 9.3: Final preliminary design of simplified case

10

Correlations

In the design process boundary conditions, variables and failure mechanisms can be correlated. Two different types of correlation can occur in the design process as mentioned in section 1.2:

- **Statistical correlation** Describing reality with a model requires gathering, processing and interpreting a lot of data. Statistical correlation describes the dependency between variables and datasets, which is the result of common usage or derivation of the same basic principles.
- **Physical correlation** This type of correlation occurs when multiple processes interact. For instance wind driven waves and wind driven up or down surge of the water level. Furthermore a rubble mound breakwater consists of multiple components. All these components are subject to changes and influence each other. For example smoothing out of toe could result in a less stable armour layer. These processes are defined as physical correlation.

In this chapter an explanation and a solution is given on how to deal with these types of correlation in the fully probabilistic (level III) design process . First all different aspects on which statistical correlation can occur are discussed and subsequently the physical correlation is explained in further detail.

10.1. Statistical correlation

Statistical correlation occurs when variables or failure mechanisms are depended and interact with each other. As concluded from the total process diagram given in appendix C statistical correlation appears on three different points in the probabilistic design process:

- Boundary conditions
- Failure mechanisms
- Multiple sections

These different points are discussed and the consequences of the occurring statistical correlation are given.

10.1.1. Boundary conditions

Statistical correlation in the boundary conditions appears due to the depended variables. A clear example are the hydraulic boundary conditions generated via models such as SWAN [The SWAN Team, 2006]. Each simulation of a model gives a set of hydraulic boundary conditions as output. These output variables such as the wave height (H_s), mean wave period ($T_{m0,2}$) and mean energy wave period ($T_{m-1,0}$) are correlated via the energy density spectrum [Holthuijsen, 2007]. This correlation should be taken into account and implement in the probabilistic calculation procedure. For a certain value of $T_{m0,2}$ only a number of values for H_s are possible. So one should not make the mistake to pick the largest wave period with the largest significant wave height as design condition when this is not possible given the spectra generated from the model.

When a model is applied the solution is quite simple by just keeping the output variables of each simulation together. In the simplified case however no model is used and the wave height is calculated with the formula of Brettschneider, see section 8.1.1. To ensure correct representation of reality the correlation should be implemented in the probabilistic calculation. This is done by describing the correlation with an factor, as given in table 10.1 for the northerly direction.

Variable	Variable	Correlation factor	Distribution	Mean (µ)	Deviation (σ)
H_s	$H_{2\%}$	$H_{2\%}/H_{s}$	Normal	1.30	0.05
H_{s}	<i>T</i> _{m0,2}	Steepness (s)	Normal	0.12	0.02
$T_{m0,2}$	$T_{m-1,0}$	$T_{m0,2}/T_{m-1,0}$	Normal	0.74	0.03

Table 10.1: Factors to described hydraulic boundary conditions for northerly direction (0°)

These factors are determined on the basis of the data from the semi-probabilistic design. By examining all the factors in all directions and taking the mean (μ) of these values and calculating the standard deviation (σ), see equation 10.1, a normal distribution is created. Values which are larger or smaller than $\mu \pm \sigma$ are neglected and a new mean and standard deviation is calculated.

$$\mu = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \sigma = \sqrt{\frac{1}{N} \sum_{i=1}^{N} (x_i - \mu)^2}$$
(10.1)

The ratio between H_s and $H_{2\%}$ has in deep water a fixed value of 1.4. However on shallow foreshores the wave height distribution changes [Battjes and Groenendijk, 2000]. Assuming a Weibull distribution this value is between 1.2 and 1.4. In the simplified case this values is assumed to be the same for all wind directions. The ratio between H_s and $T_{m0,2}$ is described with the steepness (s), see equation 10.2. Due to the large variations between each directional wind bin this factor is determined separately for each bin. The same is done for the ratio between $T_{m0,2}$ and $T_{m-1,0}$, this value should be in the order of 0.92 [Verhagen et al., 2008]. For the northern direction these values are given in table 10.1. For the other directions the factors are given in appendix F2.

$$s = \frac{H_s}{L_0} = \frac{H_s \cdot 2\pi}{g T_{m0.2}^2}$$
(10.2)

To determine the 2% wave height ($H_{2\%}$) first the significant wave height is calculated (H_s) and subsequently a random value out the normal distribution given in table 10.1 is drawn. This Matlab procedure is given in figure 10.1b, note that this is part of the total Monte Carlo simulation. The results for $H_{2\%}$ for all directions is pictured in figure 10.1a.



Determining H2%	
For x = i Where i = 1 : N	
Pick dir_w subsequently u_w M-file: wind direction, wind speed Output "dir_w(i),u_w(i)"	
Determine F for Wind direction (dir_w(i))	
Calculated Hs(u_w,dir_w,F) with Brettschneider form	ula
Pick factor (b) H2%/Hs from normal distribution	
Calculated H2%(i) = Hs(i) * b(i)	

(a) 2% wave height $(H_{2\%})$ against significant wave height $(H_s).$

(b) Matlab procedure to calculated $H_{2\%}$

Figure 10.1: Statistical correlation between $H_{2\%}$ and H_s

Statistical correlation in the geotechnical boundary conditions are in the simplified case caused by the type of soil. Each soil has it's own characteristic and should not be interchanged. This is straight forward and not as complex as for the hydraulic boundary conditions. Therefore it is not further explained in this chapter.

10.1.2. Failure mechanisms

A rubble mound breakwater can fail due to multiple failure mechanisms, see section 3.4. Between these failure mechanisms interaction and correlation can exist and this has influence on the probability of failure of the total system. In this section, first a single failure mechanism is considered and subsequently a system with multiple failure mechanisms is examined.

Single failure mechanism

The first option is the situation where only one failure mechanism is present. This is not a realistic situation for a rubble mound breakwater, however it is a good starting point to explain the situation where multiple failure mechanisms are present.

In a single failure mechanism statistical correlation can occur due to dependent input variables. To determine if the input variables are dependent the values should be derived to the basic variables (H_s , $H_{2\%}T_{m0,2}$, *etc.*). The statistical correlation between the basic variables is given in the previous section and implemented in the probabilistic design process.

If variables are statistical correlated due to the same basic variables this should be taken into account. To illustrate this correlation the Van der Meer formula for the stability of the armour layer is considered.

$$\frac{H_s}{\Delta d_{n50}} = c_{pl} P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$
(10.3)

The number of waves (N) depends on the duration of the storm (h_{storm}) and the mean wave period ($T_{m0,2}$).

$$N = \frac{h_{storm}}{T_{m0,2}} \tag{10.4}$$

In addition the Iribarren number ξ_m , see 10.5, also depends on the mean wave period. During the Monte Carlo simulation it is important that those variables are derived via the basic variables. If a probability distribution is applied for the number of waves and for the Iribarren number, indirectly different values are used for the mean wave period. This results in a incorrect probabilistic calculation. So the Iribarren number should be derived via α , H_s , g and $T_{m0,2}$ i.e. the basic variables.

$$\xi_m = \tan \alpha / \sqrt{2\pi H_s / (gT_{m0,2}^2)}$$
(10.5)

Below an overview is given of all the input variables for the design formulas which should be derived to the basic variables to take into account the statistical correlation. All the design formulae are given in section 6.4. In table 10.2 is referred to the corresponding breakwater component for which the design formula is applied. Only basic variables which cause statistical correlation by occurring multiple times in input variables are listed in this table. For example the storm duration is not included since it is only required to calculated the number of waves. In the formula of Bishop for macro stability no dependent input variables are present.

Variables		Design formula		
Input variable	Basic variables	Armour -	Rear-side armour -	Toe -
		Van der Meer	Van Gent and Pozueta	Van der Meer
N	$T_{m0,2}$	X	Х	-
ξ_m	<i>H</i> _s , <i>T</i> _{m0,2} , g	Х	-	-
ξcr	c_{pl}, c_s, P, α	Х	-	-
$\xi_{s-1,0}$	$H_s, T_{m-1,0}, \alpha, g$	Х	Х	-
Δ	ρ_s, ρ_w	Х	Х	Х

Table 10.2: Basic variables derived from the input variables with corresponding design formula

In conclusion statistical correlation for a single failure mechanisms can occur due to variables which are depended. In the semi-probabilistic design method usually this does not cause any problems. Since only one set of boundary conditions is picked as the governing scenario. In the probabilistic design process on the other hand multiple scenarios are examined and variables of one scenario should not be mixed with variables of an other scenario.

Multiple failures mechanisms

A more realistic situation is a system which consists of multiple failure mechanisms in one cross-section. In the semi-probabilistic (level I) design method failure mechanisms are examined separately. So only for the probabilistic (level II and III) design process correlation due to multiple failure mechanisms occurs. The mentioned statistical correlation which occurs at a single failure mechanisms is also present when there are multiple failure mechanisms.

To calculated the total probability of failure of the system, the relation and correlation of the failure mechanisms should be known. First the limit state function for each failure mechanisms needs to be determined as in chapter 6. Next the relation between the failure mechanisms is described (a parallel or series system). Furthermore each failure mechanism should be examined to see if there is any correlation with other failure mechanisms. These concepts will be explained on the basis of the fault tree of the imaginary system shown in figure 10.2.



Figure 10.2: Fault tree imaginary system

As mentioned above the first step is to described the relation between the failure mechanisms M1 and M2. Failure mechanisms are related in series or in parallel. [PIANC, 1992]

In a series system if any of the failure mechanisms (M = 1, 2, ..., n) fails it results in failure of the top event, see figure 10.2.



Figure 10.3: Series system

In figure 10.2 a series system correspondents with an or-gate. The probability of failure of the top event for a series system P_f^s depends on the level of correlation between the failure mechanisms. The upper bound implies full correlation and the lower bound corresponds with no correlation at all. [PIANC, 1992]

Lower bound
$$P_f^s = max P_f^M$$
 (10.6)
Upper bound $P_f^s = 1 - (1 - P_f^1)(1 - P_f^2)...(1 - P_f^n)$ (10.7)



Figure 10.4: Parallel system

In a parallel system failure of the top event only occurs when all the failure mechanisms fail, see figure 10.4. As seen in 10.4 all the mechanisms (M1, M2, ..., Mn) should fail when the mechanisms are in a parallel system to induce failure of the top event, this is displayed by an and-gate (figure 10.2).

The lower and upper bounds for the failure probability of the system are given below and corresponding to no correlation respectively full correlation. [PIANC, 1992]

Lower bound	$P_f^s = P_f^1 \cdot P_f^2 \dots P_f^n$	$\lim_{n\to\infty} P_f^s \approx 0$	(10.8)
Upper bound	$P_f^s = min P_f^M$		(10.9)

When the upper en lower bounds of the system are known, a first estimate of the total failure probability of the system can be made, as picture in figure 10.5.



Figure 10.5: Probability of failure for series and parallel systems [Schiereck, 2001]

To determine the exact failure probability of the system the correlation between the failure mechanisms should be determined. Failure mechanisms can be correlated in two ways [Campos et al., 2011; PIANC, 1992]:

Statistical correlation due to common parameters (e.g. *H_s*)

Physical correlation, slight changes in a component can decrease the reliability of other components or failure of one mechanism induces failure of an other mechanism.

Only the statistical correlation is examined in this section, however for simplicity the situation where one failure mechanism induces the other (physical correlation) is also taken in account in the following example. The situation where one component does not fail completely but decreases the reliability of a component is explained in the next section.

Given the fact that failure mechanisms can be correlated, the system, given in figure 10.2, can have six different failure events including the option of no failure at all [Campos et al., 2011]:

• No failure (N_0)

• Failure of $M_2(N_{2,X})$

- Failure of $M_1(N_{1,X})$
- Failure of M_1 induces failure of M_2 ($N_{1,2,X}$)
- Failure of M_2 induces failure of M_1 ($N_{2,1,X}$)
- Failure of M_1 and M_2 simultaneously (N_{12})

This can be pictured a scheme, see figure 10.6 where N is the possible event and X symbolize a stable situation (no more failure mechanisms are developed).



Figure 10.6: Fault tree of system with two failure mechanisms [Campos et al., 2011]

In figure 12.3 the influence of the correlation on the total system reliability is pictured.

Case	Mutually exclusive	Independent	Dependent
Correlation coefficient. $\rho_{Z1,Z2} =$	-1	0	1
Venn diagram	F1 F2	F1 F2	F1, F2
System failure probability $P(F)$	$P(F_1) + P(F_2)$	$P(\mathbf{F}_1) + P(F_2) - P(F_1) \cdot \mathbf{P}(\mathbf{F}_2)$	$Max(P(F_1, P(F_2)))$

Figure 10.7: Influence of correlation on total system failure probability of a series system [Jonkman et al., 2015].

As can be concluded from figure 12.3 a stronger correlation between the failure mechanisms results in a lower total failure probability of the system. Furthermore the relation of the failure mechanisms has a influences on the total failure probability of the system, see figure 10.5. So, to determine the total failure probability of a breakwater system the relation (parallel or series), statistical correlation and physical correlation between all the failure mechanisms should be known.

Simplified case

In the simplified case the statistical correlation and physical correlation where one failure mechanism induces failure of the other is implemented as follows. The first step is to determine if it is a series or parallel system. The simplified case is a series system, if one of the failure mechanisms fails the whole system fails.

In theory the probability of failure for each mechanism can be calculated. To get the total failure probability, these failure probabilities are combined considering the correlation parameters. These are the steps 7 till 9 in figure 5.2. However in practise this is quite difficult since the correlation parameters should be determined. These correlation parameters described the statistical correlation that appears in the process, see figure 10.8.



Figure 10.8: Statistical correlation in the probabilistic design process

The solution for this problem is to make one Monte Carlo simulation for the whole fully probabilistic calculation. Part of this procedure is given in the process structure diagram in figure 10.9. For each simulation made with the Monte Carlo procedure the reliability function is examined. If failure occurs this results in a 1 and no failure is represented with a 0. In the simplified case four different failure mechanisms are examined, so four different reliability functions (figure 10.9b). If one of those fails the complete system fails which is also included in the Matlab procedure in figure 10.9a.



(a) Calculating system reliability

(b) Examining if failure occurs for each reliability functions

Figure 10.9: Example of failure probabilities and correlation for failure mechanisms M1-M4

For each simulation is checked if one or more failure mechanisms fail. In table 10.3 a few possible outcomes of the Monte Carlo simulations are given and the conclusion if the system fails or not. The total reliability of the system is calculated by dividing the failed simulations by the total number of simulations (section 4.2.4).

Simulation number	Seaside armour	Rear-side armour	Toe	Macro stability	Total system
1	0	0	1	0	1
2	1	1	0	0	1
3	1	0	1	1	1
	-	-	-	-	-
N	0	0	0	0	0

Table 10.3: Physical correlation multiple failure mechanisms

With this method the statistical and part of the physical correlation is considered. Regarding the statistical correlation all the possible scenarios are taken into account. The situation where failure of one failure mechanisms results in failure of an other mechanism (i.e. physical correlation) is indirectly taken into account. This relation between failure mechanisms is not described with a formula or correlation parameter in the fully probabilistic calculation. Because the system is characterized as a series system it does not matter if one or multiple failure mechanisms fail for one simulation, this result in both cases in failure. If for example the rear-side armour in simulation number 3 fails in reality because failure of the seaside armour results in failure for the 3th simulation so there is no difference in the total reliability of the structure. In conclusion the following scenario's are taken into account for each simulation via the Monte Carlo simulation (0 = no failure, 1 = failure):

- No failure (0)
- Failure of one of the failure mechanisms (1)
- Failure of multiple failure mechanisms at the same time (1)
- Failure of a failure mechanism induces failure of an other failure mechanisms (1)

10.1.3. Multiple sections

In the previous section the situation for multiple failure mechanisms and the corresponding probability of failure is discussed. These correlations between failure mechanisms are assumed to be in a single cross sections. However a breakwater is a three-dimensional structure and failure can occur along the entire length of the structure. This length effect can be seen as a correlation between different sections of the system.

Not much is written about the influence of this length effect on the total probability of failure of breakwaters. In the Netherlands a method is developed to take in account this length effect when designing a dike [Rijk-swaterstaat, 2014b]. This is for only one failure mechanism and not for the whole system. So this method can not be directly applied to determine the probability of failure for a breakwater system. However it is of good use to get a first idea of the influence of the length effect and to show the influence on the probability of failure of the structure.

In the proposed method of Rijkswaterstaat [2014b] the maximum acceptable probability of failure (P_{max}) for the whole structure is seen as a failure space. This failure space is distributed among the different failure mechanisms. To determine the allowable failure probability (P_M) per mechanisms the failure space factor (ω) and the length-effect factor (N) should be determined, see equation 10.10.

$$P_M = \frac{P_{max} \cdot \omega}{N} \tag{10.10}$$

Here the focus will only be on the length-effect factor (*N*), for further explanation and values of the space factor (ω) is referred to Rijkswaterstaat [2014b].

The length-effect factor depends on the fraction of the length of the section where the failure mechanism is present(*a*), the total section length ($L_{section}$) and the length of the independent, equivalent parts (*b*). This results in the equation 10.11.

$$N = 1 + \frac{a \cdot L_{section}}{b} \tag{10.11}$$

As seen from equation 10.10 and 10.11 an increase of the length where the failure mechanism is present $(a \cdot L_{section})$ results in a lower acceptable probability of failure for a failure mechanism (P_M) . Or in other words the failure probability of the structure increases with an increasing length $(L_{section})$.

As mentioned before this method can't be one on one applied for breakwaters however it shows the effect of an increase in length on the probability of failure for a dike. Although the length effect will have effect on the total probability of failure of the breakwater in the simplified case it is not taken into account in this study.

10.2. Physical correlation

Physical correlation occurs when processes and failure mechanisms interact. Physical correlation can occur on two aspects in the design process:

- · Boundary conditions
- Failure mechanisms

10.2.1. Boundary conditions

Physical correlation occurs for the boundary conditions due to processes which interact and influence each other. For instance the wind is resulting in rise of the water level and causes wind driven waves [Holthuijsen, 2007]. The risk is that processes are combined which can not occur in reality.

A good example are the offshore wind and water level conditions to determine the hydraulic boundary conditions. Within this data correlation exist between directions and wind speed. In the probabilistic calculation for the simplified case each direction has a different probability distribution for the wind speed as pictured in appendix D. In the simplified case a clear correlation is seen between the direction and the occurring wind speeds, see figure 10.10a. Each direction gives an other set of possible wind speeds.



Figure 10.10: Physical correlation in boundary conditions

Besides the correlation in the wind speed and direction, often a correlation exists between water levels and wave heights. Combining each possible water level with a possible wave height would result in unrealistic combinations. In figure 10.10b the water level versus the wave height is pictured. Each blue circle is one Monte Carlo simulation ($N = 3 \cdot 10^5$). As seen for low water levels only wave height up to 3 meters occur. This is the opposite of what is logical when looking at figure 10.10a. From directions 45 and 67.5 degrees high wind speeds are present and these winds also result in a down surge, see figure 8.8. However the waves are wind driven and the fetch length for the northerly directions is significantly smaller than for the southerly

directions. This explains why the highest waves occur for the southern direction and highest water levels.

This example shows that physical correlation exists in the boundary conditions of the simplified case. Neglecting these correlations could result in unrealistic simulations and a incorrect determination of the reliability of the rubble mound breakwater. In the simplified case this physical correlation is taken into account by creating a table with possible hydraulic boundary conditions and keeping each simulation in one line, see table 10.4. Each set of conditions is examined for all the failure mechanisms. In the simplified case the number of simulations (N) is $3 \cdot 10^5$.

Simulation number	Wind direction [°]	Wind speed [m/s]	Wave height [m]	Wave direction [°]
1	202.5	18.3	2.38	217
	-	-	-	-
Ν	0	13.2	0.95	342

Table 10.4: Hydraulic boundary conditions as generated by the Monte Carlo simulation

10.2.2. Failure mechanisms

Physical correlation can occur in the failure mechanisms due to the following:

- 1. Failure of mechanism (M1) induces failure of other failure mechanisms (M2) (e.g. failure of the armour layer results in failure of the concrete element)
- 2. Slight changes in a component (i.e. failure mechanism) can decrease the reliability of other components. A few examples are given [Maciñeira, 2016]:
 - If the toe is damaged (but does not fail), the failure probability of the armour layer could increase.
 - If the armour has an specific level of damage (but no failure yet), the concrete wall on top of the breakwater screen is more exposed to wave action.
 - As the roughness of the sea side armour layer decreases, overtopping increases.

The first scenario is already explained in the previous section and taken into account via a Monte Carlo simulation. The second situation is more complex. The simplified case is used as an example to illustrate this.

- Failure due to instability of rear-side armour slope (M1)
- Failure due to instability of seaside armour slope (M2)
- Failure due to instability of toe at sea side (M3)
- Failure due to slip of subsoil (M4)

For simplicity the failure mechanisms are assumed to have the same failure probability and are all correlated and independent, see figure 10.11a.

In the simplified case the toe (M3) fails if the damage factor N_{od} reaches 5 or more. When for instance some flattening out of the toe occurs, which corresponds with $N_{od} = 2$, the toe remains intact according to the set definition of failure $N_{od} = 5$. However some flattening out of the toe could result in an seaside armour layer which is more unstable, because the seaside armour rests on the toe. In other words an increase of the failure probability of the seaside armour layer (M2). The relation between the toe and the seaside armour stability could be described as follows:

If the toe is partly damaged (e.g. $N_{od} = 2$), no failure occurs according to the maximal allowable damage ($N_{od} = 5$). However the probability of failure for the seaside armour layer increases due to a less stable armour layer.

This is illustrated by increasing the circle of the failure probability of the seaside armour (M2) as given in figure 10.11b. In figure 10.11a the failure spaces for the initial situation ($N_{od} = 0$) are given and in figure 10.11b the situation is given where $N_{od} = 5$.



Figure 10.11: Example of increase in failure probability of a failure mechanism (M2) due to partly failure of an other mechanism (M3)

This example shows that physical correlation between failure mechanisms could result in an increase of the failure probability of the system. Assumed is that this type of physical correlation has no affect in the simplified case. In other cases however this could have a significant effect on the total reliability of the structure.

Possible solutions to include physical correlation

Although it is assumed that in the simplified case the above described type of physical correlation does not occur, a possible solution is given in this paragraph. Damage to the toe which influences the armour layer stability is used as an example.

The main issue for this problem is that the Monte Carlo simulation only makes a distinction between non failure and failure for the toe, based on the damage factor N_{od} . All values for $N_{od} < 5$ result in non-failure (0) and the values $N_{od} \ge 5$ result in failure (1).

In reality values for $2 < N_{od} < 5$ result in some smoothing of the toe and this has an effect on the armour layer stability. This should be implemented in the calculation. The Van der Meer formula for the armour layer is as follows:

$$\frac{H_s}{\Delta d_{n50}} = c_{pl,d} \ P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{10.12}$$

Two possible solutions are described to include the physical correlation between toe toe and the armour layer. The described methods require that first the toe stability is checked and subsequently the armour stability:

• Lowering the stability number $Stn = \frac{H_s}{\Delta d_{n50}}$ for the armour layer.

This could be seen as the opposite of increasing the stability of the armour layer for oblique waves [Galland, 1994; Wolters and van Gent, 2010]. If $N_{od} < 5$ a reduction factor $f_c > 1$ is applied for the stability number (*Stn*) in the Van der Meer formula. This results in a corrected stability number (*Stn*_c), which means that the armour layer fails for example for a lower wave height than in the initial situation. The procedure how it should be applied in Matlab is given in figure 10.12a.

$$Stn_c = \frac{H_s}{\Delta d_{n50}} \cdot f_c \tag{10.13}$$

• Lowering the allowable damage factor *S*_d for the armour layer.

An other possibility is to lower the maximal allowable damage S_d for the armour layer for $2 < N_{od} < 5$. The armour layer fails if $S_d \ge 8$ in the simplified case. By lowering the maximal allowable damage to for example $S_d = 6$ the armour layer will fail for smaller loads. The procedure how it should be applied in Matlab is given in figure 10.12b.

Physical correlation between toe and seaside armour stability		Physical correlation between toe and seaside armour stability
For x = i		For x = i
Toe stability - Van der Meer		Toe stability - Van der Meer
If angle_att < 45	False	If angle_att < 45
angle_att_t= angle_att	angle_att_t = 45	angle_att_t= angle_att_t= 45
Calculate Hs_cr (Significant wave height reducted for oblique waves b	oy wolters)	Calculate Hs_cr (Significant wave height reducted for oblique waves by wolters)
Calculate reliability function gives: N_od		Calculate reliability function gives: N_od
If N_od < 5 True	False	If N_od < 5 True False
No failure = 0 If N_od < 2	Failure = 1	No failure = 0 If N od < 2
True Fals f_c = 1 f_c (N_od) = 1 + N_od *	a	True False S_d = S_d S_d (N_od) = S_d - N_od * b
Calculate seaside armour stability with correct stability number (Stn_c Stn_c = H_s/(Delta ^ d_n50) ^ f_c	:):	Calculate seaside armour stability with S_d
(a) Via corrected stability number (St	(n_c)	(b) Via reduced damage factor (S_d)

Figure 10.12: Procedure to included physical correlation between toe and seaside armour stability

These are two possible ways to take into account the physical correlation between failure mechanisms. The value for the reduction factor a and the reduction in the damage number b should be determined with physical model tests and depends on the level of damage of the toe N_{od} . This example shows that including this type of physical correlation in the fully probabilistic (level III) calculation is possible. Further investigation is necessary to determine the correction values (e.g. a and b) between failure mechanisms.

11

Probabilistic calculation

This chapter gives the complete procedure to carry out a fully probabilistic calculation for the simplified case of Taman in practise. In the previous chapters several steps are already taken and the full probabilistic calculation (level III) is almost the last step in the design process, see figure 11.1. The results of this probabilistic calculation are given in the chapter 12.



Figure 11.1: Indication of design requirements in probabilistic design process

Before the actual fully probabilistic calculation is made a lot of preparation is required. This is done in the previous chapters, a short overview is given below with steps are taken in the probabilistic design process as given in the flowchart in figure 5.2.

- 1. Required boundary conditions are determined (Step 1)
- 2. Design requirements for Ultimate Limit State (ULS) are determined (Step 2)
- 3. Preliminary design to determine structure properties (Step 3)
- 4. Fault tree with relevant failure mechanisms and interaction (series or parallel) and correlation (where does correlation occur) (Step 4 and 8)
- 5. Reliability functions are derived (Step 5)
- 6. Probability distribution for all input parameters are determined (Step 6)

The steps which still have to be carried out are:

- 1. Determine probability failure for each failure mechanism (Step 7)
- 2. Calculate probability of failure total system (Step 9)
- 3. Check design requirements (Step 10)

11.1. Applied method

The fully probabilistic calculation is made with a level III probabilistic design method. Additionally a level II method can be used to examine the sensitivity of different variables on the system reliability, see section 4.2.3. However a Monte Carlo simulation offers multiple benefits over a level II calculation method. As concluded in chapter 10 executing step 7 and 9 separately would result in a complex system where the correlation should be described with correlation coefficient. This is prevented by carrying out one Monte Carlo simulation for the whole fully probabilistic calculation. Which results in combining step 7 and 9 to one step. A comparison is made between a level II FORM and a level III Monte Carlo simulation in table 11.1.

Characteristics	Reliability method		
Characteristics	Level II FORM	Level III Monte Carlo simulation	
Calculate P_f per mechanism	X	X	
Insight in governing failure mechanism	Х	X	
Increasing number of variables is increasing complexity	Х		
Sensitivity of input variables	Х		
Correlation coefficient required	Х		
Statistical and physical correlation easily included		X	
Insight in governing failure situations		X	

Table 11.1: Comparison between a level II FORM method and a Level III Monte Carlo simulation

In conclusion the Monte Carlo method suits the best for the fully probabilistic design calculation in this case because:

- · No restrictions of number of variables and integrals due to complexity
- Statistical and physical correlation is taken into account without using a correlation coefficient
- Influence of different failure mechanisms can be easily compared
- Insight in governing failure situations for each mechanisms is obtained

The largest disadvantage of using a Monte Carlo simulation instead of a level II FORM is the lack of insight in the sensitivity of the input variables. In a level II probabilistic calculation the sensitivity of the input variables is calculated and given via the α -values.

As seen in figure 11.1 making a probabilistic design is a iterative process and it's possible that several adjustments have to be made before the design requirements are fulfilled. The result of the fully probabilistic calculation is the reliability (i.e. probability of failure) of the total structure taking into account the imposed design requirements.

In this research the focus is on the applicability of a fully probabilistic calculation in practise. Due to time restrictions three simplifications are made. These restrictions are determined in such a way that the fully probabilistic calculation still gives realistic results. The applied simplifications are listed below:

1. Brettschneider formula instead of the model SWAN

To determine the nearshore wave conditions the model SWAN is used in the semi-probabilistic approach. Only one simulation is required to determine the governing conditions in contrast for the fully probabilistic calculation this would result in a lot of SWAN simulations which takes to much time.

In the fully probabilistic calculation this procedure is replaced with the Brettschneider formula to determine the significant wave height, see section 8.1.1. The other wave characteristics are determined via the correlation with the significant wave height, see chapter 10.

2. Basic equation of water level set-up instead of the model Delft3D

In the semi-probabilistic calculation Delft3D is used to calculate the water level set-up due to the wind. As same as for the model SWAN this would result in a lot of simulations with the Delft3D model when used in the fully probabilist calculation. For this reason the basic equation for water level set-up is used, see section 8.1.1.

3. 1 slip circle instead of the model D-Geo Stability

For the failure mechanism macro stability the model D-Geo Stability is used in the semi-probabilistic design process. This model examines multiple slip circles and determines the most unfavourable situation. If this is applied in the fully probabilistic calculation this would result in a lot of simulations with D-Geo Stability and for each simulation multiple slip circles are analysed. This is possible to included in the calculation however it would result in an enormous calculation time. For simplicity the most unfavourable slip circle, so only 1, determined via D-Geo Stability in the semi-probabilistic calculation is examined with the formula of Bishop in the fully probabilistic calculation.

From now on the models described above are referred to as mathematical models. In these research mathematical models are defined as models which describe the hydrodynamic processes (SWAN and Delft3D) and geotechnical stability (D-Geo Stability).

11.2. Monte Carlo Simulation

The whole Matlab procedure consists of nine different m-files. In this section the Program Structure Diagrams (PSD) are presented to clarify the Matlab routines. For each m-file a PSD is made. The Matlab codes are given in appendix G. The whole procedure is discussed top to down. Started is with the calculation of the total reliability of the structure and the last step is determining the required boundary conditions.

The main m-file from where all the procedure starts, defines the number of simulations (N) and processes the outcomes of the reliability functions. As explained in section 10.1.2 the statistical correlation is taken into account by examining each simulation separately to see if failure occurs. To show that statistical correlation occurs, also the yearly probability of failure for the non correlated situation is examined. In this situation is assumed that the failure mechanisms are mutually exclusive, see figure 12.3. To get a clear overview, the complete Matlab procedure to carry out the Monte Carlo simulation is given in figure 11.2. Each box is a different m-file and is discussed in the next sections.



Figure 11.2: Complete overview Matlab procedure to carry out Monte Carlo simulation simplified case

11.2.1. Total reliability

The total failure probability per year for the correlated situation is calculated as pictured in the PSD given in figure 11.3. The complete Matlab procedure is given in appendix G.1. The key is that each simulation (N) is examined separately. If failure occurs in a simulation the number of failed simulations (N_f) is increased with 1. After the last simulation the yearly failure probability ($P_{f,y}$) is calculated by dividing the number of failures (N_f) by the total number of simulations ($N = 3 \cdot 10^5$), see equation 11.1.

$$P_f = \frac{N_f}{N} \tag{11.1}$$

Subsequently the total probability of failure (P_{f,sys,t_L}) for rubble mound breakwater is calculated in this procedure with equation 11.2 which depends on the design lifetime $(t_l = 50 years)$.

$$P_{f,sys,t_L} = 1 - (1 - P_{f,y})^{t_l}$$
(11.2)



Figure 11.3: PSD of Total reliability of the system

11.2.2. Monte Carlo calculation

In this procedure the Monte Carlo (m-file: Monte_Carlo) procedure is executed. In this Matlab script the input variables are defined, for each variable the probability distribution (e.g. normal or deterministic) and corresponding characteristics (μ , σ) are given. All the probability distributions and characteristic of the input variables are given in the previous chapters. Only for the hydraulic boundary conditions, which are required as input, different Matlab procedures are used to generate these input variables. At last the actual Monte Carlo simulation is carried out with all the input variables. For this procedure the probabilistic OpenEarth toolbox of Deltares is used to execute the Monte Carlo simulation. The benefit of this toolbox is that instead of looping through all input variables N times, a matrix is created with randomly drawn values for all variables (n) from their probability distribution, see table 11.2.

Simulation number	Gravitational acceleration, g $[ms^{-1}]$	Density of armour stone, $\rho_s [kgm^{-3}]$	 n
1	9.81	2593	
Ν	9.81	2610	

Table 11.2: Matrix created by OpenEarth Toolbox, N by n matrix

This whole procedure is given in a process structure diagram in figure 11.4 and the Matlab procedure is given in appendix G.2. The probability distribution for the correlation factor between the significant wave height (H_s) and $H_{2\%}$, $T_{m-1,0}$ and $T_{m2,0}$ is loaded by a separated file "Windbins". These correlation factors are used to determine $H_{2\%}$, $T_{m-1,0}$ and $T_{m2,0}$ via the significant wave height (H_s) and differ per directional wind bin.



Figure 11.4: PSD of Monte Carlo simulation

11.2.3. Examining reliability functions

The next step is to define all the reliability functions and load all the hydraulic boundary conditions in to the procedure. All the reliability functions are defined in chapter 6. Furthermore variables which are correlated to the basic variables are calculated, such as the number of waves (N) and the surf similarity parameter ($\xi_{m-1,0}$). This is explained in section 10.1.2. The m-file Reliability functions contains all these procedures and makes the actual calculation of the Monte Carlo simulation. The procedure is given in figure 11.5 and the Matlab script is given in appendix G.3.

[z_vdm, z_t, z_r , z_bis] = Reliability functions (varagin)	DWL = HyBC(1,5);
%% Creates samples-structure based on input arguments samples = struct($H_2(i,1) = H_s^{samples.b(i)};$
'b', [], % Bed level with respect to BD 'b', [], % Ratio between Hs and H2%	Tm_02(i,1) = sqrt((H_s/s)*(2*pi/samples.g(i)));
- , W, samples = setproperty(samples, varargin(:));	Tm_10(i,1) = Tm_02(i,1)/a;
%% Calculate N number of simulations Windspeed, wave height and angle M-file: HydraulicBC	xi_s1 = samples.tan_a(i)/(sqrt((2*pi*H_s)/(samples.g(i)*Tm_10(i,1)*2)));
INPUT "N" OUTPUT "HyBC"	h = DWL - samples.bl(i);
%% Perform Monte Carlo by loop through all samples and derive z-values	N = samples.St_h(i)*3600/Tm_02(i,1);
M-file: Toe stability - Van der Meer Output "0 or 1"	(abs(samples.A_str(i)-Wave_ang)>180) True False
i = 1:length(N);	A_at = 360-(abs(samples.A_str(i)-Wave_ang)) A_at = (abs(samples.A_str(i)-Wave_ang))
% Read Significant wave angle out HyBC Wave ang = HyBC(L2);	detta = samples.rhos(i)/samples.rhow(i)-1
% Read Significant wave height out HyBC H_s = HyBC(i,6);	M-file: Armour stability - Van der Meer INPUT "Required variables (i)" OUTPUT "z_vidm(i,1)"
% Define which bin j = HyBC(i,4);	M-file: Toe stability - Van der Meer INPUT "Required variables (i)" OUTPUT "2_t(1,1)"
% Determine correlation factors for Tm02 (s) and Tm-10 (a)	M file: Dear eide stability. Van Gent and Pozueta
for j = 1:16	INPUT: Required viables (i)" OUTPUT "Z_(i,1)"
j == 1 True	M-file: Macro stability - Bishop INPUT "Required variables (i)"
s = samples.s(j)(i); a = samples.a(j)(i) j = j + 1	Output "Z_DIS((,1)"

Figure 11.5: PSD of procedure to examine reliability functions

11.2.4. Hydraulic boundary conditions

The PSD for hydraulic boundary conditions is given in figure 11.5. The hydraulic boundary conditions are the most complex boundary conditions to determine. In section 8.1.1 this is explained in further detail. Here the total procedure is given in figure 11.6 and the Matlab routine is pictured in appendix G.4. The results is a table with N hydraulic boundary conditions, see 8.1.

X = rand(1,1);
% Windspeed = $.1/\lambda^{1} \log(1-X) + \varepsilon$
Windspeed(i,1) = -Windbins(2,j)*log(1-X) + Windbins(1,j);
% Wave andle
HyBC(i,2) = Windbins(3,j);
%% Calculate wave height, give Windspeed and Fetch, with Brettschneider
HyBC(i,3) = Hs(Ws,F,g,h)
% Included uncertainity, via factor with normal distribution
HyBC(i,6) = HyBC(i,3) + HyBC(i,3) * normrnd(0,0.025);
%% Calculate design water level
% Mean sea level (0) MSL = .0.19
% Tide +/- 0.05 x1 = rand(1,1); Tide(i,1) = (x1-0.5)*0.1;
$x^2 = rand(1,1);$ Seas(i,1) = (x2-0.5)*0.2;
% Global Sea Level Rise Linear
$x_3 = rand(1,1); x_4 = rand(1,1);$
GSLR(i,1) = (x3/200) * (x4*50);
% Storm Surge % Cress formule
55(1,1) = MyBC(1,1)*2 * 55_T;
% Design water level = SUM(0-5)
HybC(1,3) = Wolt Hubb(1,1) + 0.02K(1,1) + 0.02K(1,1) + 0.02K(1,1),

Figure 11.6: Procedure to calculated hydraulic boundary conditions

To determine the hydraulic boundary conditions for the simplified case the wind occurrence per directional bin is required. This function (Winddata.m) is explained in detail in section 8.1.1. The PSD for this procedure is given in figure 11.7 and the complete Matlab routine is shown in appendix G.5.

[WindOcr] = Winddata(Widat)				
% Replace all values larger than 348.5 with x-360, to make correct bins For i = 1 : length(Widat)				
Widat(i)>3	48.5			
True	False			
WidatCor(i) = Widat(i)-360;	WidatCor(i) = Widat(i)			
Create 16 bins of 22.5 degrees. Bincenters 0 - binedges = [-11.5 348.75]	Create 16 bins of 22.5 degrees. Bincenters 0 - 337.5 binedges = [-11.5 348.75]			
% Convert WindOcr to normalized count bincounts = histc(WidatCor, binedges) totalcounts = sum(bincounts,1)				
For ii = 1:16				
WindOcr(ii,1) = bincounts(ii,1) / totalcounts;				

Figure 11.7: Procedure to calculated wind occurrence for 16 wind bins

11.2.5. Reliability functions

The last step is to calculate all the reliability functions for the four different failure mechanisms, see chapter 6, which are examined in the Monte Carlo simulation. These m-files are already introduced in figure 11.5. Each reliability functions has it's own function in the Monte Carlo analysis. The results for all these procedures is the value for the reliability function (Z) of the examined design formula.

Seaside armour stability - Van der Meer formula

The first function is the formula of Van der Meer to examine the seaside armour stability. In equation 11.3 the reliability function for deep water and plunging waves is given as example.

$$Z = \frac{H_s}{\Delta d_{n50}} - c_{pl,d} \ P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$
(11.3)

See figure 11.8 for the PSD and appendix G.6 for the complete Matlab routine. The routine determines if the Van der Meer formula for deep or for shallow water should be applied. Subsequently a distinction is made between plunging or surging waves based on the input variables.



Figure 11.8: Procedure to examine seaside armour stability with Van der Meer formula

Rear-side armour stability - Van Gent and Pozueta

Besides the stability of the seaside armour layer also the stability of the rear-side armour layer is examined. This is done with the formula of Van Gent and Pozueta:

$$Z = D_{n50} - 0.008 \left(\frac{S_d}{\sqrt{N}}\right)^{-1/6} \left(\frac{u_{1\%} T_{m-1,0}}{\sqrt{\Delta}}\right) (\cot \alpha_{rear})^{-2.5/6} \left(1 + 10 \exp\left(\frac{-R_{c,rear}}{H_s}\right)\right)^{1/6}$$
(11.4)

The uncertainty which is caused by the empirical derivation of this formula is included by picking a random number out of the normal distribution with mean $\mu = \frac{S_d}{\sqrt{N}}$ and standard deviation $\sigma = 0.1$. The PSD is given in figure 11.9. The complete Matlab routine is shown in appendix G.7.

[z_r] = Rearside (A_at,gamma_f,c1,c0,xi_s1,H_s,DHWL,cr_l,cr_lr,g, gamma_fc,B_c,d_r,Sd,Tm_10,delta,tan_ai,N_waves,j)				
angle_att < 80				
True	False			
angle_att_r= angle_att_ar	angle_att_ar = 80			
Calculate gamma_b(angle_att_ar)				
Calculate gamma(gamma_f,gamma_b)				
Calculate c2(c1,c0)				
Calculate p(c1/c0)				
lf ksi_s1 > p				



Figure 11.9: Procedure to examine rear-side armour stability with formula of Van Gent and Pozueta

Toe stability - Van der Meer formula

The next failure mechanisms that is examined is the toe stability with a formula by Van der Meer:

$$Z = D_{n50} - \frac{H_s}{\left(2 + 6.2\left(\frac{h_t}{h}\right)^{2.7}\right) \cdot N_{od}^{0.15} \Delta}$$
(11.5)

In figure 11.10 the PSD is given of the procedure to check the toe stability. The complete Matlab routine is shown in appendix G.8. The factor h_t/h is used to included the uncertainty to take into account the empirical nature of this formula. A random number is picked out of the normal distribution with mean $\mu = h_t/h$ and standard deviation $\sigma = 0.05$, see section 6.4.3.



Figure 11.10: Procedure to examine Toe stability with Van der Meer formula

Macro stability - Bishop

The last failure mechanism that is examined is the macro stability this is done with the formula of Bishop:

$$F = \frac{\sum \frac{c + (\rho_s gh - p) \tan \phi}{\cos \alpha_s (1 + tan(\alpha_s) tan(\phi/F))}}{\sum \rho_s gh \sin \alpha_s}$$
(11.6)

The used Bishop formula is solved via an iterative process, three iteration steps are carried out to get a reliable answer. One slip circle is examined with 12 parts, see chapter 6. Depending on the water level the procedure changes since there is more or less water pressure and saturated or unsaturated soil. The procedure is given in the process structure diagram in figure 11.11 and the exact Matlab routine in appendix G.9.



Figure 11.11: Procedure to examine macro stability with formula of Bishop

12

Results

In this chapter the results are presented of the fully probabilistic calculation (level III). The result of the calculation is the probability of failure (P_f) of the structure for a design lifetime $t_L = 50$ years. This gives the total failure probability for the structure (P_{f,sys,t_L}) over the lifetime. The starting point for each simulation has the following input conditions:

- Rubble mound breakwater of which one cross-section is examined, see chapter 6.
- Four different failure mechanisms, see chapter 6 and 11.
- The system reliability (i.e. probability of failure) is examined via a level III method with a Monte Carlo simulation, explained in detail in chapter 4 and 11.
- Design requirements for the fully probabilistic design process as defined in 7.
- Boundary conditions for the design as explained in chapter 8.
- Stone sizes D_{n50} as calculated in chapter 9.
- Statistical and physical correlation are taken into account, as described in chapter 10

The following simulations are made to see the influences of different factors on the failure probability (P_{f,sys,t_L}) :

- Number of simulations
- Failure probability over the set lifetime
- Different stone classes
- · Uncertainties in input variables and design formulas
- Damage level S_d and N_{od} expressed in a fragility curve

Subsequently the design requirements are checked, which is the last step in the probabilistic design process see figure 12.1. If the design requirements are met, no further optimization is carried out (this is not part of the research).



Figure 12.1: Indication of design requirements in probabilistic design process

12.1. Number of simulations

In appendix E is proven that at least 10^5 simulations (N) are necessary to obtain an uniform distribution for the variable x. This analysis is also done for the total probability of failure (P_{f,sys,t_L}), to make sure that this value is not varying for each time a run is made. In figure 12.2 the results of multiple runs with different number of simulations is pictured. As seen in the figure after approximately $2 \cdot 10^5$ simulations (N) the P_{f,sys,t_L} approaches a constant value. For the convenience $3 \cdot 10^5$ simulations are made each time a Monte Carlo simulation is executed to obtain the results discussed in this chapter.



Figure 12.2: Total probability of failure plotted against number of Monte Carlo Simulations

12.2. Failure probability

In this section first the failure probability of the total lifetime is determined depending on the level of correlation. Subsequently different simulations are made to see the conditions for which the mechanisms fail.

12.2.1. Level of correlation

In chapter 10 all the aspects in the fully probabilistic design process where correlation occurs are explained. To see the actual influence of statistical and physical correlation on the reliability of the structure three different simulations are made.

The system of the simplified case with the four failure mechanisms is in series. For the convenience figure 12.3 is pictured here again to show the different states of correlation for the series system.

Case	Mutually exclusive	Independent	Dependent
Correlation coefficient. $\rho_{Z1,Z2} =$	-1	0	1
Venn diagram	F1 F2	F1 F2	F1, F2
System failure probability $P(F)$	$P(F_1) + P(F_2)$	$P(\mathbf{F}_1) + P(F_2) - P(F_1) \cdot \mathbf{P}(\mathbf{F}_2)$	$Max(P(F_1, P(F_2)))$

Figure 12.3: Influence of correlation on total system failure probability of a series system [Jonkman et al., 2015].

As mentioned in this section three different states are examined. The first one is the situation where the failure mechanisms are mutually exclusive ($\rho = -1$), which gives the upper bound for the failure probability. The second one is the situation where the failure mechanisms are complete dependent ($\rho = 1$), which is the lower bound. The failure probabilities are calculated separately for each failure mechanism and the upper and lower bounds are calculated via the formulae given in chapter 10.1.2 and figure 12.3.

The results in table 12.1 give the upper and lower bounds of the series system for the simplified case. In section 10.1.2 is explained that correlation can influence the failure probability of the system. In the simplified case this is taken into account via the Monte Carlo simulation (see table 10.3). With this approach the probability of failure for the total system is calculated directly. The result of this simulation is given in table 12.1.

Reliability functions	Yearly probability of failure (P_f) [%]	Total probability of failure $(P_{f_{tl}})$ [%]
Seaside armour stability	5.333e-03	0.27
Rear-side armour stability	3.333e-04	0.017
Toe stability	3.6667e-03	0.18
Macro stability	0	0
Lougl of completion		

Level of correlation		
Mutually exclusive ($\rho = -1$)	9.3333e-03	0.47
Dependent ($\rho = 1$)	5.333e-03	0.27
Correlation via MC	9.3333e-03	0.47

Table 12.1: Different states of correlation for the series system of the rubble mound breakwater ($S_d = 8$ and $N_{od} = 5$)

From these results is concluded that the series system has a mutually exclusive behaviour for these input conditions ($S_d = 8$ and $N_{od} = 5$). This means that only one failure mechanisms fails at a time and never simultaneously. So for example, if the seaside armour layer fails, no other fail mechanisms fails. The correlation coefficient for the four failure mechanisms is in this case $\rho_{M1,M2,M3,M4} = -1$.

In chapter 10 is stated that the correlation which occurs in the fully probabilistic design process has an influence on the failure probability of the system. However from the results presented in table 12.1 it seems that correlation has no impact on the probability of failure of the system. The failure probabilities are quite small and the conclusion that the system is mutually exclusive only holds for the taken input conditions. To get a better insight if the correlation is influencing the probability of failure, another simulation is made for a damage level classified as 'start of damage' ($S_d = 2$ and $N_{od} = 0.5$).

The upper and lower bounds are calculated and given in table 12.2. Subsequently the failure probability of the system is calculated via the Monte Carlo simulation which takes into account the correlation. From these results is concluded that the series system of the simplified case for these input conditions is independent. The correlation coefficient of the system is somewhere between $-1 < \rho < 1$, this is referred to as an independent series system. Situations occur where mechanisms fail simultaneously and also situations occur where only one mechanism fails.

In conclusion the level of correlation in the simplified case depends on the number of situations where failure occurs. If the failure probabilities are low in the simplified the series system has a mutually exclusive behaviour. When the input conditions are such that more situations of failure occur the series system of the simplified case is independent.

12.2.2. Failure areas

From the results in table 12.2 is concluded that the series system for the input condition 'start of damage' is independent. For this reason the series system where exceeding the damage factors $S_d = 2$ and $N_{od} = 0.5$ corresponds to failure is used in this section. This means that there are three different failure situations: no failure, failure of only one mechanism and failure of multiple mechanisms.

This can be made easily understandable by making graphs with different failure areas (Z<0 no failure, 0>Z>0 (no) failure, Z>0 failure). First this is done for the most dominant (largest contribution to the failure probabil-

Reliability functions	Yearly probability of failure (P_f) [%]	Total probability of failure $(P_{f_{tl}})$ [%]
Seaside armour stability	0.10	4.88
Rear-side armour stability	2.6667 e-03	0.13
Toe stability	0.90	36.37
Macro stability	0	0
Level of correlation		
Mutually exclusive ($\rho = -1$)	1	39.49
Dependent ($\rho = 1$)	0.90	36.37
Correlation via MC	0.98	38.97

Table 12.2: Different states of correlation for the series system of the rubble mound breakwater ($S_d = 2$ and $N_{od} = 0.5$)

ity) failure mechanisms, the seaside armour stability and the toe stability. The main load parameter for these failure mechanisms is H_s which is plotted against the main strength parameter D_{n50} . For each value mechanism $3 \cdot 10^5$ situations for the reliability function (Z) are calculated. In figure 12.4 a grey dot corresponds with no failure (Z<0) and a coloured * or + with failure (Z>0).



(a) Seaside armour stability



Figure 12.4: Failure areas for seaside armour and toe stability

In this graph different areas are recognized. First the areas with a clear distinction where no failure or failure occurs. Furthermore there are areas where no failure and failure occurs. This is because the reliability functions does not only depend on the D_{50} and H_s . Other input variables are also present which influence the outcome of the reliability functions. At last there are areas where no simulations are present and logical reasoning gives no answer (increase of D_{50} (strength) and H_s (load)), these are indicated with 'unknown'.

From the results given in figure 12.4a it is concluded that the seaside armour layer fails most of the time for a significant wave height between 1.5 and 3 meter. Remarkable is that larger wave heights for the same D_{n50} does not automatically result in failure. This is due to the fact that these wave heights only occur for higher water levels (see figure 12.5). When higher water levels are present the deep water equation for Van der Meer is used instead of the one for shallow water, see chapter 6. From the large spread in failure situations it is concluded that a lot of input variables influences the outcome of this reliability formula for the seaside armour.

The results in figure 12.4b show that the toe fails most of the time for high significant wave heights in combination with (relatively) small nominal stone diameters. This is what is expected since the reliability for toe stability has only a few input variables and strongly depends on H_s and D_{n50} .

To see if multiple mechanisms fail at the same time, common input variables for the failure mechanisms on the x-axis and y-axis are required. For this reason the significant wave height (H_s) is plotted against the water level (DWL). The three failure mechanisms which contribute to the failure probability of the system are in-

cluded, see figure 12.5.



Figure 12.5: Failure areas for series system of simplified case

Several different failure areas are distinguished. First of all the conditions for which non of the mechanisms fails (Z<0). Subsequently the area where for some of the conditions result in failure and others not. The situations for which failure always occurs (Z>0) only holds for a few conditions. Besides these three areas there are also areas for which no data is available in this case which is displayed with 'unknown'.

From this graph it is concluded that low water levels in combination with high significant wave height result in failure of the sea side armour. Conditions where high water levels are present in combination with high waves result in failure of the toe. Furthermore it seems that for the toe stability the water level is not governing. Since failure occurs for high and low water levels for significant wave heights larger than 2.4 meters. For the rear-side armour only conditions were the water level is very high result in failure. This is not for all high water levels because waves from a direction of 90 degrees or more to the normal are not taken into account for the rear-side armour stability. The last conclusion is that for situations were failure always occurs (Z,0), in most case the toe and seaside armour stability fails simultaneously (circle and cross). So indeed the considered series system is independent, as also concluded from the results given in table 12.2.

12.3. Different stone classes

In the chapter 9 a distinction is made between three different solutions for the simplified case:

- 1. Design as applied in project Taman, see 12.3.
- 2. Calculated design simplified case via the The Rock Manual [2007]
 - (a) Simplified case D_{n50} , see table 12.1.
 - (b) Stone class simplified case, see table 12.4.

From these results conclusions can be drawn, important to realise is that the failure probability only is based on four failure mechanisms in the ULS.

Reliability functions	Yearly probability of failure (P_f) [%]	Total probability of failure (P_{ftl}) [%]
Seaside armour stability	0.0417	2.06
Rear-side armour stability	3.333e-04	0.017
Toe stability	0.0140	0.70
Macro stability	0	0
Correlation via MC	0.0549	2.71

Table 12.3: Applied stone class in the case of Taman, 60 - 300 kg for seaside armour and toe, and 5-40kg for rear-side armour, solution 1

Reliability functions	Yearly probability of failure (P_f) [%]	Total probability of failure $(P_{f_{tl}})$ [%]
Seaside armour stability	0	0
Rear-side armour stability	6.667 e-04	0.0333
Toe stability	0	0
Macro stability	0	0
Correlation via MC	6.667 e-04	0.0333

Table 12.4: Stone classes 300 - 1000 kg for seaside armour and toe, and 5-40kg for rear-side armour, solution 2b

Applying the D_{n50} which is calculated for the simplified case (solution 2a, table 12.1) results in a failure probability of approximately 0.5%. A failure probability of 15% is allowed so the design is over dimensioned. If the corresponding stone classes is applied (solution 2b, table 12.4) this leads to an even further over dimensioning ($P_{f_{1l}} = 0.033\%$). The design made for the project Taman has a failure probability of 2.7%. It meets the set design requirement for the failure probability of the ULS.

12.4. Varying uncertainty

To take into account all the present correlations a Monte Carlo simulation is used for the level III method. Unfortunately no insight is obtained on the influence of the uncertainties on the reliability of the structure. To get an idea of the influences of the uncertainties on the P_{f,sys,t_L} several Monte Carlo simulations are made with varying levels of uncertainties. Different types of uncertainties are distinguished:

- Inherent uncertainties and statistical in the input variables
- Model uncertainties in the design formulae

Uncertainness are present in the input variables due to natural fluctuations (inherent) and reliability of the used data (statistical). Furthermore uncertainties are present in the design formulas due to the empirical derivation (model) of these formulas. First multiple Monte Carlo simulations are made to see the influence of the uncertainness in the input variables on the P_{f,sys,t_L} . Subsequently the uncertainty in the design formulas is varied to see if the P_{f,sys,t_L} is influenced. Each input variables and design formula has an *initial* uncertainty. This is assumed to be the actual uncertainty in the simplified case. The initial uncertainties are used in the Monte Carlo simulations which gives the results as presented in tables 12.2, 12.3 and 12.4.

12.4.1. Input variables

The uncertainties in the input variables are caused by inherent and statistical uncertainties. All the input variables have a normal distribution expect for the significant wave height. However for the significant wave height a factor with a normal distribution is applied to take into account the uncertainties, see section 8.1.1. The uncertainties in these variables are varied by changing σ in the normal distribution to: no uncertainty at all, + 10% and +25%. In this simulation a distinction is made between variables which result in a load on the structure and variables which are related to the strength.

The load variables in this case are the significant wave height (H_s) and the water level (DWL). The initial uncertainty of H_s is assumed to be 2.5% and for DWL to be 0%. For H_s this results in four simulations with the following uncertainties in the variable: 0% (non), 2.5% (initial), 12.5% (+10%), 27.5% (+25%). Since the initial uncertainty in the water level is assumed to be 0%, for the DWL only three simulations are made: 0% (non and initial), 10% (+10%), 25% (+25%).

The strength variables in this case are the nominal stone diameter (D_{n50}), soil parameters (γ , ϕ), density of stone (ρ_s) and geometric parameters (B, α , cr, h_{toe}). The initial uncertainness for these variables are explained in chapter 9 and are given in table 9.3. In these parameters is assumed that a maximal variation in uncertainty is possible up to +10%. Larger uncertainties are not realistic for the project Taman and the simplified case. The results are shown in table 12.5 and figure 12.6.

Variable	Total probability of failure $(P_{f_{tl}})$ [%]			
variable	Non	Initial	+ 10 %	+ 25%
H_s (Initial 2.5%)	0.40	0.47	2.16	24.46
DWL (Initial 0%)	0.47	0.47	0.66	0.70
D_{n50} (Initial 5%)	0.37	0.47	7.48	-
Soil parameters (Initial 2.5%)	0.49	0.47	0.57	-
$ ho_s$ Density of stone (Initial 0.5%)	0.37	0.47	18.24	-
Geometric parameters (Initial 5%)	0.53	0.47	2.18	-

Table 12.5: Sensitivity of different input variables on the total probability of failure



Figure 12.6: Sensitivity of different input variables on the total probability of failure

These results show that the load parameter H_s has a large influence on the probability of failure of the system. Remarkable is that an increase of uncertainty in the other load parameter, the water level (DWL), has no influence on the failure probability. Furthermore larger uncertainties in the main strength parameters D_{n50} and ρ_s for the hydraulic failure mechanisms result in significant increase in the failure probability. An increase in the uncertainties of the soil parameters only results in marginal increase of the failure probability. This could be explained by the fact that the geotechnical failure mechanism macro stability has no contribution to the failure probability of the structure. At last the uncertainties in the geometric variables have a medium influence on failure probability. So when the design is finalized the geometric variables should be defined accurately (with an allowable deviation), especially for the contractor who will realize the design.

12.4.2. Design formulae

In this simulation the model uncertainties in the formulas are reduced or increased via the empirical fit parameters. In the formula of Bishop for macro stability no fit parameter is included since this formula is not empirical derived. In the Van der Meer formula the uncertainty is included in the values for c_{pl} and c_s . In the formula of Van Gent and Pozueta in the factor $\frac{S_d}{\sqrt{N}}$, with a $\sigma = 0.1$ for $S_d < 10$. In the formula of Van der Meer formula the factor h_t/h with $\sigma = 0.05$. The uncertainties are changed for all the formula's at the same time, so +10% certainty is added to every design formula for the same Monte Carlo simulation. The results are given in table 12.6 and figure 12.7.

Variable		Failure probability per failure mechanism [%]($P_{f_{tl}}$)			
		Initial	+ 10 %	+ 25%	
Van der Meer sea side armour	0.23	0.27	4.05	68.59	
Rear-side armour stability Gent Pozueta	0	0.017	0	0	
Toe stability Van der Meer	0	0.18	20.59	77.74	
Probability of failure total lifetime	0.23	0.47	24	93	

Table 12.6: Influence of varying model uncertainties in the design formulas on the failure probability



Sensitivity analysis of design formulae

Figure 12.7: Sensitivity of the design of formulae on the total probability of failure

From the results given in figure 12.7 is clearly seen that an increase in uncertainty in the design formulae of Van der Meer is resulting in a higher probability of failure. On the other hand increasing the uncertainty in the formula of Van Gent and Pozueta has no significant influence. This could be explained by the fact that the contribution of the rear-side armour stability to the total failure probability is marginal.

12.5. Varying damage level

The last simulations are made with a varying damage level. The design requirement for the damage level in the ULS is set to 'failure', which corresponds with $S_d = 8$ and $N_{od} = 5$. In this section other damage levels are analysed with the Monte Carlo simulation to see which level of damage is likely tot occur during the design lifetime. The results are presented in table 12.7.

Damage level	Total probability of failure $(P_{f_{tl}})$
Start of damage $S_d = 2$ and $N_{od} = 0.5$	0.3897
Intermediate damage $S_d = 5$ and $N_{od} = 2$	0.0250
Failure $S_d = 8$ and $N_{od} = 5$	0.0047

Table 12.7: Probability of failure (P_{f,sys,t_L}) for varying damage levels

The formula specific design requirements are given in design codes and standards. Important is to select the design requirements for the selected level of damage. The results given in table 12.7 are for the standard damage levels [The Rock Manual, 2007]. Interesting to see is what the probability of exceedance is for all possible damage factors. This is done via a fragility curve where the damage level is varied, see figure 12.8. Both of the damage parameters (S_d, N_{od}) are varied at the same time to give the same damage level for the complete breakwater. Assumed is that the damage level has a linear trend between the different damage levels as given in table 12.7.



Figure 12.8: Fragility curve of damage level S_d and N_{od}

As seen in figure 12.8 'start of damage' ($S_d = 2$ and $N_{od} = 0.5$) is very likely to occur during the lifetime and more severe damage is less likely to occur. In the simplified case the failure probability (P_{f,sys,t_L}) for 'start of damage' for a life time of 50 years is 39% in the simplified case. Remarkably this is approximately the same as determined with the semi probabilistic design (1/100 year storm, $P_{f_{il}} = 40\%$) for this level of damage. Failure of the rubble mound breakwater has a probability of occurring of 0.47%.

12.6. Checking design requirements

The design requirements for the ULS in the fully probabilistic design calculation are given in table 12.8.

Design requirements		Calculation method		
		Semi probabilistic	Probabilistic	
Overall requirements Target reliability		1/100 year storm	$0.15 P_{f_{tl}}$	
•	Design lifetime	50 years	50 years	
Specific requirements	Damage number, S_d	2	8	
•	Damage number, N _{od}	0.5	5	
•	Stability number, F	1	1	
•	Factor cohesion, y_c	1.25	1	
•	Factor friction angle, y_{ϕ}	1.4	1	

Table 12.8: Design requirements for the Ultimate Limit State (ULS) for the simplified case

For the results given in this chapter the specific requirements as given in table 12.8 for the probabilistic design are used. In table 12.4 is seen that the failure probability $(P_{f_{tl}})$ is 0.033% for thee stone classes which should be applied according to the preliminary design. This is significantly smaller than the 15% which is the maximum allowable failure probability. So this design meets the set probabilistic design requirements.

In the case of Taman other stone classes are applied based on a semi-probabilistic design method, expert judgement and 3D physical model tests. These results are given in table 12.3. The structure has a failure probability of 2.5% for a life time of 50 years for the ULS. This is smaller than the set design requirement of 15%, so this design also meets the design requirements for the examined ULS and failure mechanisms.

All the examined scenarios meet the design requirements (1, 2a, 2b). To see if the structure is over dimensioned based on the four failure mechanism another simulation is made with lower stone class:

- Sea side armour stability: $40-200 \text{ kg} (D_{n50} = 0.34 m)$
- Rear-side armour stability: $2.1-2.8 \text{ kg} (D_{n50} = 0.097 m)$
- Toe stability: $40-200 \text{ kg} (D_{n50} = 0.34m)$

The results are given in table 12.9. From these results is concluded that a lower stone class for each mechanisms results in a $P_{f_{tl}}$ of 11.25% and still meets the design requirement of 15%.

Reliability functions	Yearly probability of failure (P_f) [%]	Total probability of failure $(P_{f_{tl}})$ [%]
Seaside armour stability	0.16	7.77
Rear-side armour stability	1.33e-03	0.07
Toe stability	0.08	4.04
Macro stability	0	0
Correlation via MC	0.24	11.25

Table 12.9: Probability of failure for the simplified case with lower stone classes

From these results the conclusion can be drawn for this simplified case that the design values in the semiprobabilistic design formulae result in a conservative design. A suggestion is to change the design values in these design formulae to make a more efficient design. However this is only based on this case, so more cases should be examined to see if this is a general trend.

Furthermore it is important to realize that only four failure mechanisms are taken into account in this simplified case. In the next section is explained that other failure mechanisms which are not included can contributed to the total probability of failure. So it is risky to apply lower stone classes if the contribution of other failure mechanisms (e.g. earthquakes) are not known. Because only a 3.75% percent of failure space is available for the mechanisms which are not examined.

12.7. Fault tree

In section 6.3 the assumption is made that the four selected failure mechanisms have the largest contribution to the total failure probability of the rubble mound breakwater in section B. In this research no other failure mechanisms are examined, so it is not possible to make an comparison with for example the SLS mechanisms. However with the calculated results of the fully probabilistic calculation an overview can be made of the contribution of each failure mechanisms to the total failure probability. In figure 12.9 the fault tree is pictured. The numbers in black give the failure probability per failure mechanisms and the contribution to the total failure probability (P_{f,sys,t_L}). The green number of 15% is the allowable failure probability for the ULS of the breakwater as set by the design requirements, see table 12.8.

As seen in the fault tree, the remaining failure mechanisms (other failure mechanisms) could have a contribution of approximately 14.5% to the failure probability (P_{f,sys,t_L}) with the set design requirement of 15%. Expected is that the dynamic loading (earthquakes) also contributes significantly to the failure probability (P_{f,sys,t_L}). This mechanism is not included in the simplified case.


Figure 12.9: Fault tree for ULS in simplified with probability of failure $(P_{f,sys,t_L})\ [\%]$

The top event of the simplified case was the port down time, see 12.10. Important is that not only the failure probability of the ULS for the breakwater is calculated but also the failure probability of the SLS and other events. This is not carried out in the simplified case. Figure 12.10 places the calculated probability of failure (0.47 %) for the ULS in perspective to the other possible events which contribute to the failure probability (P_{f,sys,t_L}).



Figure 12.10: Fault tree with top event port downtime for simplified case

From these failure tree is concluded that with the design applied in the simplified case the design requirement for the ULS of the breakwater is met. However to make a complete probabilistic calculation for the port downtime, the SLS of the breakwater and contribution of other events should also be taken into account. These events could have a large contribution to the failure probability of the top event port downtime.

III

Discussion, conclusions and recommendations

13

Discussion

In this chapter the examined case and calculation method are discussed and evaluated. First a literature review is made in which the contributions of this research to the current knowledge on the probabilistic design method for a rubble mound breakwater are given. Subsequently the made decisions, assumptions and simplification during this research are criticized. The following subjects are treated: the chosen limit state with corresponding failure mechanisms, the simplifications which are applied to avoid the use of mathematical models¹ in the fully probabilistic calculation, the assumed uncertainties in the basic variables, the physical correlation which was only partly taken into account, the chosen target reliability with the corresponding level of damage and the design values for the semi-probabilistic design. The influence of these decisions, assumptions and simplifications on the results are discussed and evaluated.

13.1. Literature review

In the beginning of this research a literature study has been performed to investigate the existing knowledge on the topic of probabilistic design for rubble mound breakwaters. In conclusion multiple studies show the feasibility of designing rubble mound breakwaters with a probabilistic design process in theory. However in practice only few breakwaters are designed with a probabilistic design method. Recently in Spain the secondary breakwater in the new harbour basin of the outer port of La Coruña [Maciñeira, 2016] is designed with a full probabilistic design method.

The reason is the lack of information regarding the following aspects of the full probabilistic method when applied in practice:

- Design requirements
- Boundary conditions
- Statistical and physical correlations

Information on how to set the design requirements and derive the boundary conditions is already available. However how to deal with the statistical and physical correlation in a fully probabilistic calculation was not yet know exactly. In PROVERBS [Allsop et al., 1999] is proposed that first the probability of failure for each mechanisms should be calculated separately and subsequently combined via correlation coefficients (see figure 5.2). This research concluded that the applying a Monte Carlo simulation for the full procedure is a more convenient way to take into account the correlation when designing a rubble mound breakwater.

Furthermore this research is a case which shows that applying the semi-probabilistic design values as proposed in The Rock Manual [2007] can result in a over dimensioned design. Important note is that this should be further investigated by including all failure mechanisms for both the SLS and ULS situation. It may be that the SLS is governing in some situations, e.g. wave overtopping events.

¹Models which describe the hydrodynamic processes (SWAN and Delft3D) and geotechnical stability (D-Geo Stability)

In conclusion the contributions of this research to the existing knowledge are as follows:

- Proving that statistical and physical correlations influence the calculated failure probability of a rubble mound breakwater
- Showing that a Monte Carlo simulation is a good way to take into account these correlations
- Providing a case which shows that the formulae in the semi-probabilistic design result in an over dimensioned design.

13.2. Failure mechanisms and limit state

In this research the decision has been made to only investigated the ULS with a couple of corresponding failure mechanisms to reduce the complexity. The calculated failure probability only covers these relevant failure mechanisms and is probably larger when all the present failure mechanisms are included. Moreover the failure mechanisms regarding the SLS also contributes to the total failure probability of the examined rubble mound breakwater. Sometimes the failure mechanisms regarding the SLS are governing (e.g. crest level for overtopping criteria)

Due to the decision to only included the ULS with a couple of failure mechanisms, the actual probability of failure is, although the most relevant parameters have been selected, slightly underestimated. It is advisable to included all the present failure mechanisms (SLS and ULS) to make sure the calculated failure probability is correct. However, with this case the essence of the fully probabilistic calculation and the corresponding problems are captured. Including more failure mechanisms is a interesting direction for further research as also described in section 14.2.

13.3. Model simplifications

During this study three simplifications have been made due to time restrictions and to reduce the complexity of the fully probabilistic calculation:

- Brettschneider formula instead of the model SWAN
- Basic equation of water level set-up instead of the model Delft3D
- 1 slip circle instead of the model D-Geo Stability

In the semi-probabilistic approach the model SWAN was used to determine the nearshore wave conditions. In the fully probabilistic calculation this model should also be implemented in the procedure to take into account the nearshore wave processes. This requires multiple SWAN simulations and results in many calculations with a large computation time. For this reason, the Brettschneider formula has been used to determine the significant wave height (section 8.1.1). In the examined case this is a good method since the wave characteristics are wind driven. In other cases (e.g. project located at a ocean coast) this is not a good approximation.

The other wave characteristics for the simplified case have been determined via the correlation factor with the significant wave height, see chapter 10. This correlation factor has been derived from the data of the semi-probabilistic design i.e. a single SWAN calculation. This procedure is probably subject to errors due to major simplifications. The Brettschneider formula requires the input of the fetch length and the water depth, these values are different for each wind direction, however no variation in water depth or fetch for a wind direction is taken into account. Furthermore the used correlation factors, to determine the other hydraulic boundary conditions, are derived from the semi-probabilistic design. All this simplifications could lead a less reliable determination of the hydraulic boundary conditions.

The second simplification was the use of the basic equation for wind driven water level set-up. In the semiprobabilistic calculation the model Delft3D, was used which is accurate but a complex model to calculate the wind driven water level set-up. Due to this simplification the water level set-up due to the wind used in the probabilistic calculation is maybe subjected to errors. For my case, the influence of the water level variation on the failure probability is marginal. The last simplification is concerning the failure mechanisms macro stability. In the semi-probabilistic calculation the model D-Geo Stability was used. This model examines multiple slip circles and determines the most unfavourable situation with the corresponding stability number (F). The ideal situation would be to included this model in the fully probabilistic calculation. However, due time restrictions for this study a simplification was applied. Only one slip circle has been examined during the fully probabilistic calculation. This slip circle was selected on the basis of the semi-probabilistic calculation. This could definitely result in an underestimation of the failure probability for the mechanism macro stability, since not all the possible slip circles are examined.

13.4. Assumed uncertainties

For all the basic variables a probability distribution and corresponding characteristics were determined. In this research most of these probability distributions are an estimation and not thoroughly investigated. The results in chapter 12 show that the uncertainties in the basic variables can have a large influence on the probability of failure of the structure. For example the significant wave height (H_s) has an assumed model and inherent uncertainty of 2.5% of the mean. However when this uncertainty increases to 12.5% due to for instance few measurements the probability of failure increase to 2.2% instead of the 0.5% which was calculated. This shows that the assumed uncertainties in the basic variables can have significant influence on the probability of failure, although this is not extensively examined in this research.

13.5. Level III instead of level II

In chapter 11 has been concluded that for the simplified case a fully probabilistic (level III) calculation carried out with a Monte Carlo simulation (MC) is the most suitable. One thing this method is not offering, is insight in the sensitivity of the different input variables. A level II method would give a better insight via the α values. However, for this method the correlation coefficients between failure mechanisms should be known, this results in a more complex calculation and is not examined in this research. In this research a sensitivity analysis is made for the expected most dominant input variables. However if all the α -values were known, a better comparison between the sensitivity of the input variables is possible. This information could be used to make an effective optimization, by for example investing time and money to reduce the uncertainties in the most sensitive input variables. These optimizations result in a lowering of the failure probability of the designed rubble mound breakwater. So a shortcoming of the applied level III method in this research is the lack of insight in the sensitivity of the input variables.

13.6. Physical correlation

Two different types of correlation occur in the fully probabilistic design process. The first one is the statistical correlation which is taken into account during this research. The second one which is the physical correlation was not completely included in the fully probabilistic design process. Only the situation where failure of a mechanism induces failure of an other mechanism has been included. The other situation where partly failure of one mechanism induces failure of other failure mechanisms was not included in the calculation. As explained in the chapter 10 this could lead to an increased failure probability i.e. an overestimation of the reliability of the structure.

13.7. Allowable failure probability

The design requirements for the probabilistic design have been determined by the three steps as described in chapter 7. This is only based on a few codes and standards and a couple of reference cases. The allowable failure probability (i.e. target reliability) has a significant influence on the design. The probabilistic calculation shows that the design made by a semi-probabilistic reliability method meets the design requirements. However, if the target reliability is defined differently the design could maybe not meet the set requirements.

The target reliability would always be a point of discussion since different standards suggest different values. This is due to the fact that the consequences of failure should be estimated and this is partly subjective. Expressing the consequence of failure in costs is a more objective way to quantify the consequences. This approach is used in a level IV risk based method. In the end this would be the best method to determine the target reliability, given that there is enough reliable data to determine the construction and failure costs.

13.8. Design values

The results of the fully probabilistic calculation show that lower stones classes could be applied for the toe, seaside and rear-side armour. So based on the examined case the design values in the semi-probabilistic design formulae could be changed to make a more efficient design. However all the failure mechanisms for the SLS and ULS should be included, instead of only the four failure mechanisms for ULS which are examined in the simplified case. Also more cases should be investigated to see if this is a general trend before adapting the current design formulae used in the semi-probabilistic design.

14

Conclusion and recommendations

In this chapters conclusions are drawn from the research presented in the previous chapters. First the final conclusions are given and subsequently further elaborated by answering the research questions as formulated in section 1.3.

14.1. Conclusions

The main objective of this research is formulated as follows:

Investigate how a probabilistic design of a rubble mound breakwater in practise can be made.

This research objective is formulated on the basis of the following problem definition:

Insight in the actual failure behaviour and probability of failure of a rubble mound breakwater is lacking and can probably be obtained by applying a probabilistic design method (Level II and III). These methods are not often applied due to the lack of knowledge on how to determine the design requirements and boundary conditions for the probabilistic design process, in addition unknown is how to deal with the statistical and physical correlation in this process.

To achieve this research objective and to find solutions for the problems which are described in the problem definition a literature study is carried out and several research questions are formulated. During this research several aspects of the fully probabilistic design process are investigated. In the next sections conclusions on this research are given.

14.1.1. Final conclusion

The conclusion of this research is that a fully probabilistic calculation for a rubble mound breakwater is possible in practise. The results show that it is certainly beneficial to apply a fully probabilistic calculation (Level III) compared to a semi-probabilistic (level I) calculation because:

- · Optimized design
- Insight in governing failure mechanism and failure behaviour

The results of the fully probabilistic calculation show that making a semi-probabilistic design based on the design rules in the The Rock Manual [2007] results in a conservative design ($P_{f,sys,t_L} = 0.5\%$). One optimization step is made for the simplified case in this research by applying lower stone classes for the toe, seaside and rear-side armour. This results in failure probability ($P_{f,sys,t_L} = 11.25\%$) which is still lower than the in general allowable probability of failure ($P_{f,sys,t_L} = 15\%$). Although not all failure mechanisms for the SLS and ULS are taken into account this study proves that a fully probabilistic calculation results in a more optimized design compared to a semi-probabilistic design method for the examine case.

Furthermore the Monte Carlo analysis gives a good insight in the failure mechanisms which have the largest contribution to the failure probability of the structure. Moreover various scenarios are examined with the Monte Carlo simulation, this provides insight in the governing failure behaviour for each mechanism. With this information adoptions can be made to improve the reliability of the designed rubble mound breakwater.

These two mentioned advantages show that it is wise to apply a fully probabilistic design method for a rubble mound breakwater. However not in all cases it is possible to apply a fully probabilistic calculation and a couple of aspects should be checked before starting the calculation:

- Take into account statistical and physical correlation
- Enough reliable data for boundary conditions should be available
- Applied mathematical models should be incorporated in the fully probabilistic calculation

This research shows that statistical and part of the physical correlations reduces the failure probability of a rubble mound breakwater in a probabilistic calculation (level II and III). Neglecting the correlations results in over dimensioning the rubble mound breakwater. So when applying a fully probabilistic calculation the correlations should be take into account. In this research is chosen to apply a fully probabilistic (level III) design method with a Monte Carlo simulation. This method proves to be a good way to included the statistical and part of the physical correlation in the fully probabilistic design process.

In the fully probabilistic calculation the basic variables have to be derived to take into account the statistical correlation. Large uncertainties in the basic variables result in a high failure probability of the rubble mound breakwater. Especially uncertainties in the significant wave height have a large contribution to the variation in the probability of failure. The results show that a fully probabilistic calculation in the simplified case is not feasible when the uncertainties in H_s have a standard deviation of 25%. In conclusion enough reliable data should be present for the wave conditions at the project location to make a fully probabilistic design method feasible.

In the semi-probabilistic design the hydraulic boundary conditions are determined via the models SWAN and Delft3D. Additional the model D-Geo Stability is used for the semi-probabilistic design to check the geotechnical failure mechanisms. In this research is concluded that all hydraulic, geotechnical and geometric conditions need to be carried out in fully probabilistic way. For the fully probabilistic calculation of the simplified case simplifications are made for the mathematical models. These simplifications give a good approximation of the models in this particular case. However performing a fully probabilistic calculation for other cases, where for example nearshore wave process are present, mathematical models should be applied. To perform a fully probabilistic calculation in other cases the implementation of mathematical models into the calculation should be further examined.

14.1.2. Research questions

All the research questions are answered in the Part II, each research question has a corresponding chapter. In this section the research questions are answered based on the solutions provided in the previous chapters.

How to describe the probabilistic design method in a process diagram?

In chapter 5 the design process is described. This analysis was carried out prior to the fully probabilistic calculation and based on literature and first estimate of the design process. During this research different aspects are investigated which caused problems in the fully probabilistic design process in practise. The solutions for these different problems result in altering the design process. In this study is shown that correlation, which appears in the design process, has influence on the failure probability of the total structure. Further explanation is given in the research questions about the statistical and physical correlation. The correlation can be included into the design process in two ways:

- Described the correlation with a correlation coefficient
- Included the correlation by performing one Monte Carlo simulation for the complete fully probabilistic design process.

In this research is concluded that performing a Monte Carlo simulation for the complete design process is a good way to take into account the correlation. This results in an alteration in the flowchart presented by PROVERBS [Allsop et al., 1999] (see figure 5.2) when applying this method to a rubble mound breakwater. Initially the failure probability of each failure mechanisms was calculated separately and eventually combined to the total failure probability via the correlation coefficients. In this research is chosen to apply a Monte Carlo simulation which takes into account the correlation without using correlation coefficients. This alteration is pictured in figure 14.1, which shows the final flowchart to execute a fully probability calculation for a rubble mound breakwater.



Figure 14.1: Design process to perform a fully probabilistic calculation

How to determine the required boundary conditions for the probabilistic design of a rubble mound breakwater?

The required boundary conditions depends on the components of the rubble mound breakwater, the project location and the applied design formulae with their corresponding input variables. Important is that the basic variables are derived from these input variables to take into account the statistical correlation, see chapter 10. In figure 14.2 this process is pictured. On the top level an example is given of the sea side armour which is examined with the Van der Meer formula. This formula requires amongst others the Iribarren number (ξ_m) as input variable. The basic variables for the Iribarren number are α , H_s , g and wave period. The required basic variables define the boundary conditions. By following this procedure only the required boundary conditions are derived and statistical correlation is taken into account.



Figure 14.2: Flowchart to determine boundary conditions

How to define the design requirements?

In this research the focus is on the ultimate limit state (ULS) of a rubble mound breakwater. So only the design requirements for the fully probabilistic calculation for the ULS are given. Two types of design requirements for the ULS are distinguished.

The overall design requirements which hold for the total structure and the specific design requirements for the design formulae. For the the specific design requirements only examples are given which are related to the case examined in this study. Important is that for the fully probabilistic calculation the safety factors (e.g. y_c) are excluded and the correct values for the safety parameters (e.g. c_{pl}) are chosen in the design formulae.

- 1. Overall design requirement:
 - Probability of failure total design lifetime (P_{f,sys,t_L}) based on:
 - Design lifetime (t_L)
 - Damage level for ULS (e.g. start of damage, intermediate damage or failure)
- 2. Specific design requirements which depends on set damage level
 - Damage factors S_d and N_{od}
 - Stability number F
 - Excluded safety factors (e.g. *y_c*) and parameters (e.g. *c_{pl}*)

The failure probability (P_{f,sys,t_L}) for the total design lifetime is determined according to the flowing three steps.

- 1. Determine target reliability based on codes and standards
- 2. Examining target reliability of similar structures which are known to have adequate safety.
- 3. More case specific estimate based on expert judgement

The failure probability strongly depends on the consequences of failure. The consequences of failure for the breakwater in the simplified case are less serious. The target probability of failure $(P_{f_{tl}})$ for the simplified case is set to 15% for serious damage (i.e. failure) for a lifetime of 50 years. This is based on the expert opinion (3), target reliability of similar structures (2) and the codes and standards (1).

In conclusion a failure probability $(P_{f_{il}})$ of 15% for complete failure is a realistic design requirement for a rubble mound breakwater where the consequence of failure are less serious. For rubble mound breakwaters with higher consequences of failure (e.g. dangerous goods in the harbour) the allowable probability of failure is in the order of 1% for failure.

Which variables have the most influence and are the most important for the fully probabilistic design??

A fully probabilistic calculation requires probability distributions for all the input variables. For the characteristics of the distributions (e.g. σ , μ) realistic assumptions are made. To see the influence of the input variables on the reliability of the structure multiple Monte Carlo analysis are executed, see section 12.4.1. From these results the following is concluded:

- Uncertainties in the boundary conditions, significant wave height (H_s) and soil parameters, result in an increased failure probability
- Having unreliable data of the available stones (D_{n50}, rho_s) result in significant increase of the failure probability
- When a design is made with a fully probabilistic reliability method the geometric variables should be specified accurately. Advised is to have a maximum deviation of 5% for the geometric variables.
- The water level has no large influence on the failure probability in this case.

In conclusion, make sure the uncertainties in the boundary conditions, especially for the hydraulic and geotechnical, are as small as possible. Often the uncertainty in the stone data is quite small so this won't be an issue in many cases. Furthermore the influence of the water level on the failure probability is in this case marginal however this could vary from case to case. At last a large spread in the geometric variables result in a increase in the failure probability. It is advised to specify that the geometric values (e.g. crest height) should have a maximum deviation of $\sigma = 5\%$.

In which aspects in the design process do statistical and physical correlation play a role?

Two types of correlation occur in the probabilistic design process:

- **Statistical correlation** Describing reality with a model requires gathering, processing and interpreting a lot of data. Statistical correlation describes the dependency between variables and datasets, which is the result of common usage or derivation of the same basic principles.
- **Physical correlation** This correlation occurs when multiple processes interact. For instance wind driven waves and wind driven up or down surge of the water level. Furthermore a rubble mound breakwater consists of multiple components. All these components are subject to changes and influence each other. For example as the roughness of the armour decreases, wave overtopping could increase. These processes are defined as physical correlation.

In this research different aspects are investigated to see where the statistical and physical correlation are present. From this research it is concluded that correlation occurs in the following aspects of the fully probabilistic design process:

Statistical correlation:

- Boundary conditions
- Failure mechanisms
- Multiple sections

Physical correlation:

- Boundary conditions
- Failure mechanisms

From the results showed in table 12.2 can be concluded that correlation plays a role in the determining the failure probability of the total system. A difference of 0.5% in failure probability of the structure for the design lifetime is seen for 'start of damage'. Although the influence is quite marginal in this case, it does result in a overestimation of the failure probability. Therefore the correlation should be taken into account. For the statistical correlation this is solved by performing one Monte Carlo simulation in the design process. The physical correlation is only partially included with the Monte Carlo procedure. Physical correlation where partly failure of a breakwater component increase the probability of failure of on other component is not included. In this research this is not investigated and only two possible solutions are given to included this physical correlation between failure mechanisms. The exact correlation should be further examined and this could be a topic for a next study.

How to determine the total probability of failure of the structure?

Determining the failure probability of the structure for the design life time is step 8 in the flowchart given in figure 14.1. From this research it is concluded that the a fully probabilistic calculation (level III) carried out with a Monte Carlo simulation is the most convenient way to calculated the probability of failure. The most important reason for this approach is that the correlation which occurs in the fully probabilistic design process is taken into account with this method. From the results given in chapter 12 it is concluded that at least $3 \cdot 10^5$ simulations should be carried out in this case to get a reliable answer. Furthermore all the boundary conditions should also be derived in a fully probabilistic way. This means that also the mathematical models (in this case SWAN, Delft3D and D-Geo Stability) should be included in the fully probabilistic calculation. In theory this is possible, however in practise it requires more investigation on how to implement these models into the fully probabilistic calculation. For the simplified case in this research it was possible to make a couple of simplifications to prevent the use of mathematical models in the fully probabilistic calculation:

The simplifications which are made in the research are listed below:

- Brettschneider formula instead of the model SWAN
- Basic equation of water level set-up instead of the model Delft3D
- 1 slip circle instead of the model D-Geo Stability

In conclusion it is possible to calculate the failure probability of a rubble mound breakwater with a fully probabilistic calculation (level III) via a Monte Carlo simulation. In this case simplifications could be applied for the models used in the semi-probabilistic calculation. Unfortunately there are also case where these simplification could not be applied. For example when nearshore wave process are present the formula of Brettschneider is not a good approximation and the model SWAN should be used. To perform a fully probabilistic calculation in other cases the implementation of mathematical models into the calculation should be further examined.

14.2. Recommendations

In this section recommendations are given for further research and how the results could be used. The recommendations are given in the sequence of the probabilistic design process. First a possible direction of further research on the allowable failure probability is discussed. Subsequently the limit states, failure mechanisms and design formula are evaluated. Next recommendations are given for further research on the boundary conditions. After that suggestions for further research on the probabilistic design calculation are given. The last recommendations are regarding optimization of the probabilistic design.

14.2.1. Allowable failure probability

One of the main problems encountered during the determination of the design requirements for the fully probabilistic calculation was the lack of reference cases. Designs for rubble mound breakwaters are not often made with a fully probabilistic calculation in practice. More references should be made by examining the failure probability of existing rubble mound breakwaters. Extra reference cases leads to a more reliable estimate for the allowable failure probability (i.e. target reliability). For further research the probability of failure for already existing rubble mound breakwaters with different consequences of failure should be calculated. Based on these results more specific guidelines for rubble mound breakwaters can be written depending on the consequences of failure (such as table 7.1).

14.2.2. Limit states and failure mechanisms

The examined simplified case consists of four failure mechanics which are all related to the Ultimate Limit State of the rubble mound breakwater. Additional failure mechanisms should be included to get a more reliable answer for the failure probability of the structure. Possible failure mechanisms which should be examined are failure of the crest element, berm stability or more case specific such as ice loads and ship collisions. An overview of all the possible failure mechanisms is given in appendix A. Furthermore the Serviceable Limit State has to be included to make a reliable estimate of the failure probability. Making a design just based on the ULS could result in an unacceptable failure probability for the SLS. For instance the amount of overtopping could be limit due to a road which is on the rubble mound breakwater, this is not taken into account in the ULS. Neglecting the SLS failure mechanisms could result in a design that does not meet all the design requirements. A next research could use the Matlab model which is made in this study to make a fully probabilistic calculation and implement all the relevant failure mechanisms for the SLS and ULS.

14.2.3. Design formulae

Multiple design formulae are applied in this research and most of them are derived empirical. This results in a certain degree of model uncertainty in these design formulae, this is included in the fully probabilistic calculation in this research. The results show that an increase in the uncertainties in these design formulae have a influence on the failure probability of the structure. Assuming the given uncertainties in The Rock Manual [2007] are correct it is not relevant to investigate the influence of increasing uncertainties in these formulae. The results show that reducing the uncertainty only has small influence on the probability of failure. Trying to reduce the uncertainties requires a lot of experiments (i.e. time and money) and does not necessarily result in a more reliable structure. So it is not recommended to do further investigation on how to reduces the uncertainties in the used design formulae (Van Gent and Pozueta, Van der Meer for toe and seaside armour).

14.2.4. Boundary conditions

The uncertainties in the boundary conditions have a large influence on the failure probability of the rubble mound breakwater. The uncertainties which are applied in this research are estimated uncertainties and most of them are not based on the actual conditions on the project location. To make a fully probabilistic calculation applicable in practise, further investigation is required on how to determine the uncertainties in the boundary conditions. If the uncertainties are too large, in for example the significant wave height($\sigma \ge 0.25 \cdot \mu$), a fully probabilistic is not feasible. Since large uncertainties result in a large probability of failure (see 12.6). Further investigation should focus on making a guideline when a fully probabilistic calculation is feasible depending on the available data for the boundary conditions at the project location. This could be done by examining existing rubble mound breakwaters and the available data at the time the structure was designed. The hypothesis is that for some project locations the uncertainties in the available data or even lack of data results in a failure probability which is too high. This makes a fully probabilistic calculation not practical and a semi-probabilistic design method is more convenient in these cases.

14.2.5. Physical Correlation

In this research a distinction is made between statistical and physical correlation. As concluded the statistical correlation is taken into account by carrying out one Monte Carlo simulation when calculating the failure probability. With this approach also part of the physical correlation is included, namely the situation where failure of a mechanism induces failure of an other mechanism. However the situation where partly failure of a mechanism induces failure of an other mechanism is not taken into account with this method. This aspect of physical correlation could results in an increase of the failure probability i.e. an overestimation of the structures reliability. Further research is required to investigate possible situation where this type of physical correlation could occur between failure mechanism. A good set up for a next study is making 3D model tests with a rubble mound breakwater. The physical correlation between the toe and the armour stability could for example be examined. By carrying out several simulations with varying damage level for the toe (N_{od}) and see for which load (H_s) the armour layer fails. From these results a correlation factor could be derived to describe the physical correlation between the toe and armour layer (see section 10.2.2).

14.2.6. Model simplifications

As mentioned in this research three simplifications are implemented to avoid using the models SWAN, Delft3D and D-Geo Stability. In this research these simplifications are suitable due to the conditions at the project location (see section 8.1.1). A possible direction for further research is to implement these mathematical models into the fully probabilistic calculation. The influences of these models on the probability of failure should be examined. Moreover solutions are needed to reduce the number of calculations to make sure that the computation time is within practical limits.

14.2.7. Probabilistic with approximations calculation method (level II)

In this research is concluded that for the simplified case a fully probabilistic (level III) calculation carried out with a Monte Carlo simulation (MC) is the most suitable. An other possibility was to apply a probabilistic calculation method with approximations (level II). This method gives a better insight in the sensitive of the failure input variables for the reliability functions via the α -values. This information could be used to make an effective optimization by for example investing time and money to reduce the uncertainties in the most sensitive input variables. However for this method the correlation coefficients between failure mechanisms should be known. Determining these correlation coefficients and subsequently performing a level II reliability method for the simplified case could be an interesting topic for further research. A comparison should be made between the level II and level III reliability method applied in this research. This comparison shows if a probabilistic calculation method with approximations offers extra information which can be used in the probabilistic design process.

14.2.8. Risk based calculation method (level IV)

A method is developed in this research to determine the design requirements for the fully probabilistic design. In this research is concluded that a risk based method (level IV) is the most accurate way to determine the target reliability (i.e. failure probability). The probability of failure is plotted against the total cost (construction cost and failure cost). The point with the lowest cost gives the most optimal target reliability. The construction costs are straightforward to determine however the costs of failure are more difficult to calculated. When this is done correctly the most cost optimum target reliability for a rubble mound breakwater is determined. A next study could examine the feasible of such a method in practise for a rubble mound breakwater. Subsequently the benefits of a risk based method (level IV) compared to a fully probabilistic method (level III) for a case should be compared.

14.2.9. Optimization

The semi-probabilistic design is used one on one in the fully probabilistic calculation. The results of the fully probabilistic calculation show that the semi-probabilistic design meets the design requirements as set for the fully probabilistic calculation. Subsequently an optimization is made by applying lower stone classes for the simplified case and this design also meets the set design requirements. Due to time restrictions only one optimization step is made in this research. Interesting topic for a next study is to change the design in such a way that the fully probabilistic design requirements are just met. Subsequently the benefits (for instance in terms of costs) of such an optimization should be examined.

Appendix



Fault tree



B

Hand calculation level 0 t/m III

In this appendix is shown that a level III calculation method with a Monte Carlo simulation for the probabilistic calculation is the most convenient way to calculate the failure probability of the structure. For each level (0 t/m III) this example is elaborated in the following sections based on the information given in Jonkman et al. [2015]. In the end a conclusion is given to explain the decision to use a fully probabilistic calculation (level III) with a Monte Carlo simulation.

In this hand calculation a case is assumed that consists of a structure with only 2 loads (S) and 1 strength (R) parameter. In this case the solicitations (S) on the structure have the following nominal values and standard deviations:

- $\mu_{S_1} = 100 \, kN$ $\sigma_{S_1} = 5 \, kN$
- $\mu_{S_2} = 50 \, kN$ $\sigma_{S_2} = 10 \, kN$

The required resistance (R) is calculated according to the chosen calculation method. The deviation in the resistance is assumed to be $\sigma = 2.5 kN$

In the the probabilistic calculations (level II and III) assumed is that the calculation failure probability is the yearly failure probability. The design requirement is that a yearly failure probability (P_f) of $3.3 \cdot 10^{-3}$ is allowed This results in a maximum failure probability (P_{f,sys,t_L}) of 15% for a design life time of $t_L = 50$ years. This is calculated with the Poisson distribution, see equation 4.15.

Furthermore this hand calculation is partly carried out in Matlab to get familiar with making a probabilistic calculation in Matlab.

B.1. Deterministic (Level 0)

When a deterministic method is used the nominal values of the solicitation and resistance are used. To guarantee the reliability of the system a global safety factor is added.

$$R_{nom} \ge \gamma_g \sum_{1}^{n} S_{nom} \tag{B.1}$$

This γ_g is often based on reference cases and are documented in codes and standards (e.g. Eurocode 7 [British Standards Institution, 2004]. For this case a global safety factor of $\gamma_g = 1.4$ is assumed. This results in a minimal required resistance for the structure of R = 210 when calculated with a deterministic calculation, see equation B.2.

$$R_{nom} \ge \gamma_g \cdot (S_1 + S_2) = 1.4 \cdot (100 + 50) = 210 \, kN \tag{B.2}$$

B.2. Semi-probabilistic (Level 1)

The semi-probabilistic method is a level I calculation. This is one of the most common ways to create a design based on regulations and guidelines. Partial safety are implemented for the strength (γ_r) as well as for the load (γ_s).

$$\frac{R_k}{\gamma_r} > \gamma_s S_k \tag{B.3}$$

These partial safety factors create a certain margin between the representative value of the strength and the loads. This margin ensures that the designed system is sufficiently reliable (figure B.1). [Jonkman et al., 2015]

Figure B.1: Probability density functions for load and strength Jonkman et al. [2015]

The value used for the load and strength are the representative values, which are calculated as follows:

$$R_k = \mu_R + k_R \cdot \sigma_R$$

$$S_k = \mu_S + k_S \cdot \sigma_S$$
(B.4)

The partial safety factors can be determined on the basis of standards and guidelines such as the Eurocode where the sensitivity factor (α) is given. These α -values are calculated via a level II design method. For the level I design method these α -values are standardized.

To select the representative values a classification for each resistance and solicitation should be made: permanent, variable or accidental actions. In this hand calculation is assumed that the resistance and solicitation are permanent. To take into account this variation in the level I method 5% - and 95% quantile values ($k_R = -1.645$ and $k_S = 1.645$) are used assuming a normal distribution.

$$R_{k} = \mu_{R} - 1.645 \cdot 2.5$$

$$S_{1_{k}} = 100 + 1.645 \cdot 5 = 108.23 \, kN$$

$$S_{2_{k}} = 50 + 1.645 \cdot 10 = 66.45 \, kN$$
(B.5)

The required nominal resistance (μ_R) is 178.79 kN given the values calculated in equation B.5.

B.3. Probabilistic with approximations (level II)

An third possibility is the use a level II method which is an probabilistic calculation with approximations. With the method the probability of failure can be determined by linearisation around the design point. For further information see section 4.2.3. A first order reliability method (FORM) is used to make the level II calculation. This calculation is made with a Matlab script of the OpenEarth toolbox of Deltares, see section B.6.

The outcome of this simulation is that the minimal resistance (R) should be 181.2 kN to have maximal probability of failure of 15% over the total lifetime (P_{f,sys,t_L}). Besides the values for the minimal resistance also the α -values are calculated: $\alpha_R = 0.22$, $\alpha_{S_1} = -0.44$ and $\alpha_{S_2} = 0.87$. This values show that the sensitivity of the load variable S_2 is the largest and has a large contribution to the failure probability.



This information could be used to try to reduce the uncertainty in this variable to increase the reliability of the structure. If the uncertainty is for example reduced by 25% to $\sigma_{S_2} = 7.5$ the total failure probability is $P_{f,sys,t_L} = 2\%$ instead of $P_{f,sys,t_L} = 15\%$

B.4. Fully probabilistic (Level III)

When a fully probabilistic method is applied, known as a level III method, the probability of failure is calculated exactly and directly linked to the reliability (i.e. probability of failure) of the element. To calculated the failure probability to methods are examined. One is the explicit calculation and a Monte Carlo analysis.

B.4.1. Explicit calculation

In this case an level III probabilistic calculation is easily done by hand via a explicit calculation. Because the variables are independently and normally distributed and the limit state is a linear function. First the reliability index β has to be calculated.

$$P_f = P\left[Z < 0\right] = \Phi\left[\frac{0 - \mu_Z}{\sigma_Z}\right] = \Phi(-\beta) \tag{B.6}$$

This results in $\beta = 2.72$ given the design requirement for the yearly failure probability $P_f = 3.3 \cdot 10^{-3}$.

The mean value (μ_Z) and the standard deviation (σ_Z) for the limit state function are determined as given in equations B.7 and B.8. A first estimate for the mean value of the resistance has to be made to see if the required β is obtained. From here on different iterations steps are made by changing μ_R to approximate the required β as close as possible. The fist estimate is $\mu = 180 kN$.

$$\mu_Z = \mu_R - \mu_{S_1} - \mu_{S_2} = 180 - 100 - 50 = 30 \, kN \tag{B.7}$$

$$\sigma_Z = \left(\sigma_R^2 + \sigma_{S_1}^2 + \sigma_{S_2}^2\right)^{0.5} = 2.5 + 5 + 10 = 11.46 \, kN \tag{B.8}$$

The first estimate of $\mu = 180 kN$ results in $\beta = 2.62$. This does not meet the set design requirement. After a few iterations steps the result is $\mu = 181.2 kN$ which results in $\beta = 2.72$.

B.4.2. Monte Carlo simulation

The Monte Carlo procedure is explained in detail in section 4.2.4. For this calculation a Matlab script is used from the OpenEarth toolbox of Deltares, see section B.6. The outcome of this simulation is that the minimal resistance (R) should be 181.1 kN to have maximal probability of failure of 15% over the total lifetime (P_{f,sys,t_l}) .

B.5. Conclusion

This simple hand calculation shows that reliability methods level II and III are both a good way to calculated the failure probability. However in the simplified case of the project Taman more reliability functions are present and the limit states functions are not linear. A level II FORM method and a explicit calculation for a level III become much more complex. In conclusion the Monte Carlo method suits the best for the fully probabilistic design calculation for the simplified case because:

- · No restrictions of number of variables and integrals due to complexity
- Statistical and physical correlation is taken into account without using a correlation coefficient
- · Influence of different failure mechanisms can be easily compared
- · Insight in governing failure situations for each mechanisms is obtained

The largest disadvantage of using a Monte Carlo simulation instead of a level II FORM is the lack of insight in the sensitivity of the input variables. In a level II probabilistic calculation the sensitivity of the input variables is calculated and given via the α -values. An overall comparison is given below:

Characteristics	Reliability method	
	Level II FORM	Level III Monte Carlo simulation
Calculate P_f per mechanism	X	Х
Insight in governing failure mechanism	Х	Х
Increasing number of variables is increasing complexity	Х	
Sensitivity of input variables	Х	
Correlation coefficient required	Х	
Statistical and physical correlation easily included		Х
Insight in governing failure situations		Х

Table B.1: Comparison between a level II FORM method and a Level III Monte Carlo simulation

B.6. Matlab script B.6.1. Reliability function

```
function [z] = prob_Simple_x2z(varargin)
1
2
  9% create samples-structure based on input arguments
3
  samples = struct(...
4
       'R', [],...
                            % [kg/m3] RhoS density sediment
5
       'S1', [],...
                             % [kg/m3] RhoS density sediment
6
       '<mark>S2</mark>', []);
                            % [kg/m3] RhoW density water
7
8
  samples = setproperty(samples, varargin{:});
9
10
  %% calculate z-values
11
  % pre-allocate z
12
  z = zeros(size(samples.R));
13
14
  % loop through all samples and derive z-values
15
  for i = 1:length(samples.R);
16
       z(i,1) = samples.R(i)- (samples.S1(i)+samples.S2(i));
17
  end
18
```

B.6.2. Executing MC and FORM analysis

```
%% Define stochast
1
  stochast = struct(...
2
       'Name', {
3
           'R'... % example stochastic variable "R" (Resistance or Strength)
4
           'S1'... % example stochastic variable "S" (Solicitation or Load)
5
           'S2'... % example stochastic variable "S" (Solicitation or Load)
6
           },...
7
       'Distr', {
8
           @norm_inv... % distribution function (of R)
9
           @norm_inv... % distribution function (of S1)
10
           @norm_inv.... % distribution function (of S2)
11
           } ,...
12
       'Params', {
13
```

```
\{181.2 \ 2.5\}\ldots % mu and sigma (of R)
14
            \{100 \ 5\}\dots
                              % mu and sigma (of S1)
15
                              % mu and sigma (of S2)
            \{50 \ 10\}\dots
16
            } ...
17
          );
18
19
20
<sup>21</sup> % Run the calculation using MC and FORM
22 % Define number of runs
_{23} N = 500000;
24
  % Run the calculation using Monte Carlo
25
   MCresult = MC(...
26
        'stochast', stochast,...
27
        'NrSamples', N,...
28
        'x2zFunction', @prob_Simple_x2z);
29
30
  % run the calculation using FORM
31
   resultFORM = FORM(\ldots)
32
        'stochast', stochast,...
33
        'x2zFunction', @prob_Simple_x2z);
34
35
36
  Pf_tlMC = 1-(1-MCresult.Output.P_f)^50;
37
38
  Pf_t Pf_t = 1 - (1 - resultFORM \cdot Output \cdot P_f)^{50};
39
```

\bigcirc

Process scheme



\square

Wind distributions



Figure D.1: Probability distribution extreme wind speeds, Direction: N



Figure D.2: Probability distribution extreme wind speeds, Direction: NNE





Figure D.3: Probability distribution extreme wind speeds, Direction: NE



Figure D.4: Probability distribution extreme wind speeds, Direction: ENE



Figure D.5: Probability distribution extreme wind speeds, Direction: E



Figure D.6: Probability distribution extreme wind speeds, Direction: ESE



Figure D.7: Probability distribution extreme wind speeds, Direction: SE



Figure D.8: Probability distribution extreme wind speeds, Direction: SSE





Figure D.9: Probability distribution extreme wind speeds, Direction: S





Figure D.10: Probability distribution extreme wind speeds, Direction: SSW

Figure D.11: Probability distribution extreme wind speeds, Direction: SW


Figure D.12: Probability distribution extreme wind speeds, Direction: WSW



Figure D.13: Probability distribution extreme wind speeds, Direction: W



Figure D.14: Probability distribution extreme wind speeds, Direction: WNW



Figure D.15: Probability distribution extreme wind speeds, Direction: NW



(a) Exponential distribution

(b) Weibull distribution

Figure D.16: Probability distribution extreme wind speeds, Direction: NNW

Uniform distribution random variable x

This procedure shows that a minimal of 10⁴ draws for the variable x should be carried out to approximate a more a less uniform distribution, see figure E.1

```
97% Drawing random value of x between 0 and 1, N-times
1
  N1 = 10000;
                     x1 = nan(size(N1));
2
  N2 = 100000;
                     x2 = nan(size(N2));
3
4
  for i = 1:N1
5
       x1(i,1) = rand(1,1);
6
  end
7
8
  for i = 1:N2
9
       x2(i,1) = rand(1,1);
10
  end
11
12
  %% Create histogram
13
  figure
14
  subplot(1,2,1)
15
  hist(x1,10)
16
   title('N = 10000'); xlabel('Value of x'); ylabel('Number of draws')
17
18
  subplot(1,2,2)
19
  hist(x2,10)
20
```

```
title('N = 100000'); xlabel('Value of x'); ylabel('Number of draws')
```



Figure E.1: Random simulation of variable x by Matlab

Hydraulic boundary conditions

F.1. Depth Chart Black Sea



Figure F.1: Depth Chart of the Black Sea [GeoGarage, 2016]

		Wind direction				
	N NNE NE ENE E					ESE
Wave direction relative to north [°]	342	360	16	30	48	78
Fetch length (F) [m]	16620	13555	15434	9316	2628	1388
Water depth Brettschneider (h) [m]	7.9	7.9	7.9	7.9	20	20
Mean (μ) factor $H_{2\%}/H_s$ [-]	1.301	1.301	1.301	1.301	1.301	1.301
Deviation (σ) factor $H_{2\%}/H_s$ [-]	0.051	0.051	0.051	0.051	0.051	0.051
Mean (μ) wave steepness (s) [-]	0.124	0.126	0.143	0.121	0.088	0.089
Deviation (σ) wave steepness (s) [-]	0.019	0.009	0.003	0.023	0.022	0.023
Mean (μ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.735	0.738	0.722	0.557	0.560	0.583
Deviation (σ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.034	0.015	0.025	0.067	0.060	0.056

F.2. Hydraulic boundary conditions factors

Table F.1: Wave characteristics for directions N till ESE

	Wind direction					
	ES ESS S SSW SW				WSW	
Wave direction relative to north [°]	210	214	215	217	220	225
Fetch length (F) [m]	7128	15222	25807	64891	76061	63586
Water depth Brettschneider (h) [m]	20	20	20	20	20	20
Mean (μ) factor $H_{2\%}/H_s$ [-]	1.301	1.301	1.301	1.301	1.301	1.301
Deviation (σ) factor $H_{2\%}/H_s$ [-]	0.051	0.051	0.051	0.051	0.051	0.051
Mean (μ) wave steepness (s) [-]	0.071	0.081	0.083	0.079	0.085	0.091
Deviation (σ) wave steepness (s) [-]	0.006	0.012	0.003	0.003	0.003	0.004
Mean (μ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.619	0.602	0.614	0.626	0.613	0.618
Deviation (σ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.045	0.018	0.026	0.014	0.020	0.009

Table F.2: Wave characteristics for directions ES till WSW

		Wind d	irection	
	W	WNW	WN	WNN
Wave direction relative to north [°]	237	267	303	323
Fetch length (F) [m]	47170	49090	22660	20048
Water depth Brettschneider (h) [m]	20	9.2	9.2	7.9
Mean (μ) factor $H_{2\%}/H_s$ [-]	1.301	1.301	1.301	1.301
Deviation (σ) factor $H_{2\%}/H_s$ [-]	0.051	0.051	0.051	0.051
Mean (μ) wave steepness (s) [-]	0.094	0.098	0.119	0.112
Deviation (σ) wave steepness (s) [-]	0.006	0.005	0.002	0.007
Mean (μ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.620	0.577	0.724	0.731
Deviation (σ) factor $T_{m0,2}/T_{m-1,0}$ [-]	0.007	0.036	0.044	0.041

Table F.3: Wave characteristics for directions W till WNN

G

Matlab scripts

G.1. Failure probability total system

```
%% Monte Carlo Simulation
1
  % Define number of Monte Carlo Simulations
2
  N = 1;
3
5 % Run Monte Carlo simulation
  Monte_Carlo (N)
6
  9% Calculate probability of failure per year
8
  \% 1 = failure, 0 = non-failure
9
10
  % Calculate prob failure Van der Meer
11
<sup>12</sup> idFail(:,1) = z_vdm(:,1) < 0;
<sup>13</sup> Pf_z_vdm
                  = sum(idFail(:,1))/size(z_vdm,1);
14
  % Calculate prob failure Toe
15
  idFail(:,2) = z_t(:,1) < 0;
16
                = sum(idFail(:,2))/size(z_t,1);
  Pf_z_t
17
18
  % Calculate prob failure Rear side stability
19
  idFail(:,3) = z_r(:,1) < 0;
20
21 Pf_z_r
                = sum(idFail(:,3))/size(z_r,1);
22
  % Calculate prob failure Macro stability
23
 idFail(:,4) = z_bis(:,1) <1;
24
                  = sum(idFail(:,4))/size(z_bis,1);
  Pf_z_bis
25
26
  % Total failure probability; If assumed non correlated
27
  Pf_total_non = Pf_z_vdm + Pf_z_t + Pf_z_r + Pf_z_bis;
28
29
  % Total failure probability; Including statistical correlation
30
  A = nan(size(N));
31
  for i = 1:N
32
       if sum(idFail(i,:)) == 0;
33
           A(i,1) = 0;
34
       else
35
           A(i,1) = 1;
36
37
       end
 end
38
```

```
<sup>39</sup>
<sup>40</sup> Pf_total_cor = sum(A(:,1))/size(A,1);
<sup>41</sup>
<sup>42</sup> % Calculated probability of failure total lifetime
<sup>43</sup> % Define design lifetime (dlt)
<sup>44</sup> dlt = 50;
<sup>45</sup>
<sup>46</sup> % Poission distribution to calculate probability failure design lifetime
<sup>47</sup> Pf_total_dlt = 1-(1-Pf_total_cor)^dlt;
```

G.2. Monte Carlo simulation

```
<sup>1</sup> function Monte_Carlo (N)
```

```
2 Windbins = textread('C:\Users\Siemen\Documents\TU\Master\Afstuderen\06 Rapport\11
Matlab\03 Input excels\01 Input for Matlab\Windbins.txt'); %#ok<DTXTRD>
```

```
3 % Define stochast
```

```
4 stochast = struct(...
```

5	'Name', {	
6	'bl'	% Bed level with respect to BD
7	'b'	% Ratio between Hs and H2%
8	's1 '	% Steepness Hs/Tm20, bin1
9	's2'	% Steepness Hs/Tm20, bin2
10	's3'	% Steepness Hs/Tm20, bin3
11	's4 '	% Steepness Hs/Tm20, bin4
12	's5'	% Steepness Hs/Tm20, bin5
13	's6'	% Steepness Hs/Tm20, bin6
14	's7'	% Steepness Hs/Tm20, bin7
15	's8'	% Steepness Hs/Tm20, bin8
16	's9 '	% Steepness Hs/Tm20, bin9
17	's10'	% Steepness Hs/Tm20, bin10
18	's11 '	% Steepness Hs/Tm20, bin11
19	's12 '	% Steepness Hs/Tm20, bin12
20	's13 '	% Steepness Hs/Tm20, bin13
21	's14 '	% Steepness Hs/Tm20, bin14
22	's15'	% Steepness Hs/Tm20, bin15
23	's16'	% Steepness Hs/Tm20, bin16
24	'al '	% Ratio between Tm10/Tm20, bin1
25	′a2′	% Ratio between Tm10/Tm20, bin2
26	′a3′	% Ratio between Tm10/Tm20, bin3
27	´a4´	% Ratio between Tm10/Tm20, bin4
28	´a5´	% Ratio between Tm10/Tm20, bin5
29	´a6´	% Ratio between Tm10/Tm20, bin6
30	í a7 í	% Ratio between Im10/Im20, bin7
31	´a8´	% Ratio between Im10/Im20, bin8
32	´a9´	% Ratio between Im10/Im20, bin9
33		% Ratio between Imi0/Im20, bin10
34		% Ratio between Imi0/Im20, bin11
35	a12	% Ratio between Imio/Im20, Din12
36	al3	% Ratio between Imio/Im20, bin13
37	a14	% Ratio between Imio/Im20, bin14
38	al5	% Ratio between Imio/Im20, Dinis
39	alb	% Ratio between imi0/im20, bini6
40	\tan_a	% fan Outer stope
41	St_f1	% Storm duration
42	A_Stf	⁷⁰ Aligie of structure relative to north
43	rnow	% Density of water
44	rnos	% Density of stone

45	'g'	% Gravitational acceleration
46	'Sd'	% ARMOUR STABILITY: Damage factor
47	'P'	% Notional permeability parameter
48	'c_pld '	% Plunging deep water coefficient = 6.2
49	'c_sd '	% Surging deep water coefficient= 1
50	'c_pls'	% Plunging shallow water coefficient = 8.4
51	'c_ss'	% Plunging shallow water coefficient = 1.3
52	'x_cw'	% Correction factor armour stones = 1.05 Wolters
53	'd_a'	% Stone size armour
54	'Nod'	% TOE STABILITY: Damage factor toe berm
55	'x cg'	% Correction factor toe = 0.6 Galland
56	'h toe'	% Height of toe
57	'd t'	% Stone size toe
58	'c0'	% REAR-SIDE SLOPE STABILITY: coefficient c0
59	'c1 '	% Coefficient cl
60	'gamma f'	% Roughness of seaward slope, rough=0.55 smooth = 1
61	'gamma fc'	% Roughnes of crest, armour = 0.55 smooth imper = 1
62	'cr l'	% Crest level
63	'cr lr'	% Crest level rear side
64	'B c'	% Crest width
65	'tan ai'	% Tan Inner slope
66	'd r'	% Stone size rear side
67	'gamma a un'	% MACRO STABILITY: Unsaturated denisty armour layer [N/m2]
69	'gamma a s'	% Saturated denisty armour layer [N/m2]
60	'gamma a s'	% Saturated denisty quarry run [N/m2]
70	'nhi a'	% Angle of internal friction armour layer [Rad]
70	'nhi a'	% Angle of internal friction quarry run [Rad]
71	pm_q	70 migie of meetinal meetion quarty fun [hau]
72	'Distr' {	
73	@norm_inv	% Bed level with respect to BD
74	@norm_inv	% Bettio between Hs and H2%
75	@norm_inv	% Steenness Hs/Tm20 hin1
70	@norm_inv	% Steepness Hs/Tm20, bin2
77	@norm_inv	% Steepness Hs/Tm20, bin2
78	@norm_inv	% Steepness $Hs/Tm20$, $bin/$
79	@norm_inv	% Steepness Hs/Tm20, bin5
80	@norm_inv	% Steepness Hs/Tm20, bin6
81	@norm_inv	% Steepness Hs/Tm20, bin7
82	@norm_inv	% Steepness Hs/Tm20, bing
83	@norm_inv	% Steepness Hs/Tm20, bin0
84	@norm_inv	% Steepness Hs/Tm20, bin10
85	@norm_inv	% Steephess Hs/III20, DIII10
86	@norm_inv	% Steephess Hs/III20, DHIII
87	@norm_inv	% Steepness Hs/III20, DIII12
88		% Steepness Hs/Im20, bin13
89	@norm_inv	% Steepness Hs/Im20, bin14
90	@norm i	$\%$ Steepness $\Pi s/\Pi \Pi 20$, $D\Pi \Pi 5$
91	@norm_inv	% Steepness Hs/1m20, Din16
92	@norm_inv	% Kallo Delween Imul/Im20, Dini
93	@norm_inv	% Katio between Imul/Im20, bin2
94	@norm_inv	% Ratio between Im01/Im20, bin3
95	@norm_inv	% Ratio between Im01/Im20, bin4
96	@norm_inv	% Ratio between Im01/Im20, bin5
97	@norm_inv	% Ratio between Tm01/Tm20, bin6
98	@norm_inv	% Ratio between Tm01/Tm20, bin7
99	@norm_inv	% Ratio between Tm01/Tm20, bin8
100	@norm_inv	% Ratio between Tm01/Tm20, bin9

{Windbins(7,11) Windbins(8,11)}...

	@norm_inv	% Ratio between Tn	n01/Tm20, bin10
	@norm_inv	% Ratio between Tn	n01/Tm20, bin11
	@norm_inv	% Ratio between Tn	n01/Tm20, bin12
	@norm_inv	% Ratio between Tn	n01/Tm20, bin13
	@norm_inv	% Ratio between Tn	n01/Tm20, bin14
	@norm_inv	% Ratio between Tn	n01/Tm20, bin15
	@norm_inv	% Ratio between Tn	n01/Tm20, bin16
	@norm_inv	% Tan Outer slope	
	@deterministic	% Storm duration	
	@deterministic	% Angle of structu	re relative to north
	@norm_inv	% Density of water	r
	@norm_inv	% Density of stone	
	@deterministic	% Gravitational ac	cceleration
	@deterministic	% ARMOUR STABILITY: Da	amage factor
	@deterministic	% Notional permea	ability parameter
	@norm_inv	% Plunging deep wa	ater coefficient = 6.2
	@norm_inv	% Surging deep wat	ter coefficient= 1
	@norm_inv	% Plunging shallow	water coefficient = 8.4
	@norm_inv	% Plunging shallow	water coefficient = 1.3
	@norm_inv	% Correction facto	or armour stones = 1.05 Wolters
	@norm_inv	% Stone size armou	ur
	@deterministic	% TOE STABILITY: Damage	ge factor toe berm
	@norm_inv	% Correction facto	or toe = 0.6 Galland
	@norm_inv	% Height of toe	
	@norm_inv	% Stone size toe	
	@deterministic	% REAR-SIDE SLOPE STAI	BILITY: coefficient c0
	@deterministic	% Coefficient cl	
	@norm_inv	% Roughness of sea	ward slope, rough=0.55 smooth = 1
	@norm_inv	% Roughnes of cres	st, armour = 0.55 smooth imper = 1
	@norm_inv	% Crest level	
	@norm_inv	% Crest level rear	side
	@norm_inv	% Crest width	
	@norm_inv	% Tan Inner slope	
	@norm_inv	% Stone size rear	side
	@norm_inv	% MACRO STABILITY: Un	saturated denisty armour layer [N/
	m2]		
	@norm_inv	% Saturated denist	ty armour layer [N/m2]
	@norm_inv	% Saturated denist	ty quarry run [N/m2]
	@norm_inv	% Angle of interna	al friction armour layer [Rad]
	@norm_inv	% Angle of interna	al friction quarry run [Rad]
	},		
'Pa	irams', {		
	$\{-8 \ 0.05\}\dots$		% Bed level with respect to BD
	{1.30113 0.051}		% Ratio between Hs and H2%
	{Windbins(7,1) Wind	lbins (8,1) }	% Steepness Hs/Tm20, bin1
	{Windbins(7,2) Wind	lbins (8,2) }	% Steepness Hs/Tm20, bin2
	{Windbins(7,3) Wind	$1bins(8,3)\}$	% Steepness Hs/Tm20, bin3
	{Windbins(7,4) Wind	$1bins(8,4) \} \dots$	% Steepness Hs/Tm20, bin4
	{Windbins(7,5) Wind	lbins (8,5) }	% Steepness Hs/Tm20, bin5
	{Windbins(7,6) Wind	lbins (8,6) }	% Steepness Hs/Tm20, bin6
	{Windbins(7,7) Wind	lbins (8,7) }	% Steepness Hs/Tm20, bin7
	{Windbins(7,8) Wind	lbins (8,8) }	% Steepness Hs/Tm20, bin8
	{Windbins(7,9) Wind	lbins(8,9)}	% Steepness Hs/Tm20, bin9
	{Windbins(7,10) Wir	ndbins(8,10)}	% Steepness Hs/Tm20, bin10

% Steepness Hs/Tm20, bin11

156	{Windbins(7,12) Win	ndbins(8,12)}	% Steepness Hs/Tm20, bin12
157	{Windbins(7,13) Win	ndbins(8,13)}	% Steepness Hs/Tm20, bin13
158	{Windbins(7,14) Win	ndbins(8,14)}	% Steepness Hs/Tm20, bin14
159	{Windbins(7,15) Wir	ndbins(8,15)}	% Steepness Hs/Tm20, bin15
160	{Windbins(7,16) Windbins(7,16) Windb	ndbins(8,16)}	% Steepness Hs/Tm20, bin16
161	{Windbins(9,1) Wind	dbins(10,1)}	% Ratio between Tm01/Tm20, bin1
162	{Windbins(9,2) Wind	dbins(10,2)}	% Ratio between Tm01/Tm20, bin2
163	{Windbins(9,3) Wind	dbins(10,3)}	% Ratio between Tm01/Tm20, bin3
164	{Windbins(9,4) Wind	dbins(10,4)}	% Ratio between Tm01/Tm20, bin4
165	{Windbins(9,5) Wind	lbins(10,5)}	% Ratio between Tm01/Tm20, bin5
166	{Windbins(9,6) Wind	lbins(10,6) }	% Ratio between Tm01/Tm20, bin6
167	{Windbins(9,7) Wind	dbins(10,7)}	% Ratio between Tm01/Tm20, bin7
168	{Windbins(9,8) Wind	dbins(10,8)}	% Ratio between Tm01/Tm20, bin8
169	{Windbins(9,9) Wind	dbins(10,9)}	% Ratio between Tm01/Tm20, bin9
170	{Windbins(9,10) Win	ndbins(10,10) }	% Ratio between Tm01/Tm20,
	bin10		
171	{Windbins(9,11) Wir	ndbins(10,11) }	% Ratio between Tm01/Tm20,
	bin11		
172	{Windbins(9,12) Win	ndbins(10,12) }	% Ratio between Tm01/Tm20,
	bin12		
173	{Windbins(9,13) Wir	ndbins(10,13) }	% Ratio between Tm01/Tm20,
	bin13		
174	{Windbins(9,14) Wir	ndbins(10,14) }	% Ratio between Tm01/Tm20,
	bin14		
175	{Windbins(9,15) Wir	ndbins(10,15) }	% Ratio between Tm01/Tm20,
	bin15		
176	{Windbins(9,16) Wir	ndbins(10,16) }	% Ratio between Tm01/Tm20,
	bin16		
177	$\{0.4 \ 0.02\}\dots$	5 Tan Outer slope sigma =	1 degree
178	{3} %	Storm duration	
179	{337} %	Angle of structure relat	ive to north
180	{1014 1} %	5 Density of water	
181	{2600 15} %	Density of stone	
182	{9.81} %	Gravitational acceleration	on
183	{8} % A	RMOUR STABILITY: Damage f	actor
184	{0.5} %	Notional permeability pa	rameter
185	$\{6, 2, 0, 4\}$	Plunging deen water coef	
	(0.2) 0.1)	i fullging uccp watch coch	111C1ent = 6.2 - 0.4 (5.5 11m1t)
	value)	riunging deep water even	fincient = 6.2 - 0.4 (5.5) fimit
186	value) {1 0.08} %	5 Surging deep water coeff	Ficient = $1-0.08$ (0.87 limit value
186	value) {1 0.08} %	5 Surging deep water coeff	Fricient = $6.2 - 0.4$ (5.5 limit Sicient= $1 - 0.08$ (0.87 limit value
186	value) {1 0.08} %) {8.4 0.7} %	5 Surging deep water coeff 5 Plunging shallow water c	Ficient = $6.2-0.4$ (5.5 limit ficient= $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit
186 187	value) {1 0.08} %) {8.4 0.7} % value)	5 Surging deep water coeff 5 Plunging shallow water c	frictent = $6.2-0.4$ (5.5 limit ficient= $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit
186 187 188	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} %	5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c	Fricient = $6.2-0.4$ (5.5 limit Sicient= 1-0.08 (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05
186 187 188	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value)	5 Flunging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c	Fricient = $6.2-0.4$ (5.5 limit Ficient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05
186 187 188	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} %	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 5 Correction factor armour	Frictent = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters
186 187 188 189 190	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} %	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 6 Plunging shallow water c 6 Correction factor armour 6 Stone size armour 60–300	Frictent = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 coefficient = 1.05 Wolters bkg
186 187 188 189 190	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} %	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 6 Correction factor armour 6 Stone size armour 60–300 0E STABILITY: Damage fact	Frictent = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters bkg or toe berm
186 187 188 189 190 191	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03}	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 5 Correction factor armour 6 Stone size armour 60–300 OE STABILITY: Damage fact % Correction factor toe =	Finite field = $6.2-0.4$ (5.5 limit Ficient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters kg or toe berm = 0.6 Galland
186 187 188 189 190 191 192 193	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115}	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 5 Correction factor armour 5 Stone size armour 60–300 OE STABILITY: Damage fact % Correction factor toe = % Height of toe	Fricient = $6.2-0.4$ (5.5 limit Scient= $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters bkg or toe berm = 0.6 Galland
186 187 188 189 190 191 192 193 194	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02}	5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 5 Plunging shallow water c 6 Correction factor armour 6 Stone size armour 60–300 OE STABILITY: Damage fact 7 Correction factor toe = 7 Height of toe 7 Stone size toe 60–300kg	Fricient = $6.2-0.4$ (5.5 limit Scient= $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters lkg or toe berm = 0.6 Galland
186 187 188 189 190 191 192 193 194 195	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02} {1.45} % R	5 Flunging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 6 Plunging shallow water c 6 Correction factor armour 7 Stone size armour 60–300 70E STABILITY: Damage fact 7% Correction factor toe = 7% Height of toe 7% Stone size toe 60–300kg 70 E STABILITY:	Fricient = $6.2-0.4$ (5.5 limit Scient= 1-0.08 (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 coefficient = 1.05 Wolters kg or toe berm = 0.6 Galland coefficient c0
 186 187 188 189 190 191 192 193 194 195 196 	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02} {1.45} % F {5.1} %	5 Flunging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 6 Plunging shallow water c 7 Correction factor armour 7 Stone size armour 60–300 7 Correction factor toe = 7 Height of toe 7 Stone size toe 60–300kg 7 EAR-SIDE SLOPE STABILITY: 5 Coefficient cl	Frictent = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 stones = 1.05 Wolters kg or toe berm = 0.6 Galland Coefficient c0
186 187 188 189 190 191 192 193 194 195 196	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02} {1.45} % F {5.1} %	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 6 Plunging shallow water c 7 Correction factor armour 7 Stone size armour 60–300 7 OE STABILITY: Damage fact 7 Correction factor toe = 7 Height of toe 7 Stone size toe 60–300kg 7 EAR-SIDE SLOPE STABILITY: 8 Coefficient c1 8 Roughness of seaward slow	Finite = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 coefficient = 1.05 Wolters by or toe berm = 0.6 Galland coefficient c0 pe, rough=0.55 smooth = 1
 186 187 188 189 190 191 192 193 194 195 196 197 198 	<pre>value) {1 0.08} %</pre>	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 6 Correction factor armour 6 Stone size armour 60–300 OE STABILITY: Damage fact 7% Correction factor toe = 7% Height of toe 7% Stone size toe 60–300kg 7% Stone size toe 60–300kg 7% Stone size toe 60–300kg 7% Stone size toe 60–300kg 7% Coefficient cl 7% Coefficient cl 7% Roughness of seaward slop 7% Roughnes of crest, armou	Finctent = $6.2-0.4$ (5.5 limit Sicient = $1-0.08$ (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 coefficient = 1.05 Wolters by or toe berm = 0.6 Galland coefficient c0 pe, rough= 0.55 smooth = 1 ur = 0.55 smooth imper = 1
 186 187 188 189 190 191 192 193 194 195 196 197 198 199 	<pre>value) {1 0.08} % i) {1 0.08} % i) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02} {1.45} % F {5.1} % {1.0 0.025} % {3.7 0.185} %</pre>	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 6 Correction factor armour 6 Stone size armour 60–300 0E STABILITY: Damage fact 7% Correction factor toe = 7% Height of toe 7% Stone size toe 60–300kg 7% Stone size toe 60–300kg 7% Stone size toe 60–300kg 7% Coefficient c1 6 Roughness of seaward slop 5 Roughnes of crest, armou 5 Crest level	Frictent = $6.2-0.4$ (5.5 limit Scient= 1-0.08 (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 c stones = 1.05 Wolters log or toe berm = 0.6 Galland coefficient c0 pe, rough=0.55 smooth = 1 ar = 0.55 smooth imper = 1
 186 187 188 189 190 191 192 193 194 195 196 197 198 199 200 	value) {1 0.08} %) {8.4 0.7} % value) {1.3 0.15} % limit value) {1.05 0.05} % {0.464 0.023} % {5} % T {0.6 0.03} {2.3 0.115} {0.41 0.02} {1.45} % F {5.1} % {1.0 0.025} % {3.7 0.185} %	5 Surging deep water coeff 5 Surging deep water coeff 5 Plunging shallow water c 5 Plunging shallow water c 5 Correction factor armour 5 Stone size armour 60–300 0E STABILITY: Damage fact 7% Correction factor toe = 7% Height of toe 7% Stone size toe 60–300kg 75 Coefficient cl 76 Coefficient cl 76 Roughness of seaward slop 76 Roughnes of crest, armou 76 Crest level 76 Crest level	Fricient = $6.2-0.4$ (5.5 limit Scient= 1-0.08 (0.87 limit value coefficient = $8.4-0.7$ (7.25 limit coefficient = $1.3-0.15$ (1.05 coefficient = $1.3-0.$

```
\{2.4 \quad 0.12\}\dots
                                % Crest width
201
            \{0.5 \ 0.017\}\dots
                               % Tan Inner slope
202
            {0.149 0.0075}... % Stone size rear side 10-40kg
203
            {11500 285}... % MACRO STABILITY: Unsaturated density armour layer [N/m3]
204
                                 % Saturated density armour layer [N/m3]
            \{17000 \ 425\}\dots
205
            {20500 515}...
                                 % Saturated density quarry run [N/m3]
206
            {0.698 0.01745}... % Angle of internal friction armour layer [Rad]
207
            {0.698 0.01745}... % Angle of internal friction quarry run [Rad]
208
            } ...
209
       );
210
211
   %% Run the calculation using Monte Carlo
212
   MonteCarlo = MC(...
213
        'stochast', stochast,...
214
        'NrSamples', N,...
215
        'x2zFunction', @Reliability_functions);
216
217
   end
218
```

G.3. Reliability functions

1 function [z] = Reliability_functions(varargin)

2 97% Creates samples-structure based on input arguments

3	samples = struct(
4	'bl', [],	% Bed level with respect to BD
5	'b', [],	% Ratio between Hs and H2%
6	's1', [],	% Steepness Hs/Tm20, bin1
7	's2', [],	% Steepness Hs/Tm20, bin2
8	's3', [],	% Steepness Hs/Tm20, bin3
9	's4', [],	% Steepness Hs/Tm20, bin4
10	's5', [],	% Steepness Hs/Tm20, bin5
11	's6', [],	% Steepness Hs/Tm20, bin6
12	's7', [],	% Steepness Hs/Tm20, bin7
13	's8', [],	% Steepness Hs/Tm20, bin8
14	's9', [],	% Steepness Hs/Tm20, bin9
15	's10', [],	% Steepness Hs/Tm20, bin10
16	's11', [],	% Steepness Hs/Tm20, bin11
17	's12', [],	% Steepness Hs/Tm20, bin12
18	's13', [],	% Steepness Hs/Tm20, bin13
19	's14', [],	% Steepness Hs/Tm20, bin14
20	's15', [],	% Steepness Hs/Tm20, bin15
21	's16', [],	% Steepness Hs/Tm20, bin16
22	'al', [],	% Ratio between Tm10/Tm20, bin1
23	'a2', [],	% Ratio between Tm10/Tm20, bin2
24	'a3', [],	% Ratio between Tm10/Tm20, bin3
25	'a4', [],	% Ratio between Tm10/Tm20, bin4
26	'a5', [],	% Ratio between Tm10/Tm20, bin5
27	'a6', [],	% Ratio between Tm10/Tm20, bin6
28	'a7', [],	% Ratio between Tm10/Tm20, bin7
29	'a8', [],	% Ratio between Tm10/Tm20, bin8
30	'a9', [],	% Ratio between Tm10/Tm20, bin9
31	'alO', [],	% Ratio between Tm10/Tm20, bin10
32	'all', [],	% Ratio between Tm10/Tm20, bin11
33	'a12', [],	% Ratio between Tm10/Tm20, bin12
34	'a13', [],	% Ratio between Tm10/Tm20, bin13
35	'al4', [],	% Ratio between Tm10/Tm20, bin14
36	'a15', [],	% Ratio between Tm10/Tm20, bin15

'a16', [],	% Ratio between Tm10/Tm20, bin16
'tan_a', [],	% Tan Outer slope
'St_h', [],	% Storm duration
'A_str', [],	% Angle of structure relative to north
'rhow', [],	% Density of water
'rhos', [],	% Density of stone
'g', [],	% Gravitational acceleration
'Sd', [],	% ARMOUR STABILITY: Damage factor
'P', [],	% Notional permeability parameter
'c_pld', [],	% Plunging deep water coefficient = 6.2
'c_sd', [],	% Surging deep water coefficient= 1
'c_pls', [],	% Plunging shallow water coefficient = 8.4
'c_ss', [],	% Plunging shallow water coefficient = 1.3
'x_cw', [],	% Correction factor armour stones = 1.05 Wolters
'd_a', [],	% Stone size armour
'Nod', [],	% TOE STABILITY: Damage factor toe berm
'x_cg', [],	% Correction factor toe = 0.6 Galland
'h_toe', [],	% Height of toe
'd_t', [],	% Stone size toe
'c0', [],	% REAR-SIDE SLOPE STABILITY: coefficient c0
'c1', [],	% Coefficient cl
'gamma_f',[],	% Roughness of seaward slope, rough=0.55 smooth = 1
'gamma_fc', [],	% Roughnes of crest, armour = 0.55 smooth imper = 1
'cr_l', [],	% Crest level
'cr_lr', [],	% Crest level rear side
'B_c', [],	% Crest width
'tan_ai', [],	% Tan Inner slope
'd_r', [],	% Stone size rear side
'gamma_a_un ',[], m2]	% MACRO STABILITY: Unsaturated denisty armour layer [N/
'gamma_a_s',[],	% Saturated denisty armour layer [N/m2]
'gamma q_s', [],	% Saturated denisty quarry run [N/m2]
'phi a',[],	% Angle of internal friction armour layer [Rad]
'phi_q', []);	% Angle of internal friction quarry run [Rad]
	0 1 7
<pre>samples = setproperty(sample)</pre>	es, varargin {:});
<pre>%% Calculate N number of sin [HyBC] = HydraulicBC (s assignin('base', 'HyBC'</pre>	nulations Windspeed, wave height and angle ize(samples.bl)); , HyBC);
97% Perform Monte Carlo by lo	pop through all samples and derive z-values
% Preallocate 7 values to s	afe time
z = nan(size(samples h)))):
z vdm = nan(size(samples, b))	s, bl));
$z_{\rm t} = nan(size(samples))$	bl)):
$z_r = nan(size(samples))$	bl)):
z his = nan(size(samples))	s. bl)):
$H_2 = nan(size(samples))$	bl)):
Tm 02 = nan(size(samples))	s. bl)):
$Tm_{10} = nan(size(sample))$	s. bl)):
in_io nun(orzo(oumpie	

```
88
89 for i = 1:length(samples.bl);
```

```
90 98% Z-function to run the openearth tool
```

```
z(i,1) = \text{samples.bl}(i);
```

```
92
   98% General Variables which are used in multiple reliability functions
93
       % Read Significant wave angle out HyBC
94
       Wave_ang = HyBC(i,2);
95
96
       % Read Significant wave height out HyBC
97
       H_s = HyBC(i, 6);
98
99
       % Define which bin
100
       i = HyBC(i, 4);
101
102
       if j == 1;
103
            s = samples.s1(i); a = samples.a1(i);
104
            elseif j == 2;
105
            s = samples.s2(i); a = samples.a2(i);
106
            elseif j == 3;
107
            s = samples.s3(i);
                                  a = samples.a3(i);
108
            elseif j == 4;
109
            s = samples.s4(i); a = samples.a4(i);
110
            elseif j == 5;
111
            s = samples.s5(i); a = samples.a5(i);
112
            elseif i == 6;
113
            s = samples.s6(i); a = samples.a6(i);
114
            elseif j == 7;
115
            s = samples.s7(i); a = samples.a7(i);
116
            elseif j == 8;
117
            s = samples.s8(i); a = samples.a8(i);
118
            elseif j == 9;
119
            s = samples.s9(i); a = samples.a9(i);
120
            elseif j == 10;
121
            s = samples.s10(i); a = samples.a10(i);
122
            elseif j == 11;
123
            s = samples.sl1(i); a = samples.al1(i);
124
            elseif j == 12;
125
            s = samples.s12(i); a = samples.a12(i);
126
            elseif j == 13;
127
            s = samples.s13(i); a = samples.a13(i);
128
            elseif j == 14;
129
            s = samples.s14(i); a = samples.a14(i);
130
            elseif j == 15;
131
            s = samples.s15(i); a = samples.a15(i);
132
            elseif i == 16;
133
            s = samples.s16(i); a = samples.a16(i);
134
       end
135
136
       % Design water level (mu, sigma = mu * 0.1)
137
       DWL = HyBC(i, 5);
138
139
       % Wave height exceeded by 2% of the waves depending on wave height
140
       H_2(i,1) = H_s * samples.b(i);
141
142
       % Mean wave period depending on wave height
143
       Tm_02(i,1) = sqrt((H_s/s)*(2*pi/samples.g(i)));
144
145
       % Spectral wave period
146
       Tm_{10}(i,1) = Tm_{02}(i,1)/a;
147
```

```
148
        % Surf similarity parameter SHW: Used in VDM and Rear-side
149
        xi_s1 = samples.tan_a(i) / (sqrt((2*pi*H_s) / (samples.g(i)*Tm_10(i,1)^2)));
150
151
        % Water depth
152
        h = DWL - samples.bl(i);
153
154
       % Number of waves
155
       N = samples.St_h(i) * 3600/Tm_02(i,1);
156
157
        % Angle of attack
158
        if (abs(samples.A_str(i)-Wave_ang)>180)
159
160
             A_at = 360 - (abs(samples.A_str(i)-Wave_ang));
161
        else
162
             A_at = (abs(samples.A_str(i)-Wave_ang));
163
        end
164
165
        % Determine delta
166
        delta = samples.rhos(i)/samples.rhow(i)-1;
167
168
   %% Van der Meer formula
169
170
        z_vdm(i,1) = Vandermeer(H_s,h,H_2(i,1), samples.c_pld(i), samples.c_sd(i), samples
171
             .P(i), samples. tan_a(i), samples. g(i), Tm_02(i, 1), samples. Sd(i), N, samples. c_pls(i)
            i), samples.c_ss(i),...
            A_at, samples.x_cw(i), samples.rhos(i), samples.rhow(i), samples.d_a(i), xi_s1);
172
173
   %% Toe stability
174
175
        z_t(i,1) = Toe(A_at,H_s,samples.x_cg(i),samples.d_t(i),samples.h_toe(i),h,
176
            samples.Nod(i),delta);
177
   %% Rear side stability
178
179
        z_r(i,1) = \text{Rearside} (A_at, \text{samples.gamma}_f(i), \text{samples.c1}(i), \text{samples.c0}(i), \text{xi}_s1,
180
            H_s,DWL, samples. cr_l(i), samples. cr_lr(i), samples.g(i),...
             samples.gamma_fc(i), samples.B_c(i), samples.d_r(i), samples.Sd(i), Tm_10(i,1),
181
                 delta, samples.tan_ai(i),N, j);
182
   %% Marco Stability with Bishop
183
184
        z_bis(i,1) = Bishop (samples.gamma_a_un(i), samples.gamma_a_s(i), samples.
185
            gamma_q_s(i), samples.rhow(i), samples.phi_a(i), samples.phi_q(i), DWL);
186
   end
187
188
   %% Assigning variable to workspace
189
   assignin('base', 'z_vdm', z_vdm);
190
   assignin('base', 'z_t', z_t);
191
   assignin('base', 'z_r', z_r);
192
   assignin('base', 'z_bis', z_bis);
193
   assignin ('base', 'H_2', H_2);
assignin ('base', 'Tm_10', Tm_10);
assignin ('base', 'Tm_02', Tm_02);
194
195
196
197
```

198 end

G.4. Reliability functions

```
function [HyBC] = HydraulicBC (N)
1
  %% Calculate occurance of wind per direction in 16 bins
3
  % Load winddata from 1992-2000
4
  load ('C:\Users\Siemen\Documents\TU\Master\Afstuderen\06 Rapport\11 Matlab\03 Input
       excels\01 Input for Matlab\ncep_ws_wr_1992_2000.mat');
  ncep_time_ws_wr_1992_2000(ncep_time_ws_wr_1992_2000(:,2) < 10.967, :) = [];
6
  Widat = ncep_time_ws_wr_1992_2000(:,3);
7
  % Calculate occurance per wind bin, Sum of occurance should be one
9
  [WindOcr] = Winddata(Widat);
10
11
  % Define bin edges
12
  WindOcr = [0; WindOcr];
13
14
  % Rewrite pdf occurance to cdf occurance
15
  for i= 2:17
16
       WindOcr(1,2) = WindOcr(1,1);
17
       WindOcr(i,2) = WindOcr(i,1) + WindOcr(i-1,2);
18
19
  end
20
  % Load probability distributions of each bin
21
  Windbins = textread('C:\Users\Siemen\Documents\TU\Master\Afstuderen\06 Rapport\11
22
       Matlab\03 Input excels\01 Input for Matlab\Windbins.txt'); %#ok<DTXTRD>
23
  %% Calculate Windspeed and Wave attack
24
  % Pull random value to determine Windbin depending on occurance
25
  % Random choosen windbin also defines Wave attack
26
  HyBC = nan(size(N));
27
  SS = nan(size(N));
28
  Tide = nan(size(N));
29
  Seas = nan(size(N));
30
  GSLR = nan(size(N));
31
  g = 9.81;
32
33
  97% Construer Matlab Table met Hydaulic Boundary conditions
34
35
  for i = 1:N
36
37
       x = rand(1,1);
38
      \% x = 0.01;
39
40
       if WindOcr(1,2)<x && x<=WindOcr(2,2)
41
           j = 1; SS_f = -0.00153977; a = 0;
42
43
       elseif WindOcr(2,2) <x && x<=WindOcr(3,2)
44
           j = 2; SS_f = -0.001059507; a = 22.5;
45
46
       elseif WindOcr(3,2)<x && x<=WindOcr(4,2)
47
           j = 3; SS_f = -0.00048059; a = 45;
48
49
50
       elseif WindOcr(4,2)<x && x<=WindOcr(5,2)
           j = 4; SS_f = -0.000483851; a = 67.5;
51
```

52	
52	elseif WindOcr $(5, 2)$ < x & x <-WindOcr $(6, 2)$
55	i = 5: SS f = -0.001236867 ; $a = 90$:
54	j = 3, 35 - 1 = 0.001230007, a = 30,
55	also if WindOcr $(6, 2)$ < x & x <-WindOcr $(7, 2)$
56	i = 6, SS = 0.001751720, c = 112.5;
57	$J = 6; SS_1 = -0.001751789; a = 112.5;$
58	$A_{1} = \left\{ \begin{bmatrix} 1 \\ 0 \end{bmatrix} \\ A_{1} = \left\{ \begin{bmatrix} 1 \\ 0 \end{bmatrix} \\ A_{2} = \left\{ \begin{bmatrix} 1 \\ 0 \end{bmatrix}$
59	elself windOcr $(7,2)$ <
60	$f = 7; 55_f = 0.001433971; a = 135;$
61	
62	elseif WindOcr $(8,2)$ <x &&="" x<="WindOcr<math">(9,2)</x>
63	$j = 8; SS_t = 0.001215628; a = 157.5;$
64	
65	elseif WindOcr $(9,2)$ <x &="" x<="WindOcr<math">(10,2)</x>
66	$j = 9; SS_f = 0.000957657; a = 180;$
67	
68	elseif WindOcr(10,2) <x &&="" x<="WindOcr(11,2)</td"></x>
69	$j = 10; SS_f = 0.001038406; a = 202.5;$
70	
71	elseif WindOcr(11,2) <x &&="" x<="WindOcr(12,2)</td"></x>
72	j = 11; SS_f = 0.000864226; a = 225;
73	
74	elseif WindOcr(12,2) <x &&="" x<="WindOcr(13,2)</td"></x>
75	j = 12; SS_f = 0.00086635; a = 247.5;
76	
77	elseif WindOcr(13,2) <x &&="" x<="WindOcr(14,2)</td"></x>
78	$j = 13; SS_f = 0.001001418; a = 270;$
79	
80	elseif WindOcr(14,2) <x &&="" x<="WindOcr(15,2)</td"></x>
81	j = 14; SS_f = 0.001433971; a = 292.5;
82	
83	elseif WindOcr(15,2) <x &&="" x<="WindOcr(16,2)</td"></x>
84	j = 15; SS_f = 0.001185322; a = 315;
85	
86	elseif WindOcr(16,2) <x &&="" x<="WindOcr(17,2)</td"></x>
87	$j = 16; SS_f = -0.001515787; a = 337.5;$
88	
89	end
90	
91	% Define number of bin
92	HyBC(i,4) = j;
93	
94	% Pull random wind speed depending prob distribution
95	X = rand(1,1);
96	
97	% Windspeed
98	HyBC(i,1) = -Windbins(2,j)*log(1-X) + Windbins(1,j);
99	
100	% Wave angle
101	HyBC(1,2) = Windbins(3,j);
102	
103	
104	% Calculate wave height, give Windspeed and Fetch, with Brettschneider
105	% Wave height
106	$HyBC(1,3) = (0.283*tanh(0.578*(g*Windbins(5,j)/HyBC(i,1)^2)^{0.75})*$
107	tanh((0.0125*(g*Windbins(4,j)/HyBC(i,1)^2)^0.42)/(tanh(0.578*(g*Windbins(5,j

```
)/HyBC(i,1)^2)^0.75))))...
             *(HyBC(i, 1)^{2/g});
108
109
        HyBC(i, 6) = HyBC(i, 3) + HyBC(i, 3) * normrnd(0, 0.025);
110
111
112
      %% Calculate design water level
113
      % Mean sea level (0)
114
      MSL = -0.19;
115
116
      % Tide (1)+/- 0.05
117
       x1 = rand(1,1);
118
       Tide(i, 1) = (x1 - 0.5) * 0.1;
119
120
      % Seasonal (2) +/- 0.1
121
       x^2 = rand(1,1);
122
       Seas(i,1) = (x^2-0.5)*0.2;
123
124
      % Global Sea Level Rise (3) Linear
125
       x3 = rand(1,1);
126
       x4 = rand(1,1);
127
       GSLR(i,1) = (x3/200) * (x4*50);
128
129
      % Storm Surge (5) % Cress formule
130
       SS(i,1) = HyBC(i,1)^2 * SS_f;
131
132
      % Design water level = SUM(0-5)
133
      HyBC(i,5) = MSL+Tide(i,1)+Seas(i,1)+GSLR(i,1)+SS(i,1);
134
135
   end
136
137
   end
138
```

G.5. Wind occurrence

```
function [WindOcr] = Winddata(Widat)
  2
         % Bins of 22.5 degrees, with centers [0 22.5 ... 337.5]
 3
         % Replace all values larger than 348.5 with value-360, to make correct bins
  4
          WidatCor = zeros(size(Widat));
          for i = 1:length(Widat)
  7
                           if Widat(i)>348.5
  8
                                           WidatCor(i) = Widat(i) - 360;
                           else
10
                                           WidatCor(i) = Widat(i);
11
                          end
12
          end
13
14
         % Define 16 bin ranges (some values are larger than 360?,
15
         % placed in bin 348,75 - 11.25
16
          binedges = [-11.5 \ 11.25 \ 33.75 \ 56.25 \ 78.75 \ 101.25 \ 123.75 \ 146.25 \ 168.75 \ 191.25 \ 213.75 \ 123.75 \ 146.25 \ 168.75 \ 191.25 \ 213.75 \ 1100 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 1000 \ 10000 \ 1000\ \ 10000 \ 1000\ \ 1000\ \ 1000\ \ 1000\ \ 100
17
                          236.25 258.75 281.25 303.75 326.25 348.75];
18
         % Count number of bins and calculate occurance per wind direction
19
          bincounts = histc(WidatCor, binedges);
20
          totalcounts = sum(bincounts,1);
21
```

22

```
% Convert WindOcr to normalized count
23
  WindOcr = zeros(size(16));
24
  for ii = 1:16
25
       WindOcr(ii,1) = bincounts(ii,1)/totalcounts;
26
  end
27
28
  % Check carried out to see if occurance is 1 (100%) THRESHOLD
29
  [SumWindOc] = sum(WindOcr, 1);
30
31
  end
32
```

G.6. Seaside armour stability - Van der Meer formula

```
function [z_vdm] = Vandermeer(H_s,h,H_2,c_pld,c_sd,P,tan_a,g,Tm_02,Sd,N,c_pls,c_ss,
1
       A_at, x_cw, rhos, rhow, d_a, xi_s1)
  %% Limiting of number of waves by Rock Manual
2
3
   if (N>7500)
4
       N = 7500;
5
   end
6
7
  %% Check deep or shallow water
8
  % Deep or shallow water based on h > 3* Hstoe
9
10
       if (h > 3 * H_s)
11
           %disp('Deep Water')
12
13
           % Criticual value of the surf similarity parameter DW
14
           xi_crd = ((c_pld/c_sd) * P^0.31 * sqrt(tan_a))^{(1/(P+0.5))};
15
16
           % Surf similarity parameter DW
17
           xi_m = tan_a / (sqrt((2*pi*H_s) / (g*(Tm_02)^2)));
18
19
                if (xi_m < xi_crd)
20
                    %disp('Plunging Waves')
21
22
                    % Stability number DW Plunging Waves
23
                    Stn = c_pld * P^{0.18} (Sd/sqrt(N))^{0.2} (xi_m)^{(-0.5)};
24
25
                else
26
                    %disp('Surging Waves')
27
28
                    % Stability number DW Surging Waves
29
                     Stn = c_sd*P^{-0.13*}(Sd/sqrt(N))^{0.2*}sqrt(1/tan_a)*(xi_m)^{P};
30
                end
31
       else
32
         %disp('Shallow Water')
33
34
          % Criticual value of the surf smililarity parameter SHW
35
          xi_crs = ((c_pls/c_ss)*P^0.31*sqrt(tan_a))^(1/(P+0.5));
36
37
            if (xi_s1 < xi_crs)
38
               %disp('Plunging conditions')
39
40
               % Stability number SHW Plunging Conditions
41
                Stn = c_pls*P^{0.18*}(Sd/sqrt(N))^{0.2*}(H_s/H_2)*(xi_s1)^{(-0.5)};
42
```

```
43
44
            else
45
               %disp('Surging conditions')
46
47
                % Stability number SHW Surging Conditions
48
                Stn = c_ss*P^{-0.13*(Sd/sqrt(N))^{0.2*(H_s/H_2)*sqrt(1/tan_a)*(xi_s1)^{P};}
49
            end
50
       end
51
52
  %% Calculation of Dn50
53
54
       % Correction factor for oblique waves according to Wolters, max 70 degrees
55
       if (A_at < 70)
56
           beta_vdm = A_at;
57
        else
58
           beta_vdm = 70;
59
       end
60
61
       f = 1/(\cos(beta_vdm/360*2*pi)^x_cw);
62
63
       delta = rhos/rhow-1;
64
65
       % Calculation of Dn50
66
       z_vdm = d_a - H_s/(delta * Stn * f);
67
```

G.7. Rear-side armour stability - Van Gent and Pozueta

function [z_r] = Rearside (A_at,gamma_f,c1,c0,xi_s1,H_s,DHWL,cr_l,cr_lr,g,gamma_fc, B_c,d_r,Sd,Tm_10, delta,tan_ai,N,j)

```
2
  % Excluded directions from the south
3
   if j == 8||9||10||11||12||13;
4
       z_r = 1;
5
   else
6
  % Reduction for oblique wave attack
8
        if (A_at < 80)
9
            beta_ga = A_at;
10
        else
11
            beta_ga = 80;
12
       end
13
14
       gamma_b = 1 - 0.0022 * beta_ga;
15
       gamma = gamma_f*gamma_b;
16
17
       p = 0.5 * c1/c0;
18
       c2 = 0.25 * (c1^2/c0);
19
20
  % Determine fictious run-up
21
        if xi_s1 > p
22
            R_u = (c1 - (c2 / xi_s1)) * gamma * H_s;
23
        else
24
            R_u = c0 * xi_s 1 * gamma * H_s;
25
       end
26
27
       R_c = cr_l - DHWL;
28
```

```
R_cr = cr_lr - DHWL;
29
       R_test = R_u - R_c;
30
31
    % If not run up formula gives NaN replace with no failure = z_r > 0 = 1
32
   % Take abs of factor sd/sqrt(N), if b<0 b^(-1/6) is NaN
33
       if (R_test > 0)
34
35
           u_1 = (1.7*((g*gamma_fc)^{0.5})*((R_u1-R_c)/gamma_f)^{0.5})/(1+0.1*(B_c/H_s));
36
37
           % Include uncertainty via normal distribution with standard deviation of 0.1
38
           a = Sd/sqrt(N);
39
           b = abs(normrnd(a, 0.1));
40
41
           % z-function
42
            z_r = d_r - (0.008 * (b^{-1/6})) * (u_1 * Tm_{10} / sqrt(delta)) * (1 / tan_{ai})^{-2.5/6}
43
                *(1+10*\exp(-R_cr/H_s))^{(1/6)};
44
       else
45
            z_r = 1;
46
       end
47
48
    end
```

G.8. Toe stability - Van der Meer formula

```
function [z_t] = Toe (A_at, H_s, x_cg, d_t, h_toe, h, Nod, delta)
1
2
  %% Stability of Toe
3
   % Reduced wave height
4
        if (A_at < 45)
5
                beta_st = A_at;
6
            else
7
                beta_st = 45;
8
        end
9
10
    H_{sr} = H_{s*}(\cos(beta_{st}/360*2*pi))^x_cg;
11
12
  % Include uncertainty via normal distribution with standard deviation of 0.05
13
   a = 1-h_toe_h;
14
   b = abs(normrnd(a, 0.05));
15
16
  % Z-function for Stability of Toe
17
   z_t = d_t - H_sr / ((2+6.2*(b)^{2.7})*Nod^{0.15*}delta);
18
```

G.9. Macro stability - Bishop

```
1 function [z_bis] = Bishop (gamma_a_un,gamma_a_s,gamma_q_s,rhow,phi_a,phi_q,DWL)
2
3 %% Macro Stability Bisshop
4 % Taking governing slip circle from deterministic design, and examining 12
5 % parts on this slip circle
6
7 % Number of iterations (x) and starting point of iteration (F)
8 F(1,1) = 1;
9 x = 3;
10
11 % Input variables from basic variables
12 gamma_w = rhow * 9.81;
```

```
13 \quad c_a = 0;
           c_q = 0;
14
15
           for i = 1:x
16
17
           F = F(i, 1);
18
19
          9% Calculate all loads an resistance each part
20
21
          % Part 1
22
                              alpha = 6.4/360*2*pi;
23
                              h_a = 0.25;
24
                              h_q = 0;
25
                              p = 0;
26
27
                              Bis_L(1,1) = gamma_as * h_a * sin(alpha);
28
                              Bis_S(1,1) = (c_a + (gamma_a + h_a - p) + tan(phi_a)) / (cos(alpha) + (1 + (tan(alpha) + tan(alpha))) / (cos(alpha) + (tan(alpha) + tan(alpha))) / (cos(alpha)) / (cos(al
29
                                               phi_a)/F)));
30
          % Part 2
31
                              alpha = 9.2/360*2*pi;
32
                              h_a = 0.678;
33
                              h_q = 0;
34
                              p = 0;
35
36
                              Bis_L(2,1) = gamma_a_s * h_a * sin(alpha);
37
                              Bis_S(2,1) = (c_a + (gamma_a_*h_a-p)*tan(phi_a)) / (cos(alpha)*(1+(tan(alpha)*tan(alpha))) / (cos(alpha))) / (cos(alpha)) / 
38
                                               phi_a)/F)));
39
           % Part 3
40
                              alpha = 12/360*2*pi;
41
                              h_a = 0.9125;
42
                              h_q = 0.0865;
43
                              p = 0;
44
45
                              Bis_L(3,1) = gamma_as * h_a * sin(alpha) + gamma_qs * h_q * sin(alpha);
46
                              Bis_S(3,1) = (c_a + ((gamma_a s * h_a + gamma_q s * h_q) - p) * tan(phi_a)) / (cos(alpha))
47
                                               *(1+(\tan(alpha) * \tan(phi_a)/F)));
48
          % Part 4
49
                              alpha = 14.8/360*2*pi;
50
                              h_a = 0.969;
51
                              h_q = 0.28;
52
                              p = 0;
53
54
                              Bis_L(4,1) = gamma_a_s * h_a * sin(alpha) + gamma_q_s * h_q * sin(alpha);
55
                              Bis_S(4,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
56
                                               *(1+(\tan(alpha) * \tan(phi_a)/F)));
57
           % Part 5
58
                              alpha = 17.6/360*2*pi;
59
                              h_a = 0.969;
60
                             h_q = 0.457;
61
                              p = 0;
62
63
                              Bis_L(5,1) = gamma_as * h_a * sin(alpha) + gamma_qs * h_q * sin(alpha);
64
```

```
Bis_S(5,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
65
                           *(1+(\tan(alpha)*\tan(phi_a)/F)));
66
       % Part 6
67
                 alpha = 20.4/360*2*pi;
68
                 h_a = 0.969;
69
                 h_q = 0.5585;
70
                 p = 0;
71
72
                 Bis_L(6,1) = gamma_as * h_a * sin(alpha) + gamma_qs * h_q * sin(alpha);
73
                 Bis_S(6,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
74
                           *(1+(\tan(alpha)*\tan(phi_a)/F)));
75
       % Part 7
76
                 alpha = 23.2/360*2*pi;
77
                 h_a = 0.969;
78
                 h_q = 0.581;
79
                 p = 0;
80
81
                 Bis_L(7,1) = gamma_as * h_a * sin(alpha) + gamma_qs * h_q * sin(alpha);
 82
                 Bis_S(7,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
83
                           *(1+(\tan(alpha)*\tan(phi_a)/F)));
84
      % Part 8
85
                 alpha = 26/360*2*pi;
86
                 h_a = 0.969;
87
                 h_q = 0.519;
 88
                 p = 0;
 89
90
                 Bis_L(8,1) = gamma_a_s * h_a * sin(alpha) + gamma_q_s * h_q * sin(alpha);
91
                 Bis_S(8,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
92
                          *(1+(\tan(alpha) * \tan(phi_a)/F)));
93
       % Part 9
94
                 alpha = 28.8/360*2*pi;
 95
                 h_a = 0.969;
96
                 h_q = 0.367;
97
                 p = 0;
98
99
                 Bis_L(9,1) = gamma_a_s * h_a * sin(alpha) + gamma_q_s * h_q * sin(alpha);
100
                 Bis_S(9,1) = (c_a+((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a))/(cos(alpha))
101
                          *(1+(\tan(alpha) * \tan(phi_a)/F)));
102
       % Part 10
103
                 alpha = 31.6/360*2*pi;
104
                 h_a = 0.969;
105
                 h_q = 0.134;
106
                 p = 0;
107
108
                 Bis_L(10,1) = gamma_a_s * h_a * sin(alpha) + gamma_q_s * h_q * sin(alpha);
109
                 Bis_S(10,1) = (c_a + ((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a)) / (cos(alpha)) = (c_a + ((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a)) = (c_a + ((gamma_a_s*h_a+gamma_q_s*h_q) - p)*tan(phi_a)) = (cos(alpha)) = (cos(
110
                           *(1+(\tan(alpha)*\tan(phi_a)/F)));
111
       % Part 11
112
                 alpha = 34.4/360*2*pi;
113
                 h_a = 0.7355;
114
```

```
h_q = 0;
115
                        p = 0;
116
117
                         Bis_L(11,1) = gamma_a_s * h_a * sin(alpha) + gamma_q_s * h_q * sin(alpha);
118
                         Bis_S(11,1) = (c_a+((gamma_a_*h_a+gamma_q_*h_q) - p)*tan(phi_a))/(cos(alpha))
119
                                      *(1+(\tan(alpha) * \tan(phi_a)/F)));
120
         % Part 12
121
                        alpha = 37.2/360*2*pi;
122
                        h_{up} = -0.17;
123
                        h_down = -0.44;
124
125
                         if DWL > h_up
126
                                      h_wet = abs(h_down-h_up);
127
                                      h_dry = 0;
128
                         elseif DWL < h_down
129
                                      h_wet = 0;
130
                                      h_dry = abs(h_down-h_up);
131
                         else
132
                                      h_dry = abs(DWL-h_up);
133
                                      h_wet = abs(h_down-DWL);
134
                        end
135
136
                        p = h_wet * gamma_w;
137
138
                         Bis_L(12,1) = h_wet*gamma_a_s * sin(alpha) + h_dry*gamma_a_un * sin(alpha);
139
                         Bis_S(12,1) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p)*tan(phi_a)) / (cos(alpha)) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p)*tan(phi_a)) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p)) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p)) = (c_a + ((gamma_a_s*h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a_s)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a)h_wet+h_dry*gamma_a_un)-p) = (c_a + ((gamma_a)
140
                                      )*(1+(tan(alpha)*tan(phi_a)/F)));
141
          %% Total load and resistance
142
143
          Bis_L_t = sum(Bis_L(:,1));
144
          Bis_S_t = sum(Bis_S(:,1));
145
146
          F(i+1,1) = Bis_S_t/Bis_L_t;
147
148
          end
149
150
          z_{bis} = F(x+1,1);
151
152
          end
153
```

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List of Symbols

Symbol	Definition	Unit
В	Crest width	т
с	Cohesion of soil	Pa
<i>c</i> ₀	Coefficient in formula of Van Gent and Pozueta	-
c_1	Coefficient in formula of Van Gent and Pozueta	-
$c_{s,d}$	Surging deep water coefficient	-
$c_{pl,d}$	Plunging deep water coefficient	-
$C_{s,s}$	Surging shallow water coefficient	-
$c_{pl,s}$	Plunging shallow water coefficient	-
c_{pl}	Constant for plunging waves	-
$\dot{c_w}$	Friction factor	-
cr _l	Crest level seaside	m
cr _{lr}	Crest level rear-side	m
d	Water depth	m
dh	Wind driven water level set-up	m
D.H.W.L.	Design high water level	m + BS
D_{n50}	Nominal stone diameter	т
f_c	Reduction factor for physical correlation	-
fss	Factor between wind speed (u) and the set-up (dh)	-
F	1. Fetch length	т
	2. Stability number	-
F_{H_s}	Variational factor on the significant wave height	-
g	Standard gravity	$m s^{-2}$
ĥ	Water depth (in front of toe)	т
h_t	Depth of toe below water level	т
$H_{2\%}$	Wave height exceeded by 2%	т
H_s	Significant wave height	т
	Central estimate for H_s with	
$H_s^{t_L}$	a return period equal to the design life	т
	time (t_L) of the structure	
	Central estimate for H_s with	
$H_s^{3t_L}$	a return period equal to the 3 design life	т
U	time (t_L) of the structure	
<i>t</i> _	Central estimate for H_s for a	
$H_s^{\iota_{P_f}}$	return period based on the allowable failure	т
U	probability of the structure	
H_{s_c}	Correct H_s for oblique waves by Galland	т
H_{θ}	Correct H_s for oblique waves by Wolters	т
k_s	Coefficient in formula of PIANC	-
k_{α}	Coefficient in formula of PIANC	-
k_{β}	Coefficient in formula of PIANC	-
Ľ	Wave length	m
L_0	Deep water wave length	т

Symbol	Definition	Unit
Lsection	Total section length	т
Ν	1. Number of waves	-
	2. Number of Monte Carlo simulations	-
	3. Length-effect factor	-
N_f	Number of failed Monte Carlo simulations	-
N _{od}	Damage factor for toe	-
N_s	Stability number	-
$N_{s,\theta}$	Stability number for oblique wave attack	-
$N_{s,\perp}$	Stability number for perpendicular wave attack	-
p	Water pressure	Pa
Р	Notional permeability of the structure	-
P_f	Failure probability	
, ת	Failure probability over the	
P_{f,sys,t_L}	design lifetime t_l	-
D	Allowable failure probability	
P_M	(PM) per mechanisms	-
R	Resistance	$kg m s^2$
D	Crest level relative to still	U
R_c	water at seaward side	т
D	Crest free board relative to	
R _{c,rear}	water level at rear side	т
D	Fictitious run-up level exceed	
$R_{u1\%}$	by 1% of the waves	т
5	1. Wave steepness	-
	2. Solicitation	$kg m s^2$
S_d	Damage factor	-
Stn	Stability number	-
T_{m-10}	Spectral mean energy wave period	S
$T_{m0,2}$	Spectral wave period	S
1110,2	Design lifetime of the structure	vear
t _P	Return interval for design storm	vear
1 j U	Wind speed	$m s^{-1}$
www.	Maximum velocity at rear side of	
U107	the crest during	$m s^{-1}$
<i>w</i> 170	overtonning exceeded by 1% of the wayes	1110
11)	Failure space factor	-
W.	Wind speed	$m s^{-1}$
	Random drawn variable X from	111 5
Χ	uniform distribution	-
7\$	Reliability function	_
α	1 Slope	0
u	2 Influence coefficient level II reliability method	_
a	Angle of rear side slope	0
a rear	Angle of slip circle	o
a s	Angle of seaside slope	o
м sea В	Reliability index	_
ß	Angle of incoming wayes	- 0
μ γ	Insturated density armour layor	N/m^2
Y a _{un}	Saturated density armour lover	N/m^2
γa_s	Saturated density annour layer	$1N/m^2$
Yq_s	Saturated density quarry run	1N / M ²
Yс	Partial safety factor for cohesion	-
$\gamma_c u$	Parual safety factor for	-
	undrained shear strength	
ſſ	koughness of seaward slope	-

Symbol	Definition	Unit
γ_{f-c}	Roughness at the crest	-
γ_{H_s}	Partial safety factor for wave height	-
γ_{H_Z}	Partial safety factor in formula of PIANC	-
γr	Partial safety factor for strength	-
γs	Partial safety factor for load	-
$\gamma \phi$	Partial safety factor for	
	friction angle	-
Δ	Relative buoyant density	-
e	Threshold for wind speed	$m \ s^{-1}$
θ	Wave angle relative to the normal of the	0
	breakwater	
μ	Mean	-
μ_Z	Mean of reliability function	-
ξ_m	Surf similarity parameter	
	using mean wave period	-
ρ _a ir	Density of air	$kg m^{-3}$
$ ho_s$	1. Density of soil	$kg m^{-3}$
	2. Density of stone	$kg m^{-3}$
$\rho_w(ater)$	Density of water	$kg m^{-3}$
σ	Deviation	-
σ_Z	Deviation of reliability function	-
$\sigma'_{F_{H_s}}$	Deviation of variational coefficient wave height (F_{H_s})	-
ϕ	Internal friction angle	0
ϕ	Wave angle	0
ϕ_a	Angle of internal friction	0
	armour layer	
ϕ_q	Angle of internal friction	0
	guarry run	

Glossary

List of Acronyms

CEG	Faculty of Civil Engineering and Geosciences	
Level 0	Deterministic reliability method	
Level I	Semi-probabilistic reliability method	
Level II	Probabilistic with approximations reliability method	
Level III	Fully probabilistic reliability method	
Level IV	Risk based reliability method	
Mathimatical models	Models which describe the hydrodynamic processes	
	(SWAN and Delft3D) and geotechnical stability (D-Geo Stability)	
PROVERBS	Probabilistic Design Tools for Vertical Breakwaters	
SLS	Sevicable Limit State	
TU Delft	Delft University of Technology	
ULS	Ultimate Limit State	
W+B	Witteveen+Bos	